RECLAMATION

Managing Water in the West

Draft Feasibility-Level Engineering Report

Continued Phased Development of the Columbia Basin Project – Enlargement of the East Low Canal and Initial Development of the East High Area

Odessa Subarea Special Study

Columbia Basin Project, Washington

U.S. Department of the Interior
Bureau of Reclamation
Technical Service Center

October 2010

Mission Statements

The mission of the Department of the Interior is to protect and provide access to our Nation's natural and cultural heritage and honor our trust responsibilities to Indian Tribes and our commitments to island communities.

The mission of the Bureau of Reclamation is to manage, develop, and protect water and related resources in an environmentally and economically sound manner in the interest of the American public.

BUREAU OF RECLAMATION

Technical Service Center
Denver, Colorado

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Odessa Subarea Special Study
Columbia Basin Project, Washington

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Acronyms and Abbreviations

ºF	degree Fahrenheit
AASHTO	American Association of Highway Transportation Officials
ACC	Groundwater – Expansion
APS	Allowance for Procurement Strategies
ASCE	American Society of Civil Engineers
ASTM	American Society for Testing and Materials
AWWA	American Water Works Association
b	bottom width of canal
BRBC	Black Rock Branch Canal
C_v	gallons per minute that cause 1 psi loss through a fully open valve
CBP	Columbia Basin Project
CMP	corrugated metal pipe
CRBG	Columbia River Basalt Group
CRI MOU	Columbia River Initiative Memorandum of Understanding
D	inner diameter of pipe work (feet)
DEIS	Draft Environmental Impact Statement
ea	each
ECBID	East Columbia Basin Irrigation District
Ecology	Washington State Department of Ecology
EG	engine generator
e.g.	abbreviation for a Latin expression meaning “for example”
etc.	abbreviation for a Latin expression meaning "and other things" or "and so on"
EHC	East High Canal

EIS	environmental impact statement
El.	elevation
ELC	East Low Canal
ES	Executive Summary
ESA	Endangered Species Act
EQU	Equation – survey terminology
f	friction factor
FDR	Franklin Delano Roosevelt
ft	feet
ft/s	feet per second
ft^2/s	square feet per second
ft^2	square feet
ft^3	cubic feet
ft^3/s	cubic feet per second
$ft^3/ ft^2/day$	cubic feet per square feet per day
G	groundwater
g	acceleration due to gravity (ft/s^2)
gpm	gallons per minute
gpm/acre	gallons per minute per acre
h_f	hydraulic headloss (feet)
H	head, feet
HDPE	high-density polyethylene
HEP	Habitat Evaluation Procedure
HVAC	heating, ventilating, and air conditioning
I-90	Interstate Highway 90

K	loss coefficient based on velocity head ($V^2/2g$)
kV	kilovolt
kVA	kilovolt Ampere
L	length of pipe work (feet)
LRFD	Load and Resistance Factor Design
M	moment magnitude
MP	most probable
MPH	most probable high
MPL	most probable low
MVA	Mega Volt Ampere
n	coefficient of roughness
NAD83	North American Datum 1983
NAVD29	North American Vertical Datum 1929
NAVD88	North American Vertical Datum 1988
NEPA	National Environmental Policy Act
NMFS	National Marine Fisheries Service
Odessa DEIS	Odessa Subarea Special Study Draft Environmental Impact Statement
Odessa Subarea	Odessa Ground Water Management Subarea
O&M	Operations and Maintenance
OM&R	operation, maintenance, and replacement
PASS	Project Alternative Solution Study
PC	point of curvature
PGA	peak horizontal ground acceleration
PMF	Probable Maximum Flood
PMT	Project Management Team

POU	Point of Use
PRV	Pressure Reducing Valve
PSHA	Probabilistic seismic hazard analysis
psi	Pounds per square inch
psig	Pounds per square inch guage
PT	Point of Tangency
PVC	Polyvinyl chloride
Q	Flow rate, cubic feet per second
r	hydraulic radius or wetted perimeter
Reclamation	Bureau of Reclamation
S	surface water
SA	Spectral acceleration
SCADA	Supervisory Control and Data Acquisition
SCBID	South Columbia Basin Irrigation District
Secretary	Secretary of the Interior
SF-6	sulfur hexafluoride
Sta	station
State	State of Washington
Study	Odessa Subarea Special Study
TAPS	Computer software “Transient Analysis of Pipe Systems”
TDH	Total Design Head
TDH_{Max}	Maximum Total Design Head
TEFC	Totally-enclosed fan-cooled
TEWAC	Totally-enclosed water-to-air cooled
TRS	Township/Range/Section

V	Velocity of fluid (feet/second)
WDFW	Washington State Department of Fish and Wildlife
WDOT	Washington State Department of Transportation
WP1	Weather Protected 1
WR^2	Mass Moment of Inertia, Weight of revolving parts and the square of the radius of gyration
WSC	water service contract
WSCG	water service contract with groundwater backup
yd^3	Cubic yards
YFB	Yakima Fold Belt

Executive Summary

The Odessa Subarea Special Study (Study) is investigating replacing groundwater currently used for irrigation in the Odessa Ground Water Management Subarea with surface water as part of continued phased development of the Columbia Basin Project (CBP). The aquifer is declining to such an extent that crop irrigation is at risk, and domestic, commercial, municipal, and industrial uses and water quality are also threatened. In response to the public's concern about the declining aquifer and associated economic and other effects, Congress has funded the Bureau of Reclamation (Reclamation) to investigate the problem. The State of Washington has partnered with Reclamation by providing funding and collaborating on various technical studies.

Potential Actions

Reclamation can only deliver water to lands authorized to receive CBP water. Up to 140,000 groundwater-irrigated acres in the Study area are eligible to receive CBP surface water.

To develop comprehensive alternatives, the Study divided actions into:

- **Water Delivery Alternatives.** Water delivery alternatives consist of infrastructure such as canals, pipe laterals, pumping plants, and re-regulation reservoirs to convey and deliver surface water to the groundwater-irrigated lands. The alternatives involve either building a new East High Canal system, expanding and extending the existing East Low Canal system, or various combinations of the two systems.

- **Water Supply Options.** Water supply options consist of new or existing storage facilities in various combinations that could store the replacement surface water supply for use in the Odessa Subarea.

The alternatives can be combined in various configurations for full operational alternatives, which would include both water delivery and storage. Several water supply options may be needed to provide sufficient water supply for an alternative.

Water Delivery Alternatives

Three water delivery alternatives were examined:

- **Alternative 1—**No Action. The No Action Alternative is a requirement of the National Environmental Policy Act (NEPA) process. This report does not discuss this alternative since no engineering work was completed for this alternative.

- **Alternative 2—Partial-Replacement Alternative.** The Partial-Replacement Alternative includes enlarging the existing East Low Canal south of Interstate Highway 90 (I-90) and constructing a 2.5-mile extension[1] of the canal east toward Connell, Washington. This alternative includes constructing pumping plants and buried pipelines to deliver the water to the irrigated fields.

- **Alternative 3—Full-Replacement Alternative.** The Full-Replacement Alternative involves constructing the northern portion of a new East High Canal system (sized to 15-percent of the capacity of the original feasibility plan) and siphons and tunnels (sized to 100-percent of that capacity); enlarging the existing East Low Canal sections south of Weber Branch Siphon (near I-90); and constructing a 2.5-mile extension east towards Connell, Washington.

Table ES.1 shows the amount of water needed for each alternative and the number of acres that each alternative would supply.

Table ES- 1. Feasibility Alternatives and Estimated Water Supply Needs

Alternative	Estimated Water Supply Needs (Acre-Feet)	Estimated Groundwater-irrigated Lands to be Supplied Water (Acres)
1	0	0
2	176,343	57,000
3	347,137	102,600

Water Supply Options

Reclamation would need to divert additional Columbia River water greater than current CBP diversions to provide a replacement water supply for groundwater irrigation in the Study area. Reclamation has a 1938 "withdrawal" which set aside water to irrigate the remaining authorized acres of the CBP. However, Reclamation would need to comply with the NEPA, the Endangered Species Act (ESA), and other regulatory requirements and procedures before it could divert additional Columbia River water.

[1] The Odessa Draft Environmental Impact Statement (Draft EIS) indicates that the extension of the existing East Low Canal is 2.1 miles. This number is based on early engineering designs. The 2.5 miles indicated in this report reflects actual engineering layouts of the canal extension utilizing the latest topographic survey information. The additional 0.4 mile extension of the East Low Canal is not expected to pose additional substantive environmental impacts in the project area. During Washington State Department of Fish and Wildlife's (WDFW) Wildlife Survey and Habitat Evaluation Procedure (HEP) analysis, field reconnaissance was completed over a wider area than the proposed footprint of the project. The lands that would be affected by this proposed canal extension are generally disturbed by ongoing agricultural operations. Should the East Low Canal extension become part of a preferred alternative, additional data collection and analysis will be conducted if needed, to meet the requirements of NEPA and SEPA for the Final EIS.

This Study assumed that water from the Columbia River would be diverted in a manner that would not affect flow objectives identified by the National Marine Fisheries Service (NMFS) to benefit salmon and steelhead listed under the ESA.

Reclamation's water diversion strategy is to divert water in the fall months, storing it for later use during the irrigation season. The replacement supply could be provided by operating existing CBP storage sites differently or constructing new storage. The Study examined modifying operations at the following storage facilities:

- **Banks Lake.** Draw the existing lake to lower levels than current operations. In an average precipitation year, the maximum drawdown would be approximately 5.0 feet for the No Action Alternative; approximately 8.4 feet for the Partial-Replacement Alternative; and approximately 13.5 feet for the Full-Replacement Alternative. This report does not discuss this option since no engineering work was completed for this option.

- **Franklin Delano Roosevelt (FDR) Lake.** Draw the existing lake to lower levels than current operations. In an average precipitation year, the maximum drawdown would be approximately 11.0 feet for the No Action Alternative, approximately 11.5 feet for the Partial-Replacement Alternative, and approximately 13.2 feet for the Full-Replacement Alternative. Since this water supply option does not require the construction of any facilities, no engineering designs were completed; therefore, this water supply option is not discussed in this engineering report.

Another option for providing a replacement water supply would be to construct a new storage facility that could be filled in September and October for use in April through August.

- **Rocky Coulee Storage Facility.** The storage facility would have an active storage capacity of 109,315 acre-feet[2]. Water would be conveyed into the reservoir from the existing East Low Canal and then pumped back to the East Low Canal to serve downstream farmlands to the south.

Cost Estimates

Cost estimates were developed based on feasibility-level engineering designs and analyses, using available data and information. The designs were based on design data developed in previous Reclamation studies (completed between the 1960s

[2] The Odessa DEIS indicates that the active storage capacity of Rocky Coulee Reservoir is 117,900 acre-feet. This number is based on early engineering designs. The 109,315 acre-foot active storage capacity indicated in this report reflects a change in the design of the storage reservoir to address limited freeboard availability in the existing East Low Canal to cope with reservoir water surface elevations generated by the Probable Maximum Flood. To address this concern, the reservoir water surface elevation corresponding to the active storage pool was lowered. This, in turn, caused a lowering of the volume of the active storage of the reservoir.

and 1980s) supplemented with limited additional data. The design data collected for future studies may change future cost estimates significantly from those presented here.

These cost estimates encompass field costs (direct cost of materials and services for construction of facilities) and noncontract costs (which include land acquisition, realty services, investigations, development of designs and specifications, construction engineering and supervision, and environmental compliance).

Three total project cost estimates were developed for each water delivery alternative and water supply option to arrive at a range of most probable estimates (i.e., most probable low, most probable, and the most probable high). These costs are presented in Table ES- 2.

Table ES- 2. Total project cost estimates

Water Delivery Alternative or Water Supply Option	Most Probable Low Cost Estimate	Most Probable Cost Estimate	Most Probable High Cost Estimate
Alt. 1 – No Action	$0	$0	$0
Alt. 2 – Partial-Replacement	To be determined	$728,303,172	To be determined
Alt. 3 – Full-Replacement	To be determined	$2,582,408,581	To be determined
Banks Lake Drawdown	$0	$0	$0
FDR Lake Drawdown	$0	$0	$0
Rocky Coulee Storage Facility	To be determined	$276,186,850	To be determined

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Chapter 1: Introduction

The Bureau of Reclamation (Reclamation) is conducting the Odessa Subarea Special Study (Study) to investigate the continued phased development of the Columbia Basin Project (CBP) to replace groundwater currently used for irrigation in the Odessa Ground Water Management Subarea (Odessa Subarea) with CBP surface water. Reclamation has completed feasibility-level investigations of three water delivery alternatives (including a No Action Alternative) and three water supply options that could provide a replacement surface water supply. The categories and options include constructing a new canal system or enlarging and expanding existing canals, as well as possibly constructing new storage facilities. The investigations examined the engineering viability and developed feasibility-level cost estimates of the proposed alternatives and options. This report documents these feasibility investigations.

1.1. Study Authority

The CBP is a multipurpose water development project in the central part of the State of Washington (State). The key structure, Grand Coulee Dam, is on the mainstem of the Columbia River about 90 miles west of Spokane, Washington. The Grand Coulee Dam Project was authorized for construction by the Act of August 30, 1935, and reauthorized and renamed in the Columbia Basin Project Act of March 10, 1943. Congress authorized the CBP to irrigate a total of 1,029,000 acres; about 671,000 acres are currently irrigated.

The 1943 Columbia Basin Project Act subjected the CBP to the requirements of the Reclamation Project Act of 1939. Section 9(a) of the 1939 Act gave authority to the Secretary of the Interior (Secretary) to approve a finding of feasibility and thereby authorize construction of a project upon submitting a report to the President and the Congress. The Secretary approved a plan of development for the Columbia Basin Project (Reclamation, 1944), which was then transmitted as a joint report, known as House Document No. 172, to the President and to the House Irrigation and Reclamation Committee in 1945, thereby satisfying these requirements. (When the Secretary recommended a project to Congress, the feasibility report and Reclamation's Regional Director's report were customarily printed as a House Document.) The Odessa Subarea Special Study is conducted under the authority of this Act, as amended, and the Reclamation Act of 1939.

Congress authorized the continued irrigation development of the CBP using a phased development approach. House Document No. 172 anticipated about a 70-year period of incremental development to complete the CBP. Reclamation is authorized to implement additional phases as long as the Secretary finds it to be economically justified and financially feasible.

This Study is a special study investigating another developmental phase of the CBP. The Study requires a feasibility-level analysis, as it is anticipated that the Office of Management and Budget and other decisionmakers may require this level of analysis before appropriations for new construction will be made. Further, this study approach will help the Secretary determine the financial and economic feasibility of a preferred alternative as stipulated in current contract provisions with CBP beneficiaries.

1.2. Purpose and Need

Groundwater in the Odessa Subarea is currently being depleted to such an extent that water must be pumped from great depths. Pumping depths are 750 feet in some areas, and well depths are as great as 2,100–2,400 feet. Well drilling costs and pumping water from this depth have resulted in expensive power costs and water quality concerns such as high water temperatures and high sodium concentrations. The ability of farmers to irrigate their crops is at risk. Domestic, commercial, municipal, and industrial uses and water quality are also affected. Those irrigating with wells of lesser depth live with uncertainty about future well production.

Washington State University conducted a regional economic impact study assessing the effects of lost potato production and processing in Adams, Franklin, Grant, and Lincoln Counties from continued aquifer decline. Assuming that all potato production and processing is lost from the region, the analysis estimated the regional economic impact would be a loss of about $630 million dollars annually in regional sales, a loss of 3,600 jobs, and a loss of $211 million in regional income (Bhattacharjee and Holland, 2005).

Another study examined the regional economic impacts for Adams and Lincoln counties from possible agricultural production losses for other crops that might result with continued aquifer decline (Razack and Holland, 2007). Two scenarios were examined. One scenario assumed a 10 percent reduction in agricultural production would occur with an estimated $20 million reduction in regional income and a 295 job loss for the two counties (Razack and Holland, 2007). A second scenario assumed a 10 percent crop production loss combined with loss of the frozen potato processing product in the two counties would occur with an estimated $30 million loss of regional income and a 465 job loss for the two counties. If all deep well agricultural production was lost, an estimated 4,650 jobs would be lost, equating to about 32 percent of total jobs in the two counties.

Action is needed to avoid significant economic loss to the region’s agricultural sector because of resource conditions associated with continued decline of the aquifers in the Odessa Subarea. The purpose of actions proposed in this report is to meet this need by replacing the current and increasingly unreliable groundwater supplies with a surface supply from the CBP as part of continued phased development of the CBP as authorized.

1.3. Study Background

As mentioned previously, the CBP is authorized to irrigate 1,029,000 acres; about 671,000 acres (approximately 65 percent of the acreage authorized by Congress) are currently irrigated. These lands, known as first half lands, were developed primarily in the 1950s and 1960s, with some acreages being added sporadically until 1985. Prior studies examined the merits of continuing the incremental development approach for the CBP. However, for various reasons, development did not occur.

The State issued irrigation groundwater permits in the 1960s and 1970s in the Odessa Subarea as a temporary measure until the CBP was developed to provide surface water to these lands. The aquifer has now declined to such an extent that the ability of farmers to irrigate their crops is at risk and domestic, commercial, municipal, and industrial uses and water quality are affected. Local constituents have advocated that Reclamation investigate CBP development to replace groundwater with CBP water as a possible solution for issues associated with the declining aquifer. In response to public concern about associated economic and other effects, Congress provided funding to Reclamation beginning in fiscal year 2005 to investigate opportunities to provide CBP water to replace groundwater use in the Odessa Subarea.

The State supports investigation of CBP development to provide a replacement for current groundwater irrigation. The State, Reclamation, and the CBP irrigation districts signed the Columbia River Initiative Memorandum of Understanding (CRI MOU) in December 2004, to promote a cooperative process for implementing activities to improve Columbia River water management and water management within the CBP. The Odessa Subarea Special Study implements Section 15 of the CRI MOU, which states in part that, “The parties will cooperate to explore opportunities for delivery of water to additional existing agricultural lands within the Odessa Subarea.” The State provided a cost-share through an Intergovernmental Agreement between Washington State Department of Ecology (Ecology) and Reclamation in December 2005 to fund this Study.

In February 2006, the State legislature passed the Columbia River Water Resource Management Act (HB 2860) that directs Ecology to aggressively pursue development of water benefiting both instream and out-of-stream uses through storage, conservation, and voluntary regional water management agreements. Reclamation’s Odessa Subarea Special Study is one of several activities identified in the legislation.

1.4. Previous Study-Related Investigations

Reclamation began the Odessa Special Study in 2005. A *Plan of Study* (Reclamation, 2006a) was first published that provided study background and purpose, described potential issues, outlined study steps and requirements, and identified required resources.

Reclamation completed a pre-appraisal-level investigation through a Project Alternative Solutions Study (PASS) late in 2006. The investigation is documented in a report entitled, *Initial Alternative Development and Evaluation, Odessa Subarea Special Study* (Reclamation, 2006c).

Reclamation then completed an appraisal-level investigation late in 2007. The investigation is documented in a report entitled, *Odessa Subarea Special Study – Appraisal Study – Report of Findings* (Reclamation, 2007d).

1.5. Scope of Feasibility Study

This feasibility study looked at three water delivery alternatives and three water supply options, either individually or in combinations, which are summarized below:

1.5.1 Water Delivery Alternatives

1.5.1.1. Alternative 1 – No Action

The No Action alternative is a requirement of the National Environmental Policy Act (NEPA) which dictates that completion of an Environmental Impact Study (EIS) for a project must include an option where no action is undertaken. Since this alternative does not require the construction of any facilities, no engineering designs were completed and therefore are not discussed in this engineering report.

1.5.1.2. Alternative 2 – Partial Groundwater Irrigation Replacement

This alternative focuses on delivering water to those groundwater-irrigated fields within the Study area that are south of Interstate 90 (I-90) and east of the existing East Low Canal (ELC). The original plan for this project assumed that these lands would be served by the proposed East High Canal (EHC) that would be constructed along the eastern boundary of the Study area and would provide water to these lands by gravity. This alternative differs from the original plan in that water would be delivered to these lands from the existing East Low Canal via pressurized pipeline systems radiating eastward from the existing canal (Figure 1- 1).

This alternative involves enlarging the existing ELC south from the existing Weber Branch Siphon (near I-90) and extending the canal from its terminus near Scooteney Wasteway approximately 2.5 miles[3] towards Connell, Washington. This alternative supplies water to 64,757 acres (56,789 groundwater-irrigated acres plus 7,968 acres associated with Water Service Contracts)[4].

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[3] The Odessa Draft Environmental Impact Statement (Draft EIS) indicates that the extension of the existing East Low Canal is 2.1 miles. This number is based on early engineering designs. The 2.5 miles indicated in this report reflects actual engineering layouts of the canal extension utilizing the latest topographic survey information. The additional 0.4 mile extension of the East Low Canal is not expected to pose additional substantive environmental impacts in the project area. During Washington State Department of Fish and Wildlife's (WDFW) Wildlife Survey and Habitat Evaluation Procedure (HEP) analysis, field reconnaissance was completed over a wider area than the proposed footprint of the project. The lands that would be affected by this proposed canal extension are generally disturbed by ongoing agricultural operations. Should the East Low Canal extension become part of a preferred alternative, additional data collection and analysis will be conducted if needed, to meet the requirements of NEPA and SEPA for the Final EIS.

[4] The intent of the Odessa Subarea Special Study is to look at providing Columbia River surface water to groundwater-irrigated lands that are located within the project boundary. During the initial stages of the feasibility study the East Columbia Basin Irrigation District requested that the water delivery alternatives

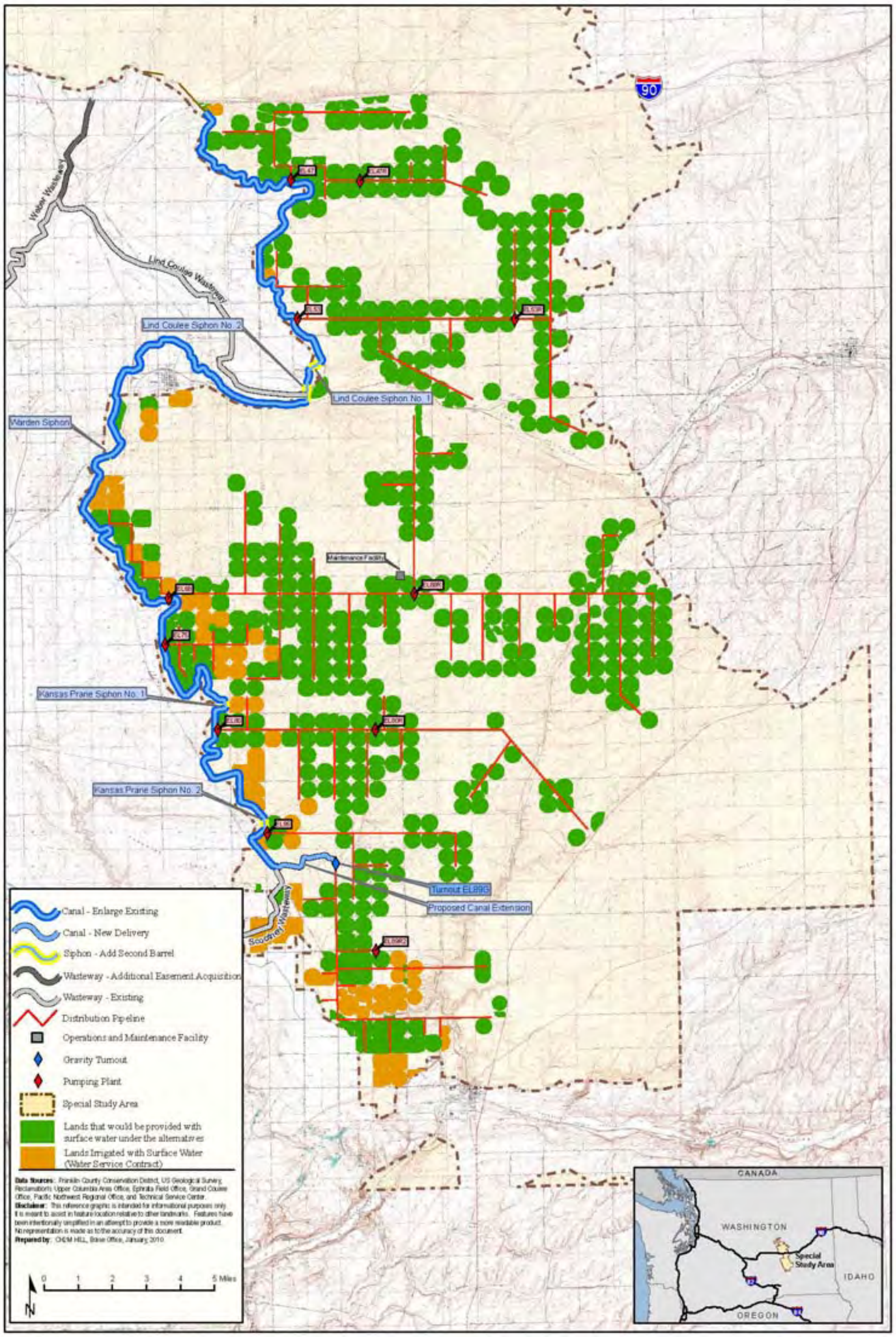

Figure 1- 1. Water Delivery Alternative 2 (also the southern component of Water Delivery Alternative 3)

also provide water to existing Water Service Contracts that currently obtain water directly from the East Low Canal as long as it is economically viable. The engineering designs discussed in this report include most of these additional Water Service Contract acres and hence the total acreage reported in this report does not match the values reported in the EIS. This applies to both Alternatives 2 and 3.

- Construction of the northern portion of the proposed EHC and Black Rock Branch Canal north of I-90 and construction of a re-regulation reservoir in Black Rock Coulee.

- Enlargement of the existing ELC south from Weber Coulee Siphon to Scooteney Wasteway. Includes constructing a second barrel for each of the existing siphons.

- Extension of ELC east approximately 2.5 miles.

- Constructing canal-side and booster pumping plants to raise the water from the canals to higher-elevation lands east of the canals.

- Constructing buried pressurized pipelines from the canals eastward to the groundwater-irrigated lands. Includes regulating tanks, valves, flowmeters, etc., that are necessary to make the pipelines functional.

1.5.2 Water Supply Options

1.5.2.1. Banks Lake Drawdown

This water supply option involves changing current operations of the Banks Lake facility to draw the existing lake to lower levels than permitted to meet the water requirement for each water delivery alternative. In an average precipitation year, the maximum drawdown would be approximately 5.0 feet for the No Action Alternative, approximately 8.4 feet for the Partial-Replacement Alternative, and approximately 13.5 feet for the Full-Replacement Alternative. Since this water supply option does not require the construction of any facilities, no engineering designs were completed; therefore, this water supply option is not discussed in this engineering report.

1.5.2.2. Franklin Delano Roosevelt (FDR) Lake Drawdown

This water supply option involves changing current operations of the Grand Coulee facility to draw the existing lake to lower levels than permitted to meet the water requirement for each water delivery alternative. In an average precipitation year, the maximum drawdown would be approximately 11.0 feet for the No Action Alternative, approximately 11.5 feet for the Partial-Replacement Alternative, and approximately 13.2 feet for the Full-Replacement Alternative. Since this water supply option does not require the construction of any facilities, no engineering designs were completed; therefore, this water supply option is not discussed in this engineering report.

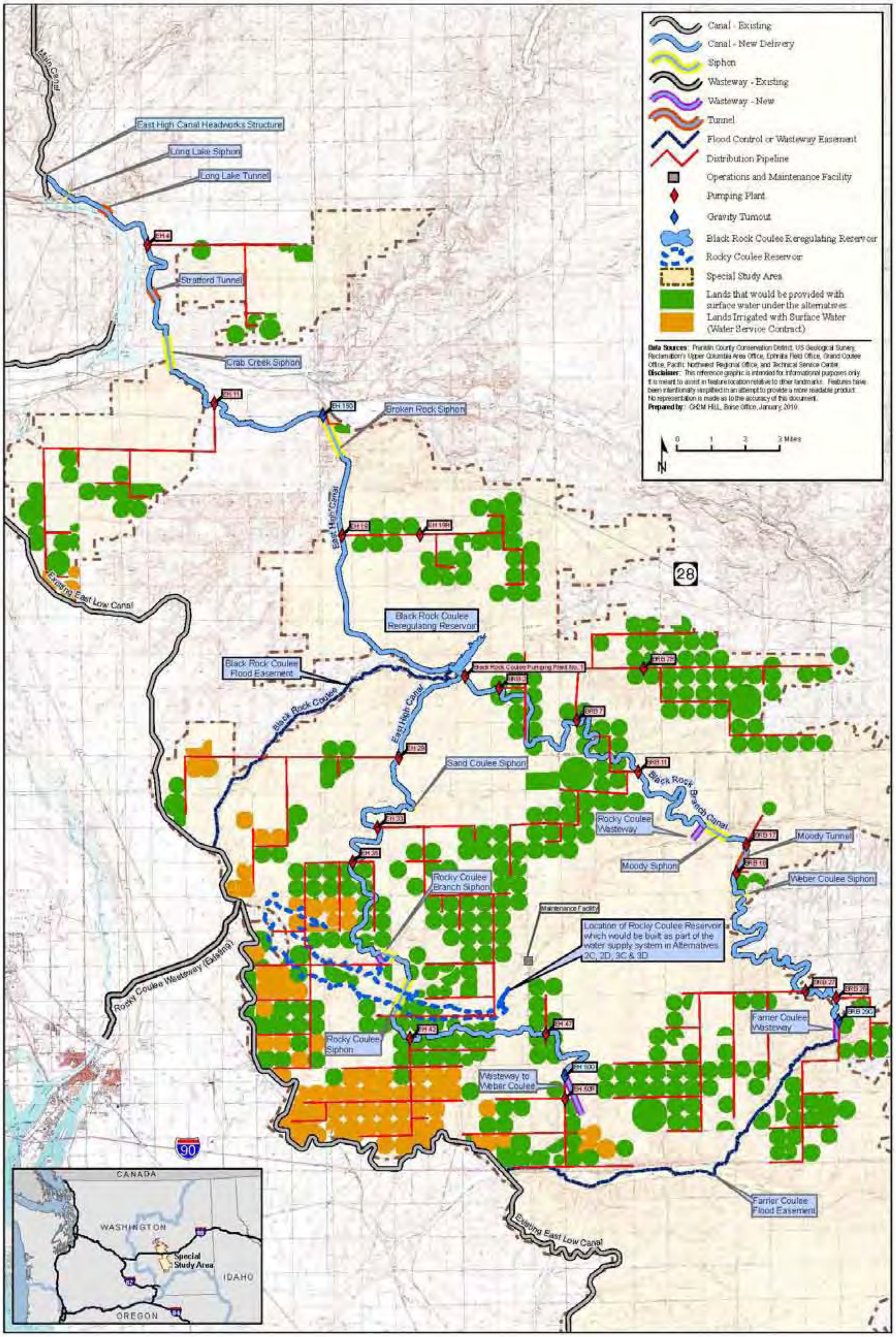

Figure 1- 2. Detail of Water Delivery Alternative 3 north of I-90

1.5.2.3. Rocky Coulee Storage Facility

This water supply option (
Figure 1- 3) would involve constructing a new storage facility in Rocky Coulee immediately east of the existing East Low Canal that would be filled in September and October for use in April through August. This reservoir would have an active storage capacity of 109,315 acre-feet[5]. Water from the existing East Low Canal would be conveyed by gravity into the reservoir and then pumped back into the East Low Canal to serve downstream farmlands to the south.

Major components of this option include:

- Constructing a zoned earthfill embankment dam including a low-level reservoir outlet works.
- Constructing an inlet/outlet channel constructed from the existing East Low Canal to the reservoir.

Major components of Alternative 2 include:

- Enlargement of the existing ELC south from Weber Coulee Siphon to Scooteney Wasteway. Includes constructing a second barrel for each of the existing siphons.
- Extension of ELC east approximately 2.5 miles.
- Constructing canal-side and booster pumping plants to raise the water from the canal to higher-elevation lands east of the canal.
- Constructing buried pressurized pipelines from the canal eastward to the groundwater-irrigated lands. Includes regulating tanks, valves, flowmeters, etc. that are necessary to make the pipelines functional.

1.5.2.4. Alternative 3 – Full Groundwater Irrigation Replacement

This alternative is essentially the preferred alternative brought forward from the 2007 appraisal-level study. This alternative focuses on delivering water to groundwater-irrigated fields within the Study area that are south of Summer Falls and east of the existing Main and East Low Canals. This alternative would construct the northern portion (Figure 1- 2) of the proposed East High Canal (EHC) system to supply approximately 55,900 acres (45,545 groundwater-irrigated acres plus 10,355 acres associated with Water Service Contracts) and to enlarge the existing East Low Canal

[5] The Odessa DEIS indicates that the active storage capacity of Rocky Coulee Reservoir is 117,900 acre-feet. This number is based on early engineering designs. The 109,315 acre-foot active storage capacity indicated in this report reflects a change in the design of the storage reservoir to address limited freeboard availability in the existing East Low Canal to cope with reservoir water surface elevations generated by the Probable Maximum Flood. To address this concern, the reservoir water surface elevation corresponding to the active storage pool was lowered. This, in turn, caused a lowering of the volume of the active storage of the reservoir.

(ELC) south from Weber Branch Siphon (near I-90) and extend ELC 2.5 miles toward Connell, Washington (Figure 1- 1), to supply approximately 64,757 acres (56,789 groundwater-irrigated acres plus 7,968 acres associated with Water Service Contracts). This alternative is capable of supplying water to 120,657 acres (102,334 groundwater-irrigated acres plus 18,323 acres[6] associated with Water Service Contracts). Water would be delivered to these lands via pressurized pipeline systems radiating from the canals.

Major components of Alternative 3 include:

- Construction of a headworks structure where the proposed EHC ties into the existing Main Canal above Summer Falls.
- Constructing a reservoir inlet structure.
- Constructing a pumping plant and switchyard.
- Constructing a pump discharge structure.

[6] The Odessa DEIS indicates that a total of 16,864 acres of land associated with "Water Service Contracts" are included in the acres of land within the Subarea that will receive water if this project is constructed. The number of acres shown in the DEIS are based on preliminary information. The project as currently envisioned in this feasibility-level study would provide sufficient water to service 18,323 acres of land associated with Water Service Contracts in addition to the groundwater-irrigated lands which are the focus of this study. This latest value is based on review of available data and actual feasibility-level engineering designs.

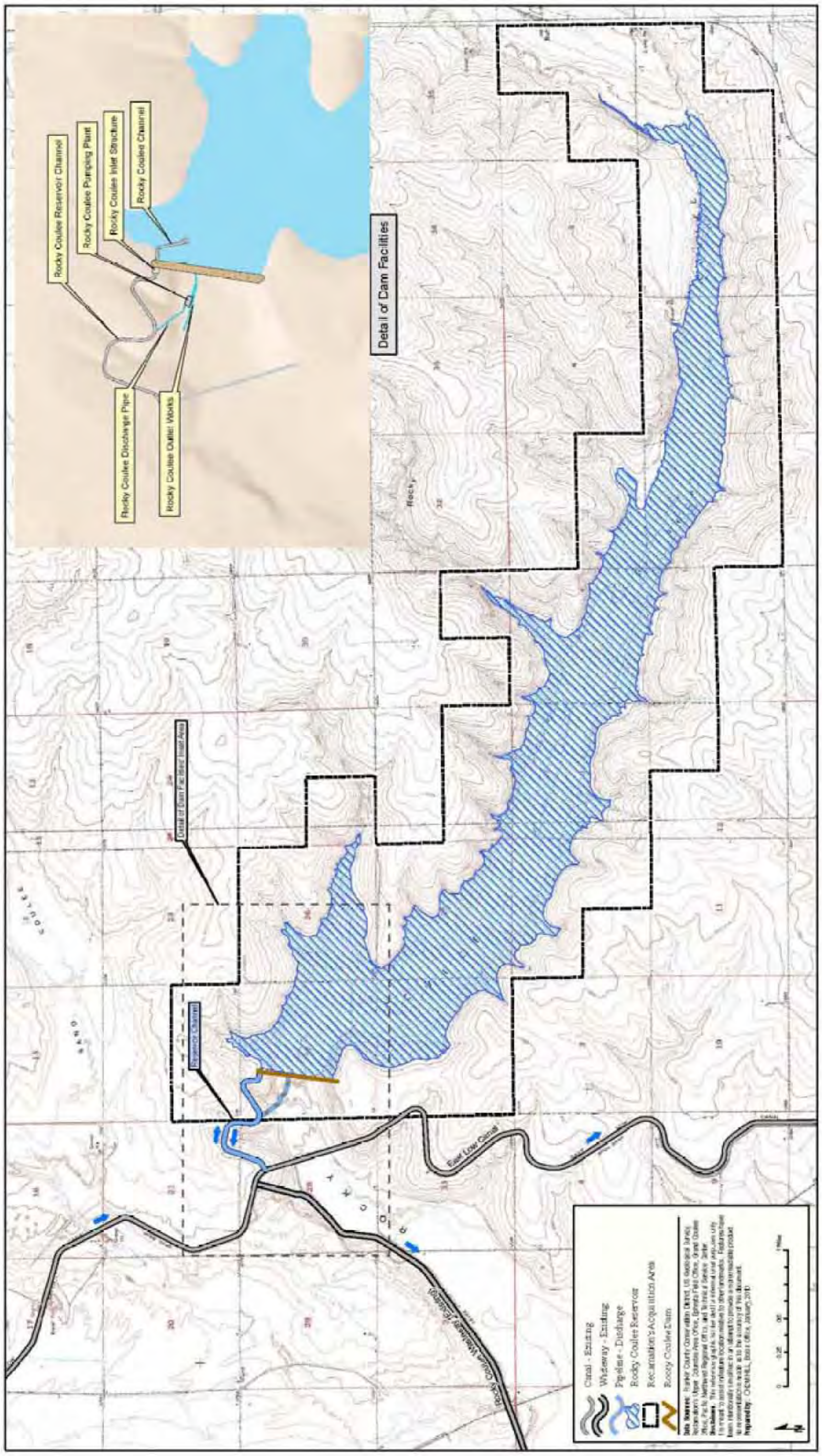

Figure 1- 3. Rocky Coulee Storage Facility

Chapter 2: Water Conveyance Features

Water conveyance features of an irrigation project are those components that are used to move water from a water source such as a lake, reservoir, river, stream, etc., to project lands that are to be irrigated. These features form a system that utilizes canals, siphons, tunnels, pipelines, and pumping plants to deliver and distribute the water to the irrigated fields included. This section documents the engineering design of these features.

2.1. Design Criteria and Data

The engineering designs completed in this feasibility study are based on basic design data and criteria that were established at the beginning of the Study. These basic design criteria are presented below.

2.1.1 Design Criteria

During the early stages of the feasibility design, the Project Management Team (PMT), which is comprised of key personnel from Reclamation, Washington State Department of Ecology (Ecology), the East Columbia Basin Irrigation District (ECBID), and the South Columbia Basin Irrigation District (SCBID), established an overall design requirement that the engineering designs developed in this study not compromise the ability of the project, at full development, to deliver water to the maximum authorized acreage of 1,029,000 acres.

With regard to the feasibility design of the proposed East High Canal (EHC) and Black Rock Branch Canal (BRBC), the PMT established an additional requirement that all key structures on these proposed canals be designed to their ultimate project development capacity. Structures for which this requirement applies are the EHC headworks, the Black Rock Coulee Dike, canals constructed completely in embankment, siphons, tunnels, canal inlet structures, canal outlet structures, and canal check structures.

Previously completed feasibility studies assumed lands higher in elevation than the existing East Low Canal (ELC) were to be served by the proposed EHC. However, for this study, the PMT established a requirement that those lands defined as “East High Canal serviced lands” that are south of I-90 are to be serviced through a network of pumping plants and pipe laterals constructed from the ELC (Alternative 2 or the southern portion of Alternative 3). For this feasibility study, these EHC lands that are south of I-90 are now referred to as the East Low Area.

As originally constructed, the ELC south of I-90 was not constructed to its ultimate capacity. For this feasibility study, the ELC south of I-90 would be enlarged so that it will have an ultimate capacity capable of serving 385,500 acres.

Another requirement was that the design team should utilize, as much as possible, design data developed for all previously completed studies dating back to the 1930s when the project was first envisioned. This data is currently stored in Reclamation's Ephrata Field Office and contains original canal layout drawings, soil analysis, geology logs and reports, engineering designs and drawings, study reports and documentation, etc. Where required, additional geologic explorations were completed prior to the start of this Study.

2.1.2 Design Data

In the early stages of the feasibility study, aerial surveys were completed to be used in the development of feasibility-level topography with 2-foot contours. These surveys also produced high- resolution aerial photographs that were used in this feasibility study to determine potential routings of canals and pipelines to avoid structures or terrain that would be difficult to construct through.

Survey controls for this study are NAD83, Washington South, for horizontal control and NAVD88 for vertical control. All previous studies completed for this project were performed using local horizontal control and NAVD29 vertical control.

Reclamation, with input from Ecology, ECBID, and SCBID, established which fields within the Study area would be serviced by the Water Delivery Alternatives developed in this feasibility study. The final data developed was provided to the design team and included information on:

1. Field identification number,
2. Irrigation type or category,
3. Number of acres,
4. Township/Range/Section (TRS) location information, and
5. X and Y coordinates.

There are 45,545 groundwater-irrigated acres north of I-90 and 57,069 groundwater-irrigated acres south of I-90, for a total of 106,614 groundwater-irrigated acres (these values do not include Water Service Contracts). The term "Irrigation Type or Category" refers to the water source used to irrigate particular fields, which are defined below:

Table 2- 1. Irrigation Category/Water Source Definitions

Irrigation Category	Water Source
G	Groundwater
ACC	Groundwater-Expansion. Fields designated as ACC are not included in a Point of Use (POU) permit, but can receive groundwater irrigation from a permitted POU when there is excess/surplus water on that POU (relative to what is being raised that season). There is not an increase in the amount of water above what the original POU permit allows.
WSCG	Water Service Contracts issued by the District **with** groundwater backup. These contracts allow individual farms to pump water directly out of the East Low Canal instead of from a groundwater source. These particular contracts **have** groundwater permits in place that would permit the farmer to pump groundwater if he is no longer permitted to pump from the ELC for whatever reason.
WSC	Water Service Contracts issued by the District **without** groundwater backup. These contracts allow individual farms to pump water directly out of the East Low Canal instead of from a groundwater source. These particular contracts **do not have** groundwater permits in place that would permit the farmer to pump groundwater if he is no longer permitted to pump from the ELC for whatever reason.
S	Surface water

The primary goal of the Study is to provide surface water to groundwater-irrigated lands that are located within the project boundary. During the initial stages of the feasibility study, the manager of the ECBID requested that the water delivery alternatives also provide water to existing Water Service Contracts that currently obtain water directly from the East Low Canal, as long as it is economically viable. The reason behind this request is that currently, the operation of these Water Service Contracts by individual farms causes some operational problems for the District. It is reasoned that if these contracts were to be included, then overall operational control of the system should improve. Since water for these contracts has already been allocated from existing authorized supplies, there would not be an increase in the water requirement for the water delivery alternatives. Refer to Appendix C for a current listing of these contracts.

Table 2- 2 is a summary of the acreage serviced by the water delivery alternatives developed in this feasibility study. Please note that the column labeled “Fields South of I-90” corresponds to the acreage served by Water Delivery Alternative 2 and the column labeled “All Fields” corresponds to the acreage served by Water Delivery Alternative 3.

Table 2- 2. Acreage served by water delivery alternatives

Irrigation Category	Fields North of I-90	Fields South of I-90	All Fields
G	43,294	55,280	98,574
ACC	2,251	1,509	3,760
WSCG	5,943	4,565	10,508
WSC	4,412	3,403	7,815
Totals	55,900	64,757	120,657

Not all acres included in the data provided are served by the water delivery alternatives developed in this feasibility study (Table 2- 3). Fields in the East High Area and East Low Area that are designated as being in the "S" (surface water) category are not served because they do not have a groundwater use or right. Six fields south of I-90 were also not supplied water in this Study due to various reasons (see Table 2- 4).

Table 2- 3. Acreage not served by water delivery alternatives

Irrigation Category	Fields North of I90	Fields South of I90	All Fields
S	787	2,564	3,351
G	0	280	280
WSCG	0	94	94
Totals	787	2,938	3,725

Table 2- 4. Irrigated fields not included in feasibility designs

Irrigation Category	Field No.	Acres	Reason for not supplying water
G	1,190	128	Not serviced due to isolation from canal and economics
G	993	139	District to use canal-side pump
G	225	3	District instructed designers that service is not required
G	228	11	District instructed designers that service is not required
WSCG	226	64	District to continue use of existing canal-side pump
WSCG	227	30	District to continue use of existing canal-side pump

For this feasibility study, the design flow at the beginning of the proposed East High Canal is 1,102 cubic feet per second (ft^3/s) versus a peak flow of 6,248 ft^3/s that would be required for the full development of the East High Canal portion of the project. The design flow decreases along the canal length as deliveries are made to lands as depicted on the drawings, and also identified in Table 2- 11 and Table 2- 12 (refer to Section 2.4, "Pipelines").

Peak water flow rate values were agreed to following several discussions held in September 2008, between Reclamation, ECBID, and SCBID. The water demand design criteria used for the feasibility study is an annual water allotment of 3.0 acre-feet per acre and a peak delivery rate as shown in Table 2- 5 and Table 2- 6.

Table 2- 5. Peak delivery rate per acres served

Number of Acres Irrigated (acres)	Peak Delivery Rate (gpm/acre)
1,000 or less	8.5
5,000 and greater	6.75

Note: Use straight line interpolation between the two values shown.

Table 2- 6. Peak flow rate by crop type

	Farm Efficiency	Peak Flow Rate			Available for Monthly Crop Uses (inches)
		inches/day	gpm/acre	Acre per ft^3/s	
Single crop	80%	0.42	8.5	52.8	10.2
Diversified Crop	NA	NA	6.75	66.5	NA

These values were based upon present irrigation usage by sprinkler systems in the project area. Typical sprinkler systems apply water at a rate of 7.5 gpm/acre. The flexibility for sublateral areas of 1,000 acres and less to increase the flow rate to 8.5 gpm/acre will facilitate higher consumptive use crops and/or more porous soil types. When the lateral is distributing to an area of 5,000 acres and greater, the lateral will be sized to provide an average rate of 6.75 gpm/acre. This assumes that up to 10 percent of the area may not be taking delivery during the peak period, and the typical sprinkler would be at the rate of 7.5 gpm/acre. Refer to Appendix F for a more in-depth discussion of the water demand design criteria used in this feasibility study.

Canals in the East High Canal area will be lined in accordance with recommendations documented in Appendix Vol. VI A (revised March 1966) of the 1960s feasibility report [Reclamation, 1966b]. These recommendations were based upon somewhat limited geologic exploration which produced an evaluation that large water losses could be expected through the fractured rock and vertical permeability of the loessal soils

The East Low Canal section will remain unlined, as a general rule, as seepage water will eventually be recaptured in Potholes Reservoir for reuse. However, ECBID requested that short sections of the canal (such as sections over cross-drainage culverts and in thorough fill sections) be lined as normal practice in order to reduce the risk of breaching. ECBID has experienced failures at these types of locations in the past. Also, other areas identified to be lined were sections with potentially high seepage rates due to fractured rock or very porous soils which may cause problems to crop production in adjacent fields.

For this feasibility study, some portions of the East Low Canal that were previously (mid-1960s) earth-lined will be concrete lined for cost estimating purposes. It is felt the earth lining has most probably deteriorated over the years and the expansion efforts would remove about half of the section.

Seepage and operational waste rates were estimated based on studies conducted for ECBID for existing canals and irrigation areas (Montgomery, 1995; 2003; 2004a; 2004b). The waste was assumed to be 30 ft^3/s per wasteway site when calculating canal capacity. Canal seepage was estimated at the rate of 0.1 ft^3/ft^2/day for a lined (concrete or compacted earth) canal section.

Proportioning of East High Canal sections was based on Reclamation guidelines.

- The guides were adapted for rock excavation sections. It appeared using 2/3 b, where b is the width of the bottom of the canal, would work well. This results in a narrower and deeper than normal canal section. It was assumed that rock excavation of a narrower and deeper section would be less expensive.
- The guides were adapted for the unusual condition of initial construction of canal section for about 15 percent of ultimate capacity (6,248 ft^3/s ultimate versus 1,102 ft^3/s for this Study). It appears that using 1.5 b for the bottom width of the canal would work well with this lesser flow rate and still have capacity to convey storm inlet flows.
- Freeboard was designed for the ultimate flow capacities. This was done to accommodate inflows that may occur due to storm flows.
- Manning's coefficient of roughness "n" will be adjusted for a hydraulic radius "r" greater than 4.0 (see Appendix A, General Design Flowcharts):
    - Concrete lined canal sections with r > 4.0, $n_{adjusted} = \dfrac{0.0463 \ r^{1/6}}{\log\left[14.8 \dfrac{r}{0.005}\right]}$
    - Earthen canal sections with r > 4.0, n adjusted by ratio of lined section and ratio of 0.0225/0.014 = 1.61.

Wasteways along East High canals with nearly all flow being distributed via pumping plants will:

- Have the passive capacity to dump large flows when electric power is lost.
- Have intakes with side-channel weir walls with a top elevation set at 0.2 feet above normal water surface except for Farrier Wasteway, which is 0.1 feet.
- Have side-channel weir walls with sufficient length to pass pumping plant rejection flows, plus cumulative 25-year storm inflow using less than 50 percent of the lining freeboard which is nearly equivalent to 25 percent of the bank freeboard.

Engine generators are provided at each check structure and wasteway site for gate operation during emergency operations during power outages. Power outages may occur during rainstorms when large surface runoff may also enter the canal.

Pumped distribution systems:

- Pipeline hydraulics were analyzed using the computer software Bentley WaterCAD Version 8i /WaterGEMS.
- Typical field size to receive deliveries is 160 acres.

- Pipelines 24” diameter and smaller will be specified as Polyvinyl Chloride (PVC) or High Density Polyethylene (HDPE). Plastic is presumed to be considerably less expensive in this range than metallic pipe, and cathodic protection is not required. Pipelines larger than 24 inches will have more material types (steel, pre-tensioned concrete, and fiber reinforced plastic) included in the specifications paragraphs.

- Minimum pressure at the outlet of the field delivery box is 10 pounds per square inch (psi). It is assumed that farmers will boost pressure to suit their system requirement.

- Each field delivery box will house an isolation valve, flowmeter (magnetic probe), and a Pressure Reducing Valve (PRV) to be hydraulically forced closed upon loss of electrical power. The solenoid-controlled PRV closure (typical 160 acres, 3.2 ft^3/s, 10-inch size valve with a typically 60-second full closure) will prevent dewatering of the pump regulating tank following loss of pumping plant electrical power, which will permit automated pump re-start following reestablishment of electrical power.

Canal inline check and siphon check inlet structures will have:

- A minimum of 0.5-foot gate loss available to maintain minimal automated water level control.

- Canal transitions to radial gate bays sized for theoretical velocity of about 5 feet per second (fps). Gate bay width to be less than 1.5 times the water normal water depth.

- A bypass weir if there is no canal wasteway located near (within approximately 1,000 feet) upstream.

- A bypass weir top elevation set at 0.0 feet above normal depth.

- A bypass weir length sufficient to pass cumulative 25-year storm inflow using less than 50 percent of the lining freeboard which is nearly equivalent to 25 percent of the bank freeboard.

Check inlet and outlet transition convergence/divergence and friction losses are to be calculated in addition to the minimum gate loss.

Canal structures to be “broken-back” style transitions, not streamline-warped nor warped styles as used in 1960s feasibility study. Streamline-warped and warped style transition construction forming costs are excessive and require skills difficult to obtain today.

Siphons are to use closed square to round transitions when flows are greater than 100 ft^3/s.

2.1.3 Studies/Reports/Analyses

The following studies/reports/analyses were performed or earlier studies were consulted. Results of the supporting studies/reports/analyses are presented within this document.

- Geology – Areas with potentially problematic soils have been delineated, such as dispersive soils, low-density soils, and expansive soils. These types of soils have a significant effect on selection of the type of lining and the foundation treatment of the canal. Soil resistivity tests for pipelines may be required in accordance with Technical Memorandum No. 8140-CC-2004-1, Corrosion Considerations for Buried Metallic Water Pipe (Reclamation, 2004).

- River morphology and river water surface elevations – Scour and degradation estimates were not completed. Scour and degradation studies will be required during final design where East High Canal siphons cross river channels and flood drainage channels. For this Study, 10 feet of scour/degradation was assumed when designing depths of cut-and-fill for siphons along the East High Canal.

- Canal operation study – A preliminary operation study was performed to determine the general regulating and protective structure requirements. The study determined the required lining and canal bank heights.

- Hydrologic studies – Design of cross drainage/runoff is based on data, computations, and graphs developed for the 1960s feasibility study.

- Physical model studies – Physical model studies are normally not required for the canal or canal structures. An exception may be at the East High Canal headworks/diversion site where a new structure is to be constructed that ties into the existing Main Canal.

- Physical hydraulic model studies – The design team recommends that physical hydraulic model studies be completed for the following structures prior to final design:

    - Rocky Coulee Reservoir inlet chute

    - Rocky Coulee Pumping Plant discharge pipeline pressure sustaining valve structure.

2.1.4 East High Canal

Size and design most EHC canal sections for about 15 percent ultimate capacity for full-development flow rate.

- Canal sections in thorough fill, typically cross-drainage culvert locations, are to be constructed to ultimate capacity size.

- Canal sections at bridge locations are to be constructed to about 15 percent ultimate capacity.

Flow measurement will be provided through ultrasonic flow meters mounted on the canal sides. Ultrasonic flow measurement will be located on both the East High Canal near the beginning of the canal at the headworks structure (Sta. 1+30) and on East High Canal after the Black Rock Reregulating Reservoir (Sta. 1333+00). Flow measurement for the Black Rock Branch Canal will be provided by flow measurement incorporated into the Black Rock Coulee Pumping Plant No. 1.

EHC cross-drainage typical design is for 25-year storm. Check to prevent embankment overtopping for 100-year storm.

- The same storm canal inflow will occur regardless of whether full development or 15-percent ultimate capacity. The 1963 Flow prediction was used.

- Criteria for determining the need for cross-drainage structures must include a method for estimating the peak discharge resulting from thunderstorms and combined snowmelt and rain. These structures would be constructed to underpass or divert into the canal the runoff water from numerous tributary draws crossed by the East High Canal.

    - It is assumed that runoff resulting from combined snowmelt and rain will occur in the nonirrigation season. During such times, the entire canal capacity would be available to convey runoff water accumulated between wasteways.

    - Runoff in late spring resulting from thunderstorms, however, will occur during the irrigation season and may coincide with the seasonal peak of water delivery. Under these conditions, the extent of the tributary area subject to the same thunderstorm must be taken into account where the area consists of several tributary draws or subareas.

    - The total runoff inflow into the canal shall not encroach on the freeboard more than 1/4 of the total freeboard (1/2 lining freeboard). Should the maximum flood inflow intercepted between wasteways when added to irrigation water expected in the canal exceed these conditions, one or more of the following changes will be made: (1) some of the drainage inlets will be changed to culverts or overchutes to reduce the amount being intercepted, or (2) additional wasteways will be provided, or (3) canal capacity will be increased.

    - Consideration will also be given to the possible damage and risk compared to the additional cost of providing cross-drainage structures where determining the capacity of the structure.

- The storm flow during emergency operation impacts the ultimate flow siphon sizing. Siphon pipe capacity to pass emergency 25-year storm flow if there is not a wasteway located upstream.
    - Inline check and siphon check inlet transitions **are not to have** bypass weir walls when located immediately downstream of a wasteway. This enables the ability to check the water depth sufficiently to force the excess water over the side-channel wasteway weir walls.

Provide EHC lining at locations similar to that described in the 1963 Feasibility Report. Most was concrete lining, but there was some earth lining (most of Black Rock Branch Canal).

2.1.5 East Low Canal

ELC embankment water-level sensors currently exist at Miles 10, 20, 23, 37, 54.9, 66, 71.5, 84.7, and 87 (Montgomery, 1995). ECBID is considering adding sensors at about Miles 30, 46, and 60.

ELC wasteway structures are located at Mile 23 (Rocky Coulee), 37 (Weber), 54.9 (Lind Coulee), and 87 (Scooteney).

Water-level sensors communicate by radio transmitter with the CBP Supervisory Control and Data Acquisition (SCADA) system.

ECBID has been monitoring and collecting information on existing seepage rates and obtaining real-time piezometric heads in the canal embankment at critical sites since 1995.

2.2. Canals

Canals are open channels that transport water from a source to distribution and delivery locations. Canals may be either lined or unlined depending on earth and rock materials encountered and other requirements established for the feasibility study. For canals to operate efficiently, several types of structures are required and discussed below:

Canal structures generally fall into the following groupings:

- <u>Conveyance structures</u> – Canals that have inline structures such as inverted siphons, road crossings, tunnels, and drop or chute structures, are utilized within the system to convey the water throughout the lands.

- <u>Regulating structures</u> – Canal headworks, turnouts, farm deliveries, check structures, pumping plants, and division structures will also be constructed within the system to perform the typical task of regulating water passing through and being delivered to lands.

- Flow measurement structures/devices – There will be flow measurement features such as ultrasonic flow meters to facilitate efficient canal operations.

- Divert drainage systems and structures (parallel drains) – These structures convey storm or drainage water away from the canal by either passing under the canal, over the canal, or into the canal. Drain inlets will be utilized to bring individual small storm runoff flows into the canal when collection and passing under/over the canal is impractical.

- Protective structures – Wasteways (operational and evacuation). An operational (passive) wasteway automatically removes excess water from a canal such as flood inflow, unused irrigation flow, or the total canal flow if a pumping plant shuts down. An evacuation (active) wasteway allows for drainage of a canal for maintenance. The two types of wasteways may be combined into a single structure. Automated gates may also serve both purposes and have been used on the East Low Canal.

- Cross drainage – Structures such as culverts and overchutes will convey storm or drainage water under and over (respectively) canals. Cross-drainage features were designed based on the 25-year frequency storm event and checked to prevent embankment overtopping for 100-year frequency storm event. The same storm canal inflow will occur regardless of whether designs developed in this Study are implemented or full project development is implemented.

- Security/Safety features – All canal structures are designed with security/ safety features to protect people and animals.

- Animal escape ramps – Escape ramps are provided in concrete-lined sections of the proposed East High and Black Rock Branch canals at the request of the Washington State Department of Fish and Wildlife (WDFW). These ramps will be constructed perpendicular to the centerline of the canal and have visual/audible barriers strung across the canal to direct animals to the ramps. These ramps will be located along the canals according to the following criteria:

    - Ramps are to be provided immediately upstream (500 feet) of each check structure, siphon inlet, tunnel inlet, and wasteway inlet. Fencing will be installed on both sides of the canal from the visual/audible barrier to the structure to prevent animals from entering the canal between the escape ramp and the structure.

    - Additional ramps will be spaced along concrete-lined canal sections of the canal approximately 1 mile on centers. The ramps will alternate between both sides of the canal.

    - Ramps will also be used for access to the bottom of the canal by operation and maintenance (O&M) personnel.

- Wildlife bridges – At various locations along the proposed East High Canal, WDFW personnel selected sites for the installation of wildlife bridges to facilitate the movement of animals across the proposed canal. The bridges will be covered with soil and native plants and boulders placed on the bridge to provide cover. These bridges will also serve as O&M canal crossings (public access will be prevented). Fencing will be installed on both sides of the canal both upstream and downstream of these bridges to direct animals to the bridge. The fencing will extend 1,000 feet in both directions.

- Gates or cattle guards - Will be used when fencing crosses public or O&M access roads.

- Safety cables with floats – Will be located upstream from canal structures such as siphon inlets, tunnel inlets, wasteways and check structures.

- Safety ladders – Will be located every 750 feet on alternating sides of the canal in all concrete-lined canal sections. Safety ladders are also to be located on each end of safety cable locations.

2.2.1 Canal Alignment and Profile

In general, the alignment of the proposed East High Canal follows the alignment developed for the 1963 feasibility study, with minor modifications due to current design criteria. The 1963 alignment was converted from the 1 inch = 400 feet TRS (Township, Range, Section) maps to an electronic format for use in the computer software AutoCAD/Civil3D. The 1963 converted alignment (paper to electronic file) was created by “tracing” or digitizing the 1963 alignment. Early in the design it was discovered that the converted alignment, because of the way it was created, could not be used successfully to model the alignment with computer software. For this reason, the design team had to regenerate the alignment from the 1963 maps.

Canal alignments have been modified in select locations from the 1963 proposed alignments when earthwork balancing revealed a need. This shifted alignment is not depicted on drawings, but is available in the electronic AutoCAD/Civil3D files.

The design criteria used in selecting the final canal alignment considered economics, excavation and fill requirements, availability of rights-of-way and easements, and environmental concerns.

Major excavations or fill areas were balanced against the expense of utilizing alternative methods of traversing the area such as tunnels or inverted siphons.

Canal profile graphs are shown at a scale at which the water surface profile and canal invert can easily be viewed. The horizontal and vertical scales are different to allow easy viewing. The water surface profile shows head losses at all inline structures. Water surface/hydraulic grade elevation requirements are shown at the source of flow, at turnouts, and at the downstream end of the canal.

2.2.2 Canal Design

The Manning's friction analysis equation was used for hydraulic analysis and design of the canal. The associated coefficients are shown in the hydraulic properties tables on the drawings.

Excavation and embankment slopes, normal water depth, lining and bank freeboard, and hydraulic properties tables are shown on drawings 222-D-50202 through 222-D-50205.

2.2.3 Canal Lining Requirements

For this feasibility study, the canal lining used for the canal is a 4-inch thick unreinforced concrete lining. Concrete lining significantly reduces loss of water through seepage. There are two common threats to the integrity of the canal embankments: burrowing animals and loss of lining integrity due primarily to cracking or spalling. Concrete lining is recommended for canal sections that have pipe crossings, 100-percent fill embankments, sections with leakage concerns, and areas that have specific foundation issues (Figure 2- 1).

Concrete canal lining requires joints for control of expansion and contraction cracking of the lining. Longitudinal polyvinyl chloride (PVC) waterstop is to be placed on 15-foot centers, located symmetrically around the canal centerline alignment, both on the canal invert and along the canal-sides. Transverse PVC waterstop is also placed on 15-foot centers. Transverse contraction joints are on a radial line spaced along the centerline of the canal. Transverse contraction joints are at the same station across the entire canal cross-section. One-inch-thick sponge rubber expansion joints are located in place of the contraction joint nearest to every canal curve Point of Curvature (PC) and Point of Tangency (PT), and every 500 feet.

Earth lining is recommended for most of the Black Rock Branch Canal reach (Figure 2- 2). Earth lining is subject to erosion, animal burrows, and vegetation growth, and has a higher rate of seepage than concrete or membrane linings. However, the cost-effectiveness for construction when used in the correct locations makes this the preferred canal lining recommendation in most instances.

Membrane lining is not recommended for this feasibility study. The ECBID manager has stated that some concrete linings have required significant maintenance and requested the investigation of membrane linings. For this feasibility study, only concrete linings have been used for estimating purposes. Examination of the appropriate use of membrane linings should be accomplished during the final design phase.

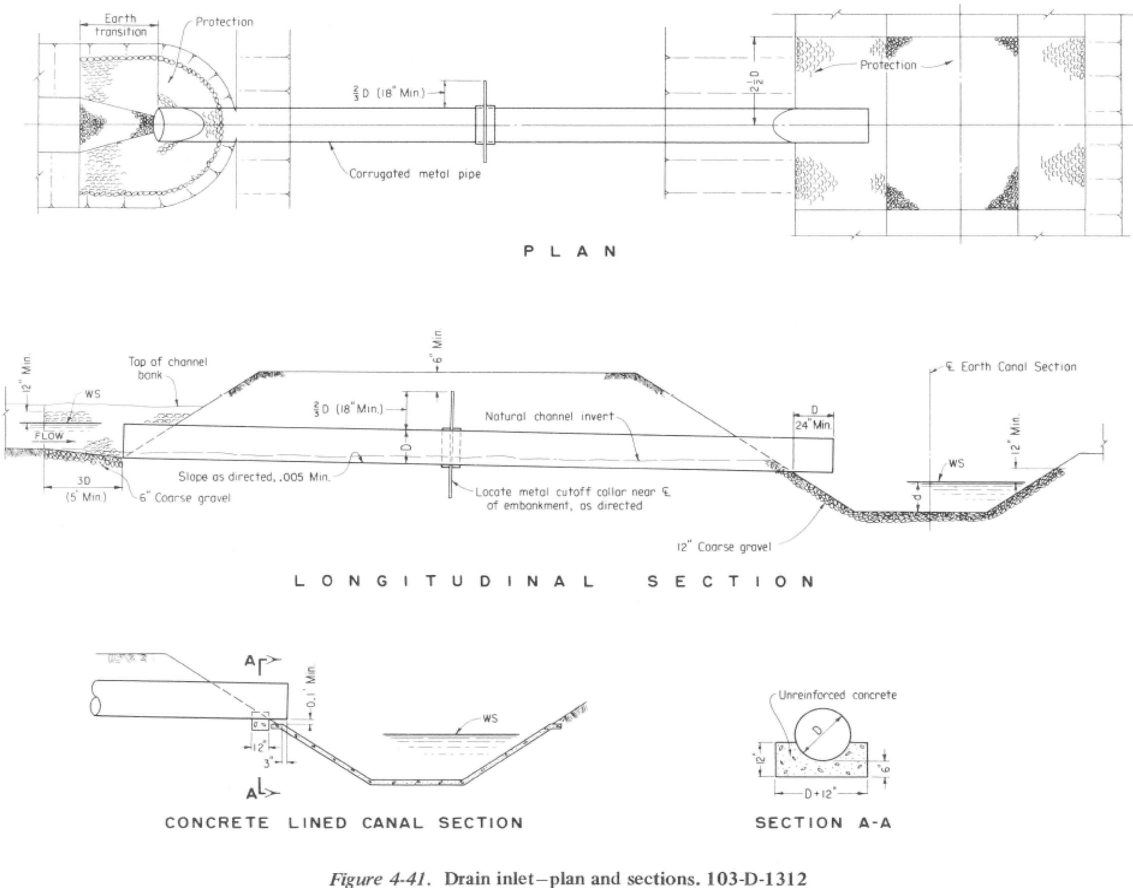

Figure 4-41. Drain inlet–plan and sections. 103-D-1312

Figure 2- 1. Drain inlet – plan and sections, 103-D-1312

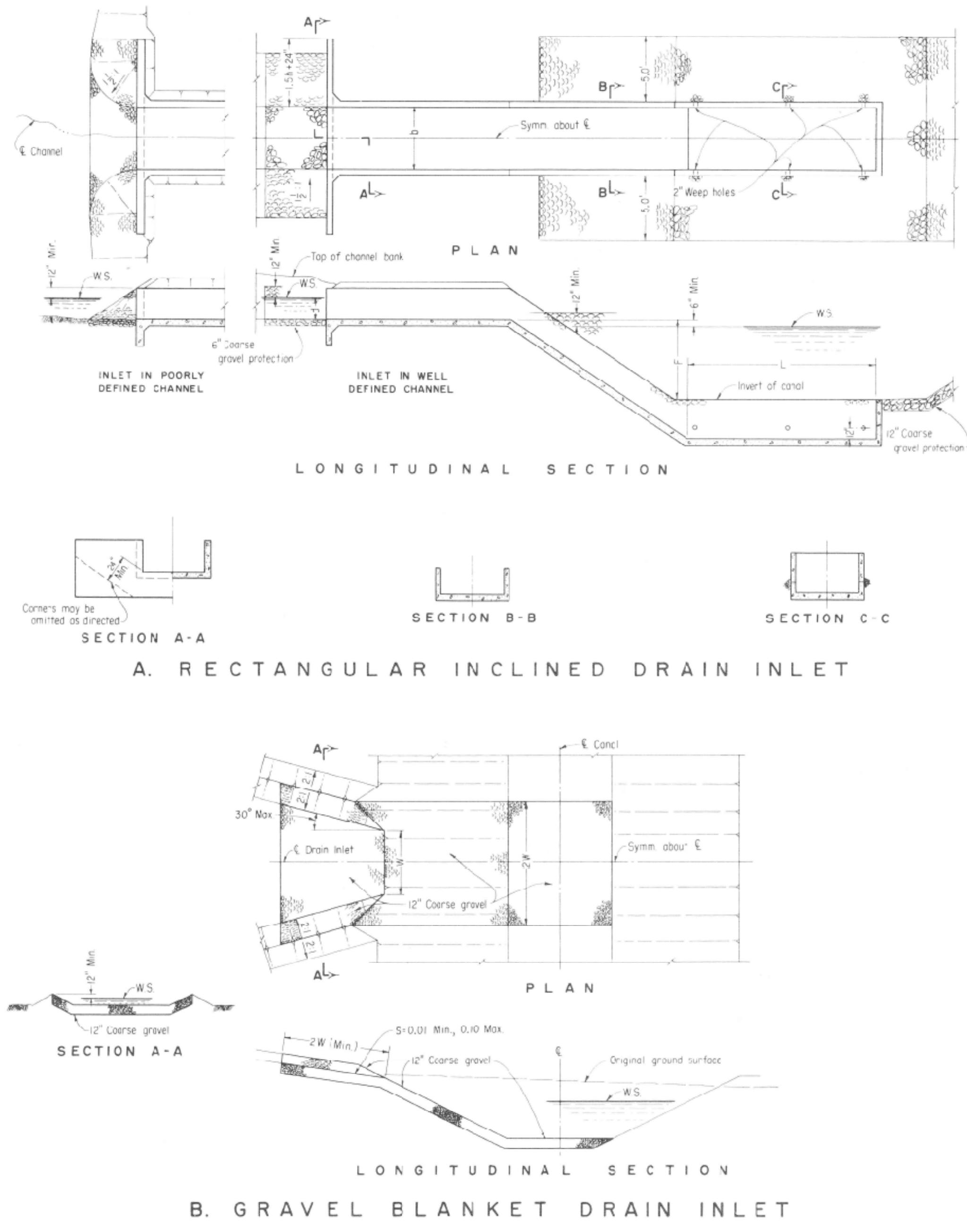

Figure 4-42. Drain inlets–plan and sections. 103-D-1313

Figure 2- 2. Drain inlet – plan and sections, 103-D-1313

2.2.4 Debris and Sediment

The existing canal systems experience little debris or sediment issues. There are no special requirements for preventing debris and sediment from entering the canal, and there are no provisions for cleaning the canal.

2.2.5 Headworks

The proposed East High Canal headworks structure is located along the existing Main Canal as envisioned in the 1960s feasibility study. This location is at a relatively high elevation of the project which minimizes re-lift of water to irrigate much of the area. The headworks structure will utilize the Main Canal bank height as the top of structure elevation.

The headworks for the East High Canal were designed for the required ultimate flow of 6,248 ft^3/s. With the present feasibility study demands of 1,102 ft^3/s, two small and two large gate bays were used. The two small gates were installed and the two large bays were bulkheaded for future use. Radial gates were selected to best regulate the diversion of flows from the Main Canal into the East High Canal. Very little foreign material is present in the existing Main Canal and debris exclusion features, sediment exclusion measures, ice concerns, and fish exclusion requirements were not justified. Flow measurement is required for safe and efficient operations of this major diversion structure.

2.2.6 Flow Control

Canal inline-check and siphon-check inlet structures are to have a minimum of 0.5-foot gate loss available to maintain minimal automated water level control. Canal transitions to radial gate bays are sized for a theoretical velocity of about 5 feet per second (fps). Gate bay widths are to be less than 1.5 times the canal normal water depth. Bypass weirs are to be provided if there is no canal wasteway located immediately upstream. Bypass weir top elevations are set at 0.0 feet above normal water depth. Bypass weir length shall be sufficient to pass the cumulative 25-year storm inflow using less than 50 percent of the lining freeboard, which is nearly equivalent to 25 percent of the bank freeboard. Check inlet and outlet transition convergence/divergence and friction losses are to be calculated in addition to the minimum gate loss. Canal transition structures are to be broken back style transitions, not streamline warped nor warped styles as used in the 1960s feasibility study. Streamline warped and warped style transition construction forming costs are excessive and require skills difficult to locate today. Siphons are to use closed square to round transitions with flows greater than 100 ft^3/s.

Engine/generators are to be located at each check and wasteway site for continuity of gate control during emergency operations at times of power outages. Power outages may occur during rain storms when large surface runoff may also enter the canal.

2.2.7 Crossings

Crossings of roads, railroads, canals, and rivers were quantified for estimating purposes. Without utility location design data it was impossible to account for pipeline and utility crossings. These were considered to be accounted for in the design contingencies. Final design phase will require a thorough utility easement search.

2.2.8 Existing Main Canal Operations

The existing Main Canal Headworks is constructed through the existing Dry Falls Dam near its left abutment. This structure has the capacity to release flows into the Main Canal in excess of the required 19,100 ft^3/s needed to feed the ultimate development of the Columbia Basin Project. Based on the headworks flow formula,[7] a simplified table showing gate opening versus discharge flow was developed (Table 2- 7). This assumes all 6 gates in operation and Banks Lake water surface between elevation 1,550 and 1,572.

Table 2- 7. Gate Opening versus Discharge Flow Capacity

Reservoir Elevation (Feet)	Gate Opening (Feet)									
	0.5	1	1.5	2	2.5	3	3.5	4	4.5	5
1550	1,160	2,311	3,455	4,589	5,715	6,831	7,937	9,035	10,124	11,203
1552	1,195	2,381	3,559	4,728	5,888	7,040	8,182	9,316	10,441	11,557
1554	1,229	2,449	3,661	4,864	6,059	7,245	8,423	9,593	10,753	11,905
1556	1,261	2,515	3,760	4,997	6,225	7,446	8,658	9,862	11,057	12,244
1558	1,293	2,579	3,856	5,126	6,387	7,641	8,886	10,123	11,352	12,573
1560	1,325	2,641	3,950	5,252	6,545	7,831	9,109	10,378	11,640	12,894
1562	1,355	2,703	4,042	5,375	6,699	8,016	9,326	10,627	11,921	13,207
1564	1,385	2,762	4,132	5,495	6,850	8,198	9,538	10,871	12,196	13,513
1566	1,414	2,821	4,221	5,613	6,998	8,375	9,746	11,109	12,464	13,812
1568	1,443	2,878	4,307	5,728	7,142	8,549	9,949	11,341	12,727	14,105
1570	1,471	2,935	4,391	5,841	7,284	8,720	10,148	11,570	12,984	14,391
1572	1,498	2,990	4,474	5,952	7,423	8,887	10,344	11,794	13,237	14,673

Reservoir Elevation (Feet)	Gate Opening (Feet)									
	5.5	6	6.5	7	7.5	8	8.5	9	9.5	10
1550	12,273	13,334	14,385	15,427	16,459	17,482	18,494	19,498	20,491	21,474
1552	12,664	13,762	14,851	15,931	17,001	18,062	19,114	20,156	21,189	22,213
1554	13,049	14,183	15,309	16,426	17,533	18,632	19,722	20,803	21,874	22,937
1556	13,422	14,592	15,753	16,906	18,050	19,185	20,312	21,430	22,539	23,639
1558	13,786	14,990	16,186	17,373	18,552	19,723	20,885	22,039	23,184	24,320
1560	14,140	15,377	16,607	17,828	19,041	20,246	21,443	22,631	23,811	24,983
1562	14,485	15,755	17,018	18,272	19,518	20,757	21,987	23,209	24,423	25,629
1564	14,822	16,124	17,419	18,705	19,984	21,255	22,518	23,773	25,020	26,259
1566	15,152	16,485	17,811	19,128	20,439	21,741	23,036	24,323	25,602	26,874
1568	15,475	16,838	18,194	19,543	20,884	22,217	23,543	24,861	26,172	27,475
1570	15,791	17,184	18,570	19,948	21,319	22,683	24,039	25,388	26,730	28,064
1572	16,101	17,523	18,938	20,346	21,746	23,139	24,526	25,904	27,276	28,640

[7] Email from John O'Callaghan to Paul Ruchti dated May 11, 2008, at 10:32 a.m.

Reservoir Elevation (Feet)	Gate Opening (Feet)							
	11	12	13	14	15	16	17	18
1550	23,411	25,308	27,164	28,978	30,750	32,479	34,165	35,807
1552	24,231	26,210	28,150	30,050	31,910	33,728	35,505	37,240
1554	25,034	27,093	29,115	31,098	33,042	34,947	36,812	38,636
1556	25,812	27,948	30,048	32,111	34,137	36,124	38,073	39,983
1558	26,567	28,778	30,954	33,094	35,198	37,264	39,294	41,286
1560	27,301	29,585	31,834	34,048	36,227	38,371	40,478	42,549
1562	28,016	30,370	32,690	34,976	37,228	39,446	41,629	43,776
1564	28,713	31,135	33,524	35,880	38,203	40,493	42,748	44,970
1566	29,394	31,882	34,338	36,762	39,154	41,513	43,839	46,132
1568	30,059	32,612	35,133	37,624	40,082	42,509	44,904	47,266
1570	30,710	33,326	35,911	38,466	40,990	43,482	45,944	48,373
1572	31,347	34,024	36,672	39,290	41,877	44,434	46,960	49,456

The Main Canal operations at Summer Falls are described[8] as “In terms of the upstream elevation, the plant has a controller that uses the turbines to maintain a constant upstream elevation, which can be set in the controller but typically isn't changed at all. It runs the same all year. The turbines have a maximum flow they are capable of passing and when the canal exceeds this amount, the plant controller then opens the check gates and uses them to maintain the constant upstream elevation. Again, this is automated and the upstream elevation does not vary significantly. In essence, the net results of the power plant and check gates operations are very simple. The plant just takes whatever the flow in the canal is and runs it through the turbines and/or the check gates, maintaining a constant upstream elevation.”

The Main Canal flow capacity of the first portion of the canal was increased to 19,100 ft^3/s about 1960, with the construction of the second barrel of Bacon siphon and tunnel. The Main Canal capacity from the tunnel outlet to Summer Falls is 9,300 ft^3/s. This is sufficient to serve the present Project needs south of Pinto Dam of approximately 8,000 ft^3/s, and the proposed East High Canal needs of approximately 1,000 ft^3/s. The check gate flow capacity was stated in a letter from the Summer Falls Power Plant designers[9] as “Under the design concept, the maximum pool elevation of 1501.7 would occur if the entire flow from the tunnels (19,300 ft^3/s) is diverted through the fully opened check structure gates. This condition assumes simultaneous shutdown of the East High Canal headworks and the Summer Falls Powerhouse.” This calculation required the canal banks to be above elevation 1,505 feet (original Bureau of Reclamation project vertical datum, or 1507.85 NAVD83) and two radial gates (40 feet wide each) be fully open with the bottom of gates above elevation 1498.2 feet (original Bureau of Reclamation project vertical datum).

[8] Email from John O’Callaghan, Ephrata Field Office – Ephrata, Washington, to Paul Ruchti, Technical Service Center, Denver, Colorado, dated October 28, 2008, at 12:55:12 p.m.
[9] Schuchart/Harza dated September 28, 1987. Attached to email from Steve Robertson to Paul Ruchti dated November 28, 2008.

2.2.9 East High Canal

The EHC sections for this feasibility study development (45,545 acres) are to be constructed for about 15 percent of ultimate (full development of 385,500 acres) flow capacity. Tunnels, siphons, in-line checks, and thorough fill canal sections will be constructed for ultimate capacity. Previously completed appraisal designs produced a spreadsheet which accumulated flow for ultimate and appraisal design Alternative B acreage by "Area" and accumulated that demand along the canals to the headworks at the Main Canal. That spreadsheet was updated based on the revised Township/Range/Section (TRS) acreage table for the feasibility designs. Peak demand (47 acres per ft^3/s, operational waste, seepage) used in the appraisal design was verified and used during the feasibility designs.

Generally, bedrock[10] in the East High Canal Area is formed by the Priest Rapids Basalt Member and/or the Roza Basalt Member of the Yakima Basalt Formation. In places, the basalt is thoroughly jointed and the joints are open enough to transmit considerable quantities of water. For the purpose of this report, it has been assumed that the canals with the water prisms in the basalt will require lining. Extensive investigations to establish the lithology of the basalt along the canal will be required, prior to construction, to verify or refute this assumption.

The East High Canal will start at Mile 8 of the Main Canal about 3,000 feet upstream from Summer Falls where the Main Canal waters drop into Billy Clapp Lake (Long Lake Reservoir). In the 25 miles from the bifurcation to Black Rock Coulee Re-Regulating Reservoir, the basalt is at or very close to the surface. In the reaches where the basalt is not exposed, the overburden consists of basalt detritus and basaltic sand mixed with a small amount of silt. It is recommended that this 25 miles of the East High Canal be concrete lined.

Between Black Rock Coulee Re-Regulating Reservoir and Weber Coulee, the East High Canal will extend another 27 miles. In these 27 miles, Mile 25 to Mile 51, the basalt is relatively close to the surface. The overburden varies from 0 to 15 feet and consists of basalt detritus, sand, and silts in ascending order. It is recommended that concrete canal lining be used from Mile 25 to Mile 51, as the water prism of the canal will intersect the contact between the basalt and overburden in a number of places.

The next few pages are listings of the depth to basalt, stationing, and lining recommendations along the East High Canal.

[10] Appendix Vol. VI A (Revised March 1966) of the 1960s feasibility report (Reclamation, 1966b)

East High Canal
Lining Requirements and Rock Information for Excavation Quantities

Station to	Station	Type of Lining	*Approx. Depth to Rock
0+00	51+68	Concrete	0
51+68	59+35	Long Lake Siphon	
59+35	112+00	Concrete	0
112+00	131+00	Long Lake Tunnel	
131+00	250+00	Concrete	0
250+00	269+00	Concrete	15'
269+00	330+00	Concrete	0
330+00	347+00	Stratford Tunnel	
347+00	355+00	Concrete	0
355+00	380+00	Concrete	15'
380+00	417+42	Concrete	0
417+42	475+13	Crab Creek Siphon	
475+13	780+92	Concrete	0
780+92	851+53	Broken Rock Siphon	
851+53	923+00	Concrete	0
923+00	950+00	Concrete	3
950+00	1011+00	Concrete	0
1011+00	1060+00	Concrete	3
1060+00	1155+00	Concrete	15
1155+00	1170+00	Concrete	4
1170+00	1325+00	Concrete	0
Black Rock Reservoir			
1333+00	1480+00	Concrete	0
1480+00	1553+00		3
1553+00	1595+00		6
1600+00	1670+00		10
1670+00	1690+00		4
1690+00	1750+00		9
1750+00	1890+00		14
1890+00	1915+00		6
1915+00	2056+50		14
2083+00	2127+00		7
2188+80	2350+00		12
2350+00	2370+00		18
2370+00	2495+00		12
2495+00	2674+50		7

Figure 2- 3. Recommended lining requirements for the East High Canal. Information extracted from Appendix Vol. VI A (Revised March 1966) of the 1960s feasibility report (Reclamation, 1966b)

The proposed Black Rock Branch Canal will begin at the outlet of the discharge line of the Black Rock Coulee Pumping Plant No. 1, which will pump the water from Black Rock Coulee Re-Regulating Reservoir. This reservoir site is located in Mile 25 of the East High Canal. The depth to basalt was obtained from power auger holes drilled at about 1-mile intervals. Results of the subsurface investigation indicate that the water prism of the first 15 miles of the canal will intersect basalt in very few places. The material overlying the basalt generally consists of sandy silt containing approximately 10-15 percent sand to a depth of 8-15 feet below ground surface. Below this sandy-silt and above the bedrock, the sandy silt continues, but contains more land and is in alternate carbonate cemented (caliche) and uncemented layers. The sandy silts have never been thoroughly wetted and are relatively permeable in natural state. The lower layered zone could provide water pathway at contact between layers. The nonplastic sandy silts are satisfactory material for heavy compacted earth 1ining when the lining is protected with a gravel cover. Such an earth canal lining is recommended for these 15 miles of canal.

In the next 3 miles of this canal (Mile 16 to 19); the auger holes indicate that basalt will generally intersect the proposed water prism of the canal. Also, the sandy silt overburden contains more sand. Here, concrete lining is recommended.

The conditions in the following reaches of this canal—Mile 19 to 23; Mile 25 to 27; and Mile 28 to Farrier Wasteway—are quite similar to those expected to be encountered in the first 15 miles of this canal. Heavy earth canal lining is recommended in these reaches.

In the following reaches of this canal—Mile 23 to 25; and Mile 27 to 28—the canal prism will lay within the sandy silt materials that are not expected to contain cemented layers. It is believed that proper priming and puddling would provide satisfactory leakage control and that no canal lining would be required.

BLACK ROCK BRANCH CANAL-LINING REQUIREMENTS AND
ROCK INFORMATION FOR EXCAVATION QUANTITIES

Station to Station		Type of Lining	*Approx. depth to Rock
36+00	75+00	Earth	2' or more below bottom grade
75+00	126+00	Earth	8'
126+00	175+00	Earth	2' or more below bottom grade
175+00	386+00	Earth	17'
386+00	562+00	Earth	2' or more below bottom grade
562+00	589+00	Earth	17'
589+00	652+00	Earth	2' or more below bottom grade
652+00	687+00	Earth	5'
687+00	699+00	Earth	13'
699+00	805+00	Earth	2' or more below bottom grade
805+00	819+02	Earth	10'
Moody Siphon			
860+63	901+00	Concrete	5'
901+00	906+00	Concrete	13'
906+00	909+00	Concrete	24'
Moody Tunnel			
937+00	945+00	Concrete	9'
945+00	969+00	Concrete	3'
969+00	988+37	Concrete	13'

Station to Station		Type of Lining	*Approx. depth to Rock
995+03	1064+00	Earth	2' or more below bottom grade
1064+00	1107+00	Earth	10'
1107+00	1215+00	Earth	2' or more below bottom grade

*Rock information taken from auger holes spaced about 1 mile apart and from Land Classification Maps.

Station to Station		Type of Lining	* Approx. depth to rock
1215	1300	None	20'
1300	1385	Earth	8'
1385	1405	Earth	Elev. 1603
1405	1455	None	2' or more below bottom grade
1455	1502+00	Earth	14'
1502	1530+20	Earth	6'

Figure 2- 4. Recommended lining requirements for the Black Rock Branch Canal. Information extracted from Appendix Vol. VI A (Revised March 1966) of the 1960s feasibility report (Reclamation, 1966b)

2.2.9.1. East High Canal Earthwork

Excavation and embankment quantities were based on three-dimensional computer model for each respective canal reach. Rock excavation percentage assumptions for each reach vary. The rock percentages for each reach were determined by comparing the location of the proposed canal invert and information on the approximate depth to rock shown in Appendix Vol. VI A (Revised March 1966) of the 1960s feasibility report (Reclamation, 1966b). For most reaches, rock excavation will need to be processed for reuse as embankment and gravel bedding materials. The number of processing plant sites necessary vary for each reach.

In some reaches, the 1960s alignment was shifted in places to decrease hauling requirements and to better balance excavation and embankment quantities. The difference between modified and 1960s alignments is not apparent on figures due to the large scale used. For this reason and because of the many minor updates that are necessary to pumping plant and pipeline locations, designs, and quantities late in the design study, the original alignment is shown on figures in this study even though the modified alignment was used to calculate the canal quantities. Future designs should begin by incorporating the modified alignment into all aspects of the design system.

East High Canal, Main Canal (Sta 0+00) to Black Rock Coulee (Sta 1327+00) - "EH1" – Rock excavation is assumed to be a percentage of the total excavation: 75 percent for the Most Probable Low (MPL) estimate, 85 percent for the Most Probable (MP) estimate, and 90 percent for the Most Probable High (MPH) estimate. Much of the canal embankment in this reach and all of the gravel bedding material will come from processed rock because most of the excavated material in this reach is rock. Also, much of the excess common excavation in the first part of the reach is expected to be wasted alongside the canal, as it will be difficult to haul the excess material available across Crab Creek. Five processing plant sites were assumed along the reach to provide the processed rock for use as embankment and gravel bedding.

The 1960s alignment was shifted approximately 500 feet east to higher ground between Stations 850+00 and 1300+00 to increase local excavation and decrease embankment. The amount of overhaul and rock processing necessary was greatly reduced by modifying the alignment in this way.

East High Canal, Black Rock Coulee (Sta 1333+00) to Rocky Branch Coulee Siphon Inlet (Sta 2055+08) – "EH2 Reach 1" – Rock excavation is assumed to be a percentage of the total excavation: 45 percent for the MPL estimate, 50 percent for the MP estimate, and 55 percent for the MPH estimate. Roughly half of the rock excavation in this reach will be processed so it can be reused as embankment and gravel bedding materials. One processing plant site was assumed along the reach to provide the processed rock for use as embankment and gravel bedding.

East High Canal, Rocky Branch Coulee Siphon Inlet (Sta 2055+08) to Weber Coulee Wasteway (Sta 2670+57) – "EH2 Reach 2" – Rock excavation is assumed to be a percentage of the total excavation: 3 percent for the MPL estimate, 5 percent for the MP estimate, and 7 percent for the MPH estimate. All of the rock excavation in this reach will be processed so it can be reused as embankment and gravel bedding materials. One processing plant site was assumed along the reach to provide the processed rock for use as embankment and gravel bedding.

The 1960s alignment was shifted approximately 200 to 300 feet to higher ground in both EH2 reaches between Stations 1640+00 and 1825+00 and also Stations 2190+00 and 2620+00 to increase local excavation and decrease embankment. This alignment modification was necessary to balance earthwork, as the 1960s alignment did not produce enough excavation to cover the embankment need.

Black Rock Branch Canal, Black Rock Coulee (Sta 0+00) to Weber Coulee Siphon Inlet (Sta 988+26) – "BRB Reach 1" – Rock excavation is assumed to be a percentage of the total excavation: 15 percent for the MP estimate; MPL and MPH estimates were not completed. No rock processing plant sites are necessary since common excavation can be used as embankment.

Black Rock Branch Canal, Weber Coulee Siphon Inlet (Sta 988+26) to Farrier Coulee Wasteway (Sta 1525+70) – "BRB Reach 2" – Rock excavation is assumed to be a percentage of the total excavation: 6 percent for the MP estimate; MPL and MPH estimates were not completed. No rock processing plant sites are necessary since common excavation can be used as embankment.

It was not necessary to modify the 1960s alignment to improve quantities in BRB Reach 1 or Reach 2.

2.2.10 East Low Canal

The ELC enlargement involves excavation of the existing initial section construction that was performed in the 1950s, to the capacity needed for this feasibility study. There are approximately 43 miles of canal to expand. There are several canal siphons which will have the second barrel constructed to increase capacity to the ultimate flow.

The stationing along ELC was originally developed based on ground survey techniques, which was normal for that era. Today, survey distances are determined by Grid survey techniques. The following table, developed by the Ephrata Field Office survey staff, provides a cross-reference to correlate the original canal stationing to the NAD83 stationing currently used.

Table 2- 8. East Low Canal stationing adjustments between profile and plat map centerline development

Record Station Ground or Plat		Drawing Station Grid	Difference
1906+30.5		1906+30.5	0
1949+76.83	EQU	1949+76.83	0
1953+15.56		1953+15.54	0.02
1961+93.52	EQU	1961+93.52	0
1982+35.34		1982+35.33	0.01
1992+37.16	EQU	1992+37.16	0
2010+46.69		2010+47.38	-0.69
2025+94.47		2025+94.15	0.32
2058+27.9		2058+28.93	-1.03
2060+66.09	EQU	2060+66.09	0
2077+49.18	EQU	2077+49.18	0
2082+77.83		2082+78.55	-0.72
2108+66.33	EQU	2108+66.33	0
2126+98	EQU	2126+98	0
2150+41.29		2150+41.74	-0.45
2230+19.65		2230+22.86	-3.21
2317+92.37		2317+93.26	-0.89
2346+32.25		2346+33.15	-0.9
2405+40.83		2405+33.33	7.5
2441+72.35		2441+64.49	7.86
2470+83.19		2470+74.93	8.26

Table 2- 8. East Low Canal stationing adjustments between profile and plat map centerline development

Record Station Ground or Plat		Drawing Station Grid	Difference
2521+14.04		2521+04.95	9.09
2573+69.73		2573+58.11	11.62
2619+13.99		2619+02.71	11.28
2662+64.8		2662+53.73	11.07
2663+25.63	EQU	2663+25.63	0
2810+85.66		2810+85.21	0.45
2899+74.25		2899+76.29	-2.04
2903+04.88	EQU	2903+04.88	0
2975+64.16		2975+62.94	1.22
3009+00	EQU	3009+00	0
3227+52.3		3227+51.62	0.68
3336+.04.3		3336+03.73	0.57
3350+00	EQU	3350+00	0
3469+52.4		3469+50.82	1.58
3470+00	EQU	3470+00	0
3525+56.7		3525+56.97	-0.27
3545+89.5		3545+93.31	-3.81
3595+32.1		3595+36.03	-3.93
3601+00	EQU	3601+00	0
3656+27.9		3656+26.22	1.68
3745+10.7		3745+05.82	4.88
3748+00	EQU	3748+00	0
3773+17.9		3773+18.9	-1
3849+65.8		3849+64.73	1.07
3946+67		3946+64.12	2.88
4069+39.3		4069+33.28	6.02
4073+00	EQU	4073+00	0
4172+19.1		4172+17.74	1.36
4273+43.7		4273+37.9	5.8
4349+95		4349+88.34	6.66
4455+57.2		4455+47.59	9.61
4569+57.4		4569+50.78	6.62

Water Service Contracts – A design for a lateral delivery option that would replace individual plants by incorporating the WSC and WSCG fields into nearby pipe lateral systems was requested. This information was also utilized to estimate the construction cost of enlarging the East Low Canal.

Basic data of plant locations and of installed features and photographs were provided by ECBID.

General field location and owners that each plant serves was provided by ECBID and Dale Lindeman.

Precise fields that are served by each plant were provided by ECBID.

Canal Enlargement – Project Management Team (PMT) direction was for the enlargement to the original ultimate earth canal dimensions. There was a proposal to perform the enlargement to the smaller dimensions which would be required to convey only the additional water rates required by the additional field acreages covered by this feasibility study. This would require excavation of approximately 33 percent of that required for the full ultimate capacity. Based upon the excavation savings, the ELC enlargement will only be to that required to carry the current additional flows.

Excavation quantities were based upon original specification ultimate design section dimensions. The aerial topography was collected when water was in the canal. This required development of a method to estimate the below-water canal section into the computer terrain model. Generally, the design invert elevation was modeled and the side slopes were based on the waterline location as photographed. ECBID personnel informed Reclamation engineers that below-water "shallows" wildlife mitigation areas have been developed by widening upper portions of the canal bank at certain locations. This will result in enlargement quantities being somewhat less than actual requirements.

More accurate total canal cross-sections will need to be collected prior to final design.

Canal lining locations were proposed by ECBID and Reclamation. The total length of the requested lining locations is 11.5 miles, located at 31 sites along the 43 miles of canal.

The lining material assumed for the feasibility design was concrete. The height of lining was calculated based on ultimate canal flow rate. In areas where the original ground was below the originally constructed canal embankment core elevation, Reclamation engineers quantified excavation of a 6-foot width from the top of core to top of concrete lining. This was done based on the concern of differential settlement and cracking of concrete lining if placed on uncompacted embankment located above the embankment core.

ECBID requested the evaluation of membrane linings[11] such as the Huesker Canal[3] 8208 PET geocomposite liner. This product has been installed with 1 foot of cover, with about 6 inches of soil and 6 inches of rock.

Most of the original timber bridges crossing the canal enlargement area have been replaced with concrete bridges. Some of the replaced bridges were not constructed to span the ultimate canal section width. Bridge inspection data was obtained from

[11] Email from Roger Sonnichsen dated 5/21/2008 at 10:37 am, included material sheet and basic drawings.

Reclamation's Pacific Northwest Regional Office. Aerial photographs were examined to verify all bridge locations and to estimate the lengths. It is proposed to transition the canal section into/out of narrow sections where existing bridges are shorter than the full ultimate canal section length required. These hydraulic losses were deemed to be minimal in light of the overall canal section being larger than required to convey the additional flow.

There are five canal siphons located along the expansion length of the canal. Quantities for concrete and reinforcement steel were assumed to be equivalent to those used during original construction. Earthwork was estimated using the terrain model developed from the aerial photography contours.

Canal extension was made based on original ultimate canal section dimensions. The extension is between the present canal end near the Scooteney Wasteway and Kansas Prairie Coulee, approximately 2.5 miles.

2.2.11 **Canal Drainage Systems**

The current estimates for canal drainage structures are based on the feasibility completed in the 1960s. To verify the accuracy, the 1960s drainage inlets and culverts were projected on recent aerial photography. These results showed a correlation between the proposed 1960s canal drainage structures and current drainage channels. Therefore, the quantities of excavation, backfill, compacted backfill, riprap, sand and gravel bedding, concrete, cement, corrugated metal pipe (CMP), and reinforced concrete pipe (RCP) were accepted from the 1960s reports. In addition, the hydrologic study was not updated, but for final design, the hydrologic study should be updated. Therefore, recalculating drainage inlet and culvert crossing locations, sizes, and slopes was not done at this time. For original calculations, see pages 233 and 234 of *Appendix Volume VI B – Plan and Estimates* (Reclamation, 1968a). See Appendix E for Drain Inlets, Culverts and Cross Drainage Locations Table.

Drainage runoff along East High Canal and Black Rock Branch Canal is handled by a combination of drainage inlets and culvert crossings. Final design will need to assess ground modifications of divert drainage channels to connect and match natural drainage areas to these inlets and crossings. Both drainage inlets and cross-drainage structures will follow typical details found in *Design of Small Canal Structures* (Reclamation, 1978). The drainage inlets begin on the uphill side of the O&M road, continue underneath the roadway, and outlets into the canal as shown in Figure 2- 1. Single- or double-barrel corrugated metal pipes (CMP) or cast-in-place concrete chutes are used to convey the runoff flow. The CMP includes a flat concrete headwall at the entrance end and a straight pipe end section.

The other type of drainage inlet is a concrete U-shaped channel and chute with a precast concrete slab/beam used to span the opening at the O&M road. The precast concrete slab/beam is a modification of the original 1960s estimate which used treated timber decking. This is the only update to the 1960s estimates. The chute continues below the base of canal invert as shown in Figure 2- 2.

Culvert crossings begin on the uphill side of the O&M road, continue underneath the canal, and outlets downhill of the O&M road on the opposite side. The concrete pipe should have straight slope and need not be a broken-back type culvert. These cross-drainage structures include a cast-in-place flared inlet section, reinforced concrete pipe, flared outlet section, and riprap scour protection (see below in Figure 2- 5).

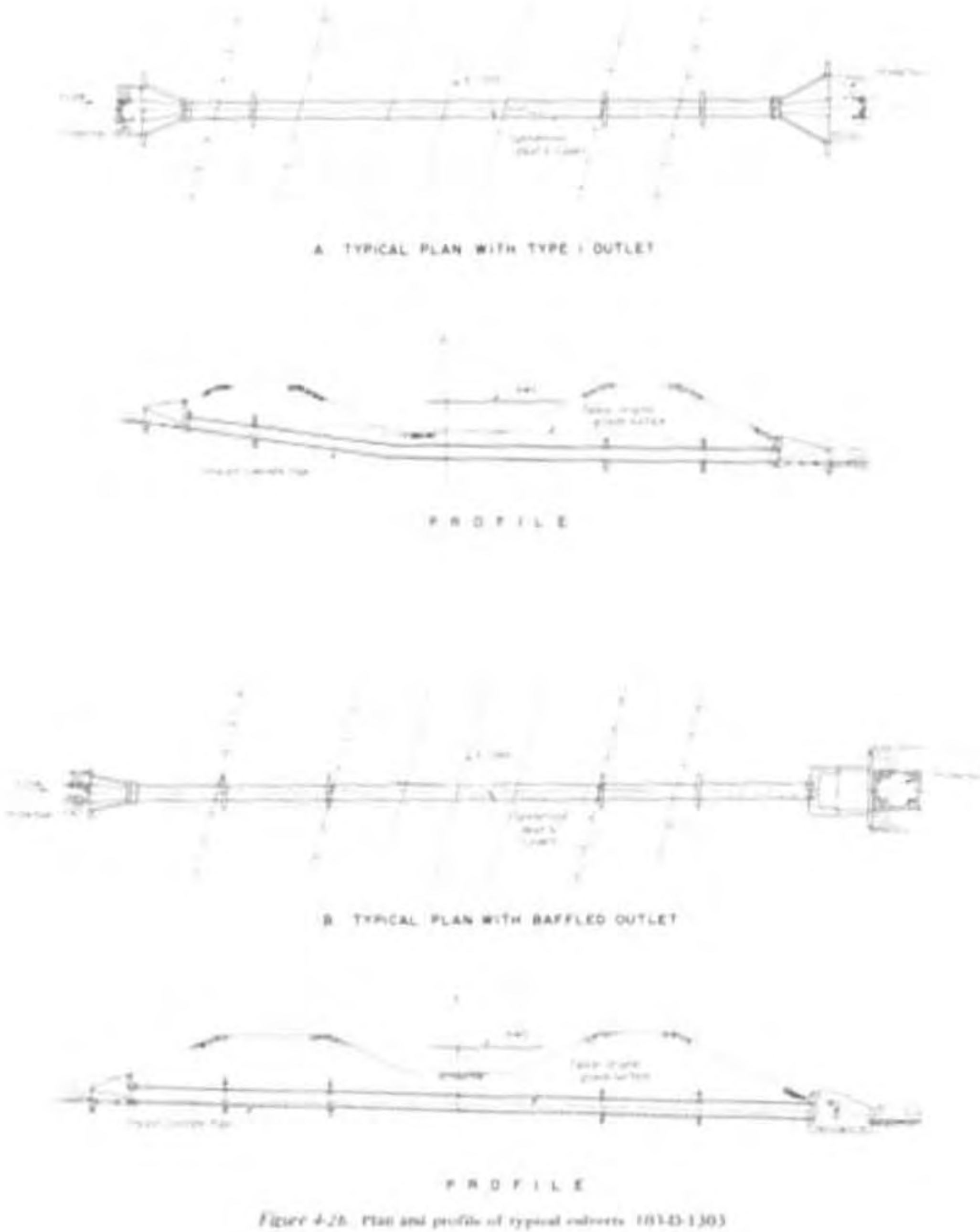

Figure 2- 5. Plan and profile of typical culverts, 103-D-1303

2.2.12 **Pipelines**

2.2.12.1. General

Pipelines transport water from a source to a delivery location(s). Pipelines can be categorized according to use:

2.2.12.1.1 ***Irrigation pipe distribution systems***

Pipe distribution systems are used to convey water to delivery points. Distribution systems can have a few feet of head (limited pressure, less than 35 psi) or be fully pressurized (78 feet (35 psi) or more). The limited-pressure pipelines are normally only used when it is not practical to fully pressurize the system. A pipe distribution system has operational, maintenance, and delivery advantages over an open ditch delivery system. The advantages include less weed control, less reshaping of canals, improved delivery water surface elevations, improved water measurement opportunities (flow meters), improved field access for landowners (less land required or ability to farm over pipe easement).

Full pressure systems – These systems allow the farmer to put in either (1) an open ditch type irrigation system, or (2) a gated pipe system (furrow), or (3) a sprinkler system.

Limited pressure pipe distribution systems – These systems only allow for open ditch irrigation (furrow). If a pressure irrigation system is desired, the farmer will need to install a pump (downstream of the farm delivery) and pipe system.

2.2.12.1.2 ***Transmission pipelines (irrigation, municipal and industrial)***

Transmission pipelines are pressurized pipelines with flow controlled by valves at the downstream end of the pipeline. Associated facilities may include water treatment plants, and storage tanks or reservoirs. There are no transmission pipelines associated with the East High Canal system.

2.2.12.1.3 ***Pumping plant discharge lines***

The discharge line is normally a pipeline or several pipelines with surge protection equipment as required and equipment to prevent return flow when the pumps start up or shut down. The discharge line is normally sized on an economic basis with considerations of power cost and capital cost of the pipe. Several of the irrigation pipe distribution systems on the project are served via pumping plants.

Associated structures and equipment for pipelines include:

- Control equipment
    - Irrigation pipelines will have flow control valves, sectionalizing valves, and turnouts/farm deliveries.

    - Pump discharge lines may include pumps, flow control valves or flap gates, surge control devices and siphon outlets.
- Flow measurement devices such as current meters, acoustic meters, and venturi meters are used as appropriate.
- Regulating tanks are used for pump control and limited storage volume.
- Protective structures of various types are used throughout the systems:
    - Pressure pipelines - air valves and blowoffs.
    - Pump discharge lines - air valves, elevated tanks, air chambers, check valves, check gates, automated valves
    - Intake screening devices.

2.2.12.2. Detailed description of proposed facilities

2.2.12.2.1 ***Pump discharge lines***

Pump discharge lines, pipelines between pumping plants and associated water tanks were sized using the computer software Bentley WaterCAD Version 8i with optimizing and pressure constraints discussed in the 0 Section. Sizing of the discharge lines between pumping plants and each associated elevated water tank were sized to keep friction losses to a minimum. This was done to facilitate pump choices and operations due to the large variation of flows required from early/late irrigation season to the mid-summer peaks in pumped lateral flow demands.

2.2.12.2.2 ***Pipe distribution systems***

Type of irrigation systems will be either gravity or sprinkler laterals.

Pipeline hydraulics – Hydraulics were analyzed using the computer software Bentley WaterCAD Version 8i. Delivery is to each field, typically 130 acre. Pipelines with diameters of 24-inches and smaller will be specified as PVC or HDPE. Hydraulic design calculations utilized actual inside diameters of PVC pipe when laterals required pipe smaller than 24-inches in diameter (PVC). There is a limited amount of steel pipe less than 24-inches in diameter to facilitate a few locations of high pressure and small diameters. Plastic is presumed to be considerably less expensive than metallic pipe in this range and cathodic protection is not required on these types of pipe. Pipelines larger than 24-inch diameters may have more material types (steel, pre-tensioned concrete, and fiber reinforced plastic) included in the specifications. Pressure provided at field delivery boxes is to be greater than 10 psi and farmers will boost pressure as necessary to suit their specific irrigation system requirements.

Anticipated irrigation schedule – The project is designed for a rotation schedule service of irrigation deliveries. Water will be ordered 24 hours in advance.

Farm delivery locations – See Appendix B for data tables.

Location and preliminary design of division boxes – Each field box will house an isolation valve, flowmeter (magnetic probe or similar) and a globe style valve to be hydraulically forced closed upon loss of electrical power. The solenoid controlled globe valve closure (typical 160 acres, 3.2 ft^3/s, 10" size valve with approximately a 30 to 60 second full closure) will prevent dewatering of the pump regulating tank following loss of pumping plant electrical power which will allow automated pump re-start following re-establishment of electrical power. See Table 2- 9 for assumed valve sizing for estimating purposes.

Table 2- 9. Valve sizing for estimating purposes

Maximum Flow (ft^3/s)	Valve Sizing (in.)
1	4
2.8	6
5	8
10	12

2.2.12.2.3 *Pipeline alignment and profile*

The pipe lateral distribution system layout delivers to each section of land that has groundwater-irrigated fields. Farmers will be responsible for the on-farm lateral extension piping, pressure reducing valves, and booster pumps as required, and equipment that will connect boxes to the farmer's center pivots. Laterals have been laid out to utilize corners of sections that are not currently farmed due to center pivot deliveries currently in use. Laterals have been placed to eliminate an excessive quantity of horizontal bends and yet have minimal impact to farm fields during construction. Lateral alignments were through field groupings along section or half-section lines. Occasionally, alignments are diagonal when fields are so arranged. Deliveries have been located to serve the maximum number of fields from one point so as to minimize construction of boxes along each pipe lateral. Pipe alignments will require more precise location for final design when specific site right of way and easement data can be provided.

Pipeline lateral plan views are shown on drawings 222-D-50180 to D-50199. See Table 2- 11, Table 2- 12, and Table 2- 13 for hydraulic data for laterals at pumping plant locations.

Hydraulic transient design was not completed. Based on flow rate, standard ranges were applied to determine elevated tank diameters, number of structural columns for support, elevated tank capacity, spherical air chamber diameter, and air chamber volume (Table 2- 10). See Table 2- 14, Table 2- 15, and Table 2- 16 for elevated tank data for laterals at pumping plant locations, and Table 2- 17, Table 2- 18, and Table 2- 19 for air chamber data for laterals at pumping plant locations.

Table 2- 10. Elevated tank and air chamber design assumptions

Flow Rate (ft^3/s)	Elevated Tank Diameter (ft)	Number of Elevated Tank Columns (ea)	Elevated Tank Capacity (gal)	Air Chamber Diameter (ft)	Air Chamber Volume (ft)
0-49	30	4	100,000	8	268
50-99	36	4	200,000	10	524
100-199	43	5	300,000	20	4,189
200-299	50	6	500,000	30	14,137
300-399	60	8	750,000	40	33,510

Table 2- 11. East Low Canal - pipe lateral hydraulic data

Lateral Turnout or Pumping Plant	Upstream Pressure Head or Canal Water Surface Elevation (ft)		Pump Head – Optimized (Between Pumping Plant and Regulating Tank) (ft)			Pump Flow Rate (ft^3/s)	Pumping Plant Discharge Line Initial Diameter (inches)	Tank Static Water Height from Local Ground (ft)
	minimum	maximum	Static	Friction	Total	Maximum		
EL47	1250.63	1253.27	230.5	1.3	231.8	113.7	60	183.0
EL47R (Relift)	16.2	-	197.6	15.7	213.3	45.7	36	155.0
EL53	1250.63	1251.24	304.4	10.0	314.4	168.9	66	183.3
EL53R (Relift)	137.93	-	112.1	8.4	120.4	76.6	48	52.4
EL68	1226.86	1229.48	338.2	5.1	343.2	345.1	126	142.3
EL68R (Relift)	79.54	-	165.4	56.7	222.1	174.6	66	67.0
EL75	1226.86	1228.72	209.4	86.1	295.4	49.5	30	52.6
EL80	1222.03	1224.21	186.0	48.5	234.5	126.2	48	75.0
EL80R (Relift)	69.3	-	90.0	53.9	143.9	51.0	30	190.0
EL85	1218.46	1219.37	212.5	31.6	244.1	53.7	36	168.5
EL89GR2 (Relift 2)	24.99	-	101.9	20.1	122.0	2.6	7.5	89.4

Table 2- 12. East High Canal - pipe lateral hydraulic data

Lateral Turnout or Pumping Plant	Upstream Pressure Head or Canal Water Surface Elevation (ft)		Pump Head – Optimized (Between Pumping Plant and Regulating Tank) (ft)			Pump Flow Rate (ft^3/s)	Pumping Plant Discharge Line Initial Diameter (inches)	Tank Static Water Height from Local Ground (ft)
	minimum	maximum	Static	Friction	Total	Maximum		
EH4	1493.55	1494.48	291.2	29.1	320.3	15.4	22.6	199.5
EH11	1478.04	1481.41	212.0	8.7	220.6	47.0	36	110.9
EH15G	1478.04	1478.04	GRAVITY LINE; NO PUMP NEEDED			1.3	5.6	no tank
EH19	1464.91	1471.13	182.6	56.8	239.4	60.4	42	22.3
EH19R (Relift)	1641.20	1647.50	178.0	56.3	234.2	52.0	36	114.7
EH29	1455.23	1456.7	149.8	27.0	176.8	47.4	36	140.9
EH33	1448.02	1450.56	289.0	39.8	328.8	88.2	48	179.4
EH35	1448.02	1449.4	129.9	0.1	130.0	113.2	54	130.9
EH42	1429.59	1433.23	166.4	3.0	169.4	108.4	54	141.0
EH47	1425.8	1429.00	64.8	0.3	65.1	21.9	22.6	44.3
EH50R (Relift)	1425.8	1426.43	97.1	0.2	97.3	7.5	13.2	136.6

Table 2- 13. Black Rock Branch Canal - pipe lateral hydraulic data

Lateral Turnout or Pumping Plant	Upstream Pressure Head or Canal Water Surface Elevation (ft)		Pump Head – Optimized (Between Pumping Plant and Regulating Tank) (ft)			Pump Flow Rate (ft^3/s)	Pumping Plant Discharge Line Initial Diameter (inches)	Tank Static Water Height from Local Ground (ft)
	Minimum	Maximum	Static	Friction	Total	Maximum		
BRB2	1646.44	1647.35	93.7	0.6	94.3	27.1	22.6	90
BRB7	1642.18	1644.53	177.8	12.0	189.8	92.9	48	124
BRB7R (Relift)	13.39	-	145.3	28.6	173.9	69.7	36	133
BRB11	1636.38	1640.77	148.6	0.0	148.7	76.3	54	141
BRB17	1627.69	1628.19	187.3	0.0	187.3	1.5	9.4	175
BRB18	1622.04	1623.86	158.0	0.6	158.6	3.8	9.4	132
BRB27	1612.48	1613.23	62.5	0.0	62.6	75.1	54	57
BRB28		1611.69	138.3	0.1	138.4	12.2	22.6	133
BRB29G		1610.89				39.5	36	N/A
Farmer booster pump Del. Sta. 6+07			50 psi estimate at field center			2.5 ft^3/s		

Table 2- 14. East Low Canal - tank hydraulic data and sizes

Lateral Turnout or Pumping Plant	Tank Static Water Height from Local Ground (ft)	Tank Diameter (ft)	Columns (ea)	Head Range (ft)	Maximum Water Surface Height (ft)	Minimum Water Surface Height (ft)	Tank Capacity (gal)
EL47	183	43	5	30	198	168	300,000
EL47R (Relift)	155	30	4	22	166	144	100,000
EL53	183	43	5	30	198	168	300,000
EL53R (Relift)	52	36	4	28	67	38	200,000
EL65	44	30	4	22	55	33	100,000
EL68	142	60	8	39	162	123	750,000
EL68R (Relift)	67	43	5	30	82	52	300,000
EL75	53	30	4	22	64	42	100,000
EL80	75	43	5	30	90	60	300,000

Table 2- 14. East Low Canal - tank hydraulic data and sizes

Lateral Turnout or Pumping Plant	Tank Static Water Height from Local Ground (ft)	Tank Diameter (ft)	Columns (ea)	Head Range (ft)	Maximum Water Surface Height (ft)	Minimum Water Surface Height (ft)	Tank Capacity (gal)
EL80R (Relift)	190	36	4	28	204	176	200,000
EL85	169	36	4	29	183	154	200,000
EL89GR1 (Relift 1)	101	30	4	22	112	90	100,000
EL89GR2 (Relift 2)	89	30	4	22	100	78	100,000

Table 2- 15. East High Canal - tank hydraulic data and sizes

Lateral Turnout or Pumping Plant	Tank Static Water Height from Local Ground (ft)	Tank Diameter (ft)	Columns (ea)	Head Range (ft)	Maximum Water Surface Height (ft)	Minimum Water Surface Height (ft)	Tank Capacity (gal)
EH4	200	30	4	22	210	188	100,000
EH11	111	30	4	22	122	100	100,000
EH15G	Gravity Lateral, no tank required						
EH19	22	36	4	28	36	8	200,000
EH19R (Relift)	115	36	4	28	129	101	200,000
EH29	141	30	4	22	152	130	100,000
EH33	179	36	4	28	193	165	200,000
EH35	131	43	5	30	146	116	300,000
EH42	141	43	5	30	156	126	300,000
EH47	44	30	4	22	55	33	100,000
EH50R (Relift)	137	30	4	22	148	126	100,000

Table 2- 16. Black Rock Branch Canal - tank hydraulic data and sizes

Lateral Turnout or Pumping Plant	Tank Static Water Height from Local Ground (ft)	Tank Diameter (ft)	Columns (ea)	Head Range (ft)	Maximum Water Surface Height (ft)	Minimum Water Surface Height (ft)	Tank Capacity (gal)
BRB2	90	30	4	22	101	79	100,000
BRB7	124	36	4	28	138	110	200,000
BRB7R (Relift)	133	36	4	28	147	119	200,000
BRB11	141	36	4	28	155	127	200,000
BRB17	175	30	4	22	186	164	100,000
BRB18	132	30	4	22	143	121	100,000
BRB27	57	36	4	28	71	43	200,000
BRB28	133	30	4	22	144	122	100,000
BRB29G	Gravity Lateral, no tank required						

Table 2- 17. East Low Canal - air chamber data and sizes

Lateral Turnout or Pumping Plant	Air Chamber Diameter (sphere) (ft)	Volume of Air Chamber (ft^3)	Maximum Pump Head (ft)	Transient Head Estimate (ft)
EL47	20	4,189	232	313
EL47R (Relift)	8	268	213	287
EL53	20	4,189	314	424
EL53R (Relift)	10	524	120	163
EL68	40	33,510	343	463
EL68R (Relift)	20	4,189	222	300
EL75	8	268	295	399
EL80	20	4,189	235	317
EL80R (Relift)	10	524	144	194
EL85	10	524	244	330
EL89GR2 (Relift 2)	8	268	122	165

Table 2- 18. East High Canal - air chamber data and sizes

Lateral Turnout or Pumping Plant	Air Chamber Diameter (sphere) (ft)	Volume of Air Chamber (ft^3)	Maximum Pump Head (ft)	Transient Head Estimate (ft)
EH4	8	268	320	432
EH11	8	268	221	298
EH15G	Gravity Lateral, no air chamber required			
EH19	10	524	239	323
EH19R (Relift)	10	524	251	339
EH29	8	268	177	239
EH33	10	524	329	444
EH35	None Required, elevated tank located at pumping plant			
EH42	20	4,189	169	228
EH47	None Required, elevated tank located at pumping plant			
EH50R (Relift)	None Required, elevated tank located at pumping plant			

Table 2- 19. Black Rock Branch Canal - air chamber data and sizes

Lateral Turnout or Pumping Plant	Air Chamber Diameter (sphere) (ft)	Volume of Air Chamber (ft^3)	Maximum Pump Head (ft)	Transient Head Estimate (ft)
BRB2	None Required, elevated tank located at pumping plant			
BRB7	10	524	190	256
BRB7R (Relift)	10	524	174	235
BRB11	None Required, elevated tank located at pumping plant			
BRB17	None Required, elevated tank located at pumping plant			
BRB18	None Required, elevated tank located at pumping plant			
BRB27	None Required, elevated tank located at pumping plant			
BRB28	None Required, elevated tank located at pumping plant			
BRB29G	Gravity Lateral, no air chamber required			

Locations of blowoffs and airvalves – These features will be located as needed at highpoints (airvalves) and low points (blowoffs) of pipe laterals and sublaterals. Computer software Bentley WaterCAD Version 8i design models were used to locate blowoffs and airvalves and are enumerated in the quantity estimate sheets for each pipe lateral.

2.2.12.3. Pipe Hydraulic Design

Steady state hydraulics and pipeline friction loss – The Darcy-Weisbach formula was used to determine the pipeline friction loss of the lateral systems. The flow of liquid through a pipe is resisted by viscous shear stresses within the liquid and the turbulence that occurs along the internal walls of the pipe, created by the roughness of the pipe material. This resistance is usually known as pipe friction and is measured in feet head of the fluid; thus, the term head loss is also used to express the resistance to flow.

Weisbach first proposed the equation we now know as the Darcy-Weisbach formula or Darcy-Weisbach equation:

$h_f = f\,(L/D) \times (v^2/2g)$

where:
h_f = head loss (ft)
f = friction factor
L = length of pipe work (ft)
d = inner diameter of pipe work (ft)
v = velocity of fluid (ft/s)
g = acceleration due to gravity (ft/s^2)

Darcy introduced the concept of relative roughness, where the ratio of the internal roughness of a pipe to the internal diameter of a pipe will affect the friction factor for turbulent flow. In a relatively smoother pipe, the turbulence along the pipe walls has less overall effect; hence, a lower friction factor is applied. For the feasibility study, computer software Bentley WaterCAD Version 8i models, a friction factor of 0.002 foot (mortar lining) was used for all pipe material types for nominal diameters of 30 inches and greater. A friction factor of 0.0001 foot (plastic) was used for all pipe material types for nominal diameters of 24 inches and smaller, except for high-pressure steel pipes where 0.002 foot (mortar lining) was used.

The Darcy Friction factor used with Weisbach equation has now become the standard head loss equation for calculating head loss in pipes where the flow is turbulent. Initially, the Darcy-Weisbach equation was difficult to apply, since no electronic calculators were available and many calculations had to be carried out by hand. The development of the personnel computer from the 1980's onwards reduced the time needed to perform the friction factor and head loss calculations, which in turn has widened the use of the Darcy-Weisbach formula to the point that all other formula are now largely unused.

Pipeline Profiles - The pipeline profiles were not created for this report, though they are all embedded as part of the computer software Bentley WaterCAD Version 8i models developed for each pipe lateral. The basic guidelines to be used at final design are:

1. Minimum cover over the top of the pipe for estimating and cost optimization is 3 feet,

2. Minimum slope for the pipe diameters larger than 24-inches should be 0.00040,

3. Vertical bends when deflection angles were greater than 5°, and horizontal bends greater than 10° should be mitered bends,

4. Blowoffs are used to drain to the low area of the pipeline as much as practical, and

5. Air valves are used to expel/admit air.

Economic Sizing Of Pump Discharge Lines Having Lateral Turnouts[12] **–** The same principles of design apply to the high-head pressure mains and pumps as to a low-head pipe system. The major problems are control of water hammer and the regulation of operating capacities and pressures in the system. The distribution system with the minimum cost will deliver just enough water to meet peak irrigation demands if the farm sprinkler systems are operated 24 hours a day, less the time needed for changing "sprinkler sets." Systems that have larger pipe capacities permit a farmer to install large "on-farm" systems so that he can cover his land faster and only sprinkle part of the time. Naturally the larger pipe systems cost more money. What percentage the increase can be determined if it is assumed that the elevation of the water source and the pressure furnished at the farm remain the same. The economic pipe sizes for a gravity system will be one set of sizes that uses all available head in friction when delivering the peak flow to hydraulic control points in the system. Only one sequence of pipe diameters will produce the minimum total pipe cost. From inspection it is apparent that the steep friction slopes should be used with pipes carrying the larger flows and relatively flatter friction slopes used with pipes conveying smaller flows.

Items of cost include:

1. Pumping plant, including prime movers and electrical controls,
2. Pipe discharge lines between pumping plant and reservoir or tank,
3. Annual costs of electrical energy, and
4. Annual cost for maintenance and operation.

The static lift is constant. The friction head varies with the size of the pipe. Where the friction head is a small fraction of the total pump head, the cost of the pumping plant and equipment will be considered constant for small changes in friction head. The annual cost for maintenance and operation can also be considered a constant for small changes in friction head. Since both operation and maintenance and pumping plant costs are considered constant, they can both be neglected from the analysis.

The objective of this analysis is, therefore, to determine a pipe diameter that will provide the smallest combination of pipe cost and the annual cost for energy used in overcoming pipe friction in the pump discharge line.

Irrigation demand will vary widely over the season. The peak flow will only be required for 10 days to 2 weeks. The feasibility designs were conducted using a 30 day average demand flow for every individual month. At final design the 10 day to 2 week peak flow conditions will need to be analyzed with the assistance of an agricultural engineer to ensure that an acceptable system design capacity is provided.

[12] Reclamation, James E. Mandry, Design of Pipe Distribution Systems for Sprinkler Projects, September 1967

The use of computer software Bentley WaterCAD Version 8i, Darwin Designer, with its genetic-algorithm-based approach, avoids the manual trial-and-error approach to finding the most efficient design. For this study, the optimized solution obtained from the computer software Bentley WaterCAD Version 8i, Darwin Designer models was used for the Most Probable design and cost estimates. With additional design efforts during final design, some further cost optimization will be possible by examining pipe diameter transition locations as well as incorporating precise alignment locations upon obtaining further ground level field design data. Where tank locations are a significant distance from pumping plants, pipe diameters were required to be larger between pumping plants and tanks as a result of available types of pumps that can function with the associated friction losses between the pumping plants and tanks.

Regulation of Pressure and Flow – One of the farmers' primary requirements for sprinkler service is a constant pressure and flow rate of delivery. Each zone consists of a pumping plant which discharges into a tank with a water surface having an elevation high enough to furnish backpressure adequate for sprinkler pressure at each farm in the zone. This back pressure also functions to keep a constant head on the pumps and prevents undesirable pump discharge fluctuations.

Automatic Operation Of Pumping Plants – The operation of the pump units is controlled by the water level in the tank on the discharge side of the pumping plant. The information is transmitted by an electrical messenger cable or a leased telephone circuit. Water levels should be set to turn pumps "on" when the water surface falls and "off" when the water surface rises as shown in Figure 2- 6. When this is done, the pump units respond automatically to any change in demand.

Sizing Pump Units – Pumps of graduated size will provide the greatest flexibility with a reasonable number of units. The capacity of the smallest pump is determined by the permissible pipe friction variation at the controlling delivery. This smallest pump is called the regulating pump. The water level controls for the regulating pump are set just above and below the normal operating water surface in the tank (see Figure 2- 6). During peak periods when all pumps are running, the fluctuation in friction between small pump "on" and small pump "off" may be undesirably large. This may dictate selection of a smaller regulating pump.

Pump motors going on and off at short intervals will burn up. Tanks have to be designed to assure that the pumps will run long enough to cool off after each start. Since the regulating pump will go on and off most often, a criteria for tank volume between the "on" and "off" water levels for the regulating pump can be established. This varies with the size of the regulating motor and ranges from 10 minutes discharge for the small motor to 90 minutes for motors of over 500 hp. Since large tanks are expensive, selection of small regulating pumps may affect a considerable savings.

As the regulating pump is operated through the entire season, two or more of these sized pumps should be installed in any plant. Only one pump can operate as a regulator at any one time but installation of an electrical selector switch will permit the small pumps to be used alternately as the regulator.

In selecting pump sizes it has been found advantageous to select full size and fractional sized units. For instance, full size, half size, and quarter size pumps may be placed in the same plant. One method is to have the sum of the capacities of the fractional pumps equal the capacity of a full size unit. More sophisticated variable speed control systems of pumps may also be used for flow regulation.

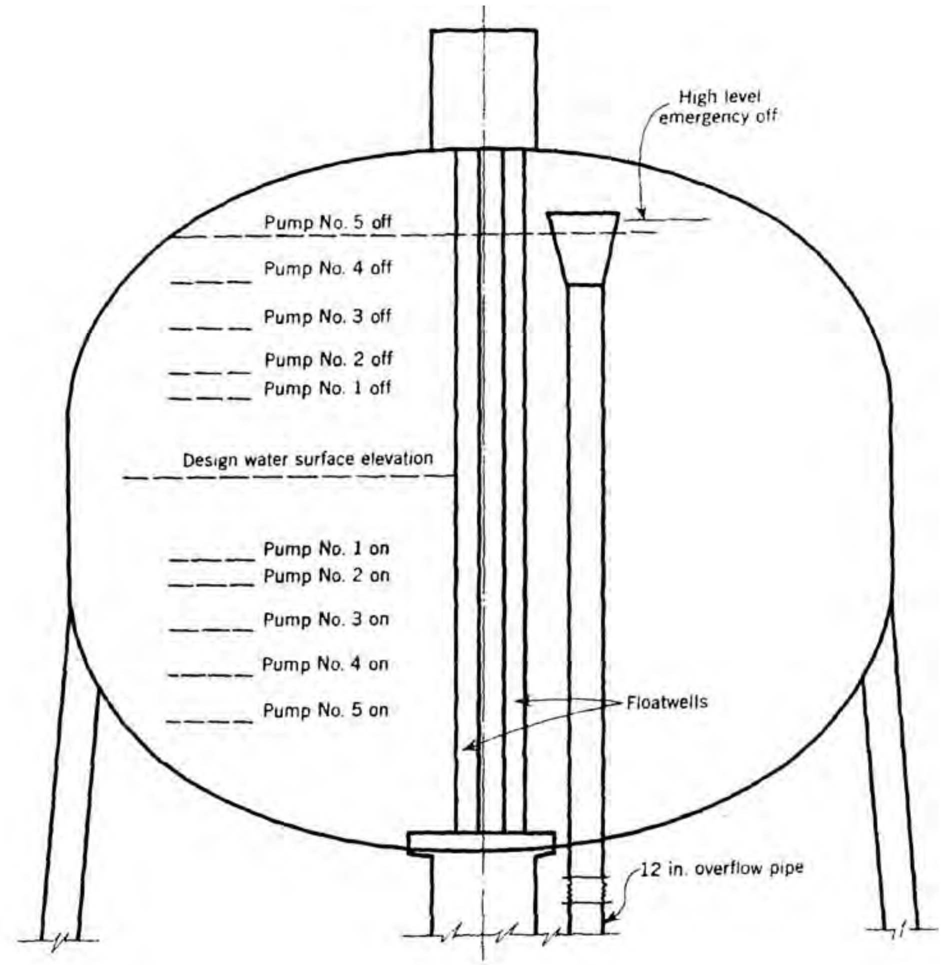

FIG. 5.—ELEVATED TANK

Figure 2- 6. Typical elevated tank

Pressure Control Valves – Operating pressure supplied to any farm must be sufficient to provide for (1) the design pressure at the base of the sprinkler nozzle plus (2) farm and farm system losses. On a project-type sprinkler system, the friction head in the long pipe feeder lines causes the pressure at the farm to fluctuate with changing demand during the irrigating season. Some pipe feeder lines could be fairly long, for example, a lateral that is approximately 3 miles long and has approximately 22 psi of friction loss at peak flows. The corresponding friction loss for minimum flows is approximately zero. For instance, assume the end is supplied with 35 psi operating farm pressure when the feeder line is delivering maximum flow, then when minimum flow is required, that same end will be supplied with approximately 57 psi. In this case the farmer's equipment will be used to throttle out the 22 psi excess pressure. Delivery locations for laterals on this feasibility study have been placed to try to reduce the length that the farm systems will require to approximately 0.5 mile maximum. This would reduce the local friction losses at peak flows from the previous example to approximately 3.5 psi. The proper location of delivery boxes therefore significantly reduces the farmer's issues with fluctuating pressures due to peak flow to minimum flow variations.

Sprinkler systems may be installed on steeply sloping or undulating terrain and high pressures at farm deliveries are often unavoidable. Where the farmer must install a sprinkler system with a design nozzle pressure of 30 psi, the excess pressure is often greater than the farmer can reasonably be expected to throttle out. This problem has been studied on several projects and the consensus is that the maximum pressure supplied at a farm should not exceed 230 feet (100 psi). Accordingly, pressure-reducing valves are installed in these systems to limit pressures to 230 feet (100 psi).

Several pressure-reducing diaphragm valves are manufactured. They are generally globe valves and have relatively high head loss. They have a screening device to exclude fine sand and silt from entering the small diameter tubing that connects to the diaphragm pressure-sensing chamber. Should this screening device plug with silt or debris, the main valve will fail to open and cease to function as a pressure reducer.

A pressure-reducing valve may be installed in an individual farm delivery. The downstream pressure is set to the farm requirements. If, however, there are a great number of high-pressure deliveries, the initial operating and maintenance costs for farm installations may become too large and it becomes more economical to use a central pressure-reducing station on the feeder pipeline.

Computer Software Bentley WaterCAD Version 8i, Darwin Designer Input – The annual allotment of water the project is based upon is 3.0 acre-feet/acre. Monthly irrigation amounts of water delivered to compute annual pumping costs are based on the following:

Table 2- 20. Monthly irrigation amounts of water delivered to compute annual pumping costs[13]

Month	% of Allotment	Peak Flow/ 100 acres	% of Month	Days of Irrigation	Demand Factor for WaterCAD
Mar	0.5%	1.34	1.8%	0.6	0.02
Apr	3.5%	1.39	12.6%	3.8	0.13
May	13.3%	1.34	48.0%	14.9	0.48
Jun	20.1%	1.38	72.6%	21.8	0.73
Jul	27.7%	1.49	90.0%	27.9	1.00
Aug	21.7%	1.35	78.0%	24.2	0.78
Sep	10.8%	1.38	39.0%	11.7	0.39
Oct	2.4%	1.33	8.7%	2.7	0.09

[13] Output from WaterCAD Version 8i

Table 2- 20. Monthly irrigation amounts of water delivered to compute annual pumping costs[13]

Month	% of Allotment	Peak Flow/ 100 acres	% of Month	Days of Irrigation	Demand Factor for WaterCAD
Total	100.0%			107.5	

Also utilize the following information as applicable:

Maximum pump unit size - 40 ft^3/s
Pump and motor combined efficiency – 70%
Minimum pressure – 5 psi (required minimum for air valves)
Minimum pressure at deliveries – 16.5 psi (38 ft) (Lateral side of delivery)

Minimum pressure at deliveries – 10 psi (23 ft) (Farm side of delivery)

Maximum allowed pipe velocity – 10 fps
Pipe roughness – 0.002 feet rugosity (Darcy formula) for pipe diameters 30 inches and larger (steel, mortar lined)
0.0001 feet rugosity for pipe diameters 24 inches and smaller (PVC)
Viscosity – 1.41 $x10^{-5}$ ft^2/s (water @ 50°F)
Liquid Specific Gravity – 0.998

Each delivery point has a "box" which contains a standard set of equipment. This delivery box will vary for each location due to the farmer's irrigated acreage, irrigation methods, and lateral line pressures. The equipment consists of:

1) Steel piping,

2) 2 Butterfly valve,

3) Magnometer flowmeter,

4) Pressure-reducing valve (globe style), and

5) Air valve.

The computer software Bentley WaterCAD Version 8i, Darwin Designer, was used in modeling and optimizing the pipe laterals. Deliveries that provide pressure less than 100 feet (43 psi) of pressure, will require the farmer to provide their own booster pump to obtain the necessary pressure required for their individual type of irrigation system. When static pressure is greater than 100 feet (43 psi) but less than 230 feet (100 psi), it was assumed that the farmer has sufficient pressure to operate his sprinkler system. When static pressure exceeds 100 psi (230 feet), a PRV is needed and will be provided by the project.

At feasibility level, it was assumed that 15 feet (6.5 psi) is the total loss through a box consisting of:

50 feet of PVC x-distance pipe
25 feet of steel piping
4 – 90-degree elbows
2 - butterfly valves
1 - globe style hydraulic operated valve
1 - air valve
1 - tee from lateral pipe

2.2.12.4. Globe valve

The feasibility design uses low-loss type valves to close and stop draining following a power outage. During operations, the valve will open fully and have a small (relative) loss (could oversize or use other suitable valve style).

2.2.12.5. Other valve locations (isolation)

Isolation valves should be located at each sublateral junction to isolate sublaterals from the rest of the distribution lateral.

2.2.12.6. Energy Cost

The lateral pumping plants will be supplied Columbia Basin Project power at the rate of $0.002/ kW (value provided by Reclamation's Ephrata Field Office).

2.2.12.7. Pipe Cost

Reclamation estimated installed pipe cost for this study by soliciting purchase prices from suppliers and adding installation costs. Pipe diameters of 24-inch and smaller were assumed to be PVC pipe, and larger sizes were mortar-lined steel pipe. Pipe trench excavation costs were estimated using an assumed proportional split of 35-percent rock excavation (0.5 horizontal:1 vertical) side slopes and 65-percent earth excavation (1.5 horizontal:1 verticle) side slopes.

2.2.12.8. Valves

Various valve loss formula:

Where:

Q = flow rate, ft^3/s
H = Head, ft
C_v = gpm that causes 1 psi loss through fully open valve
K = Loss coefficient based on velocity head ($V^2/2g$)

$$C_v = 681.82 \frac{Q}{\sqrt{H}}$$

$$K = \left(681.822 \frac{Q}{C_v}\right)^2 2g\left(\frac{\Pi d^2}{4}\right)^2$$, if valve is 100% open

Pipe Bends – When a pipeline is under internal pressure, unbalanced forces develop at changes in direction. Restraint of the thrust from the bend may be provided by development of friction forces between the pipe and the soil, by the passive earth reaction force at the bend, or by the use of blocking. The pipeline was designed with flexible joints that will require bend thrust restraint fittings.

Hydraulic Surge (water hammer) – These pipe distribution systems present difficult problems in controlling pressure surges. Each system is a special study in itself. Air chambers, one-way surge tanks, hydraulic control valves, and additional motor flywheel effect (WR^2) may be needed frequently to control pressure surges. Normally, the pump heads are too high for the effective use of simple surge tanks adjacent to the pumping plants. For this feasibility study, transient analyses were not performed; therefore, a 35-percent increase in pipe pressure class was used for estimating purposes in the quantity estimate worksheets. Specific transient analysis will have to be done at final design.

There are two basic modes of pump operation—normal or emergency. Each requires a separate set of assumptions, applications, and methods of evaluation. Normal operation of the system—while trying to accommodate a continually changing flow demand—will be governed by: 1) mechanical limitations: establishing minimum and maximum run times to prevent overheating and abnormal wear of the pumps; and 2) electrical limitations: setting the number of allowable starts per day per pump to limit power surges and electrical overloading. This is of practical significance during normal operation in that one pump will start or stop, in reply to any change in flow demand thereby eliminating any accumulation of effects due to multiple disturbances on the system. During emergency conditions resulting from electric power failure or interruption, a leak in the line, or contact with an emergency level in either the forebay or afterbay, all operating pumps could shut down simultaneously.

When the electrical power supply to the pump motors is suddenly cut off, the only energy that is left to drive the pump in the forward direction is the kinetic energy of the rotating elements. Since this energy is small when compared with that required to maintain the flow against the discharge head, the reduction in pump speed is quite rapid. As the pump speed reduces, the flow of water in the discharge line adjacent to the pump is also reduced. As a result of these rapid flow changes, low-pressure waves (downsurge) move rapidly up the discharge line to the discharge outlet, where wave reflections occur. Soon the speed of the pump is reduced to a point where no water can be delivered against the existing head. If the low-pressure wave causes the pressure at any point along the discharge pipeline to drop below the vapor pressure of water (about negative 32 feet), the liquid water column is separated by a section of vapor. Low pressures have resulted in the collapse of pipes in many systems. In addition to risks of pipe collapse from low pressure, if water column separation occurs, unpredictable and very high pressures can result when the separated columns rejoin, and these high pressures can rupture the pipeline. When the flow from the pump reduces to near zero, the check valve closes and prevents reverse flow.

Air valves are installed along the pipeline to expel and admit air during normal operations, filling and draining. Reclamation does not depend on air valves to admit the very large volumes of air very rapidly at the correct location to prevent low-pressure transients. Air valves are mechanical items that at times may not be properly maintained to have the original capabilities. Transient analyses are performed assuming no air is admitted into the pipeline.

Pump and surge tank characteristics discussions follow.

Pump Characteristics – Hydraulic transient analyses are based on assumed pump characteristic values. Pump data included were: rated head, rated flow, rotational speed, and motor/pump momentum (WR^2) values. These values were based on general characteristics of pumps that meet the required flow and pressure conditions of the system.

The WR^2 is based on general characteristics of the rotational portions of pump and motor of the required horsepower and flow range. It is the product of the weight of revolving parts and the square of the radius of gyration. Pump flows used in this analysis are based on peak flow requirements. When the electrical power supply to the pump motors is suddenly stopped, only the kinetic energy (WR^2) of the motor and pump rotating elements remain to drive the pump and it is quickly used up.

The results of any hydraulic analysis of pumps or pumping systems can be significantly affected by the pump operating characteristics used in the analysis. Approximate pump characteristics are usually used to perform such an analysis, since the actual characteristics are rarely known for a given pump. The difficulty of producing actual pump characteristics has long been considered justification for the use of these approximations. Nevertheless, an analysis using these approximations will yield approximate results.

Air Chamber Characteristics – The steady state volume in the air chamber during normal operation will lie between the COMPRESSOR ON volume and the COMPRESSOR OFF volume. The starting or stopping of one pump may cause the water surface to fluctuate beyond either one or both of these levels. The maximum fluctuation of air is the EMERGENCY OFF minimum volume and the AIR RELEASE maximum volume.

The EMERGENCY OFF and TOTAL VOLUMES were sized according to emergency conditions. The air chamber controls the transient upsurge pressure and will not dewater, which would allow air to enter the line, on the down surge pressure.

If the water level goes to the AIR RELEASE level due to an air compressor malfunction or any other reason, the air release valve (or valves) shall release a minimum 110 percent of the amount being delivered by the air compressor. This will keep the water level from dropping below the air release level.

Air Chamber – EHC Black Rock Coulee Pumping Plant No. 1

Surge Analysis – The hydraulic transients were analyzed for each pumping plant and discharge line to determine if surge protection was required. The analysis predicted that a sizeable down surge would be generated at the EHC Black Rock Coulee Pumping Plant No. 1 from a power failure causing simultaneous pump shutdown. Therefore, an air chamber was required to eliminate the negative pressures. The computer program TAPS (Transient Analysis of Pipe Systems) was used to determine various water operational levels in the air chamber.

Design Requirements – The total volume of the 22-foot diameter spherical air chamber is 5,575 cubic feet and contains a minimum level of 2,000 ft^3 air (compressor on level), a normal operating pressure of 90 psig (TDH=205 feet). The volume and design pressure were determined by hydraulic transient simulations. An air chamber of sufficient capacity is required to control the expected upsurge, and to admit a sufficient volume of water into the discharge pipe during down surges.

The air chamber has a 24-inch manhole for inspection and maintenance access and ventilation when painting. Ladders and platforms were installed on the air chamber to provide access for inspection and maintenance.

For protection from below-freezing temperatures while in use, the valves, switches, and small piping will be wrapped with heat tape and insulated.

Paints and linings were selected with corrosion prevention and long life as the major considerations. Aesthetic appearance was also considered for exposed exterior surfaces. The exterior of the air chamber exposed to sunlight is required to be painted with priming paint, and then two coats of silicone-alkyd enamel. The inside of the air chamber and portions not exposed to sunlight are to be painted with coal-tar epoxy paint.

Design Stresses and Codes – The air chamber is to be fabricated from ASTM A516, grade 60 or 70 steel. This steel is readily weldable and has physical properties most applicable for the intended pressure vessel design. The air chamber is designed and fabricated in accordance with the requirements of Section VIII, Division I of the American Society of Mechanical Engineers (ASME) Boiler and Pressure Vessel Code. The air chamber is to be endorsed with ASME code stamp.

Air Chamber Operation – The air compressor delivers air to the chamber through piping connected to an inlet at the top of the chamber. The gate valve on the air inlet piping should be open. The air relief valve is located on a tee off of the air inlet piping. The air relief valve will discharge air in the event of excessive pressure due to system malfunction. The air relief valve is set to operate only if the air release valve fails to open. The ball valves located just below and just above the air release valve should both be open. The check valve near the compressor will prevent water from entering the compressor. The automatic drain valve is for drainage of condensate inside the air inlet piping. The ball valve next to the drain valve should be open at all times during operation. The gate valve on the pipe that goes into the air chamber, located just above the relief valve assembly should be open.

The probe assembly mounted on the chamber controls the operation of the air compressor. The probe contacts are adjusted so that during normal operation the water level will be between COMPRESSOR OFF and COMPRESSOR ON. If the water level goes above EMERGENCY PUMPS OFF due to compressor malfunction, the pumps will be shut off. The ball valve above the probe unit housing should be open during operation. The ball valve on the pipe that connects the bottom of the probe assembly piping to the chamber should be open.

The sight gauge provides a visual check of the water level in the chamber. The air release valve will discharge air if the water level falls below the AIR RELEASE level.

2.2.12.9. Typical pipe trench section

Drawing No. 222-D-52066 shows a typical trench section for each proposed pipe option, rock excavation, or earth excavation. The typical pipe section assumes an average earth cover of 3 feet. Special installation requirements for compacted pipe trenches under road crossings, miter bend locations, and other special features will need to be addressed during final design.

2.2.12.10. Debris and sediment

Turnout structures from the canal to the pumping plants have trash racks and debris screens to preventing debris and sediment from entering the pipeline.

2.2.12.11. Flow Measurement

Flow measurement is required at pumping plants as well as at farm delivery boxes. For this feasibility study, ultrasonic flow measurement has been assumed for the main pumping plants and magnometer-type of flow measurement has been assumed for the farm delivery boxes, given preliminary information from ECBID. ECBID will provide delivery flow measurement equipment specifications for Final Design.

2.2.12.12. Corrosion protection requirements

Preliminary investigations conducted during the Weber Branch and Weber Coulee Siphon project revealed soil types that may be conducive to a corrosive environment for steel construction materials. Most lateral piping 24 inches and smaller is recommended to be PVC line pipe. Where steel pipe is required (large diameter and small diameter/high pressure), cathodic protection is recommended and has been provided for in the construction cost estimates. For this feasibility study, concrete mortar lining and polyethylene tape coating protection systems have been identified in the cost estimates.

Site-specific soil investigations will be required to finalize lining and coating requirements for the steel pipe laterals.

2.2.12.13. Operation criteria

Pipe laterals vary between high pressure and low pressure, dependent upon the terrain that the lateral traverses to provide water to required farm fields. High-pressure systems enable fewer pumping plants, elevated water tanks, and minimized length of pipelines to be constructed. Low-pressure systems allow for less expensive pipe types to be installed. For all lateral designs, there is a balance of the pressure constraints of providing a minimum line pressure of 5 psi and 10 psi minimum at lateral delivery points, with the location of pumping plants, tanks, and minimizing overall length of pipe laterals.

2.2.13 **Tunnels**

The tunnels on this project are used to convey water through geologic ridges within the canal conveyance system.

2.2.13.1. Stratford Tunnel

Stratford Tunnel is a part of the East High Canal system of the Columbia Basin Project which will convey agricultural water to farmers in the East Columbia Basin Irrigation District. Stratford Tunnel is located approximately 3.5 miles north and east of Stratford, Washington. The tunnel, as estimated, is a 1,607-foot-long, horseshoe-shaped, 26.5-foot finished diameter tunnel.

Geology – Basalt crops out across the top and slope of the hill through which the tunnel would be excavated. However, there are no drill holes into the basalt at the tunnel site, and the stratum (or strata) that the tunnel would be excavated through does not outcrop on the adjacent Long Lake Coulee wall, as it is beneath talus rising from the coulee floor. Hence, the nature of the rock that excavation of the tunnel will encounter was inferred from the geologist’s knowledge of basalt in general.

The following assumptions are from the geologic report (Reclamation, 1963): 1) the proposed tunnel would be excavated through basalt from portal to portal, unless it intersects a thin interflow layer of alluvial sediments; and 2) the tunnel probably would not intersect any faults.

A general discourse on basalt was included in the geologic report and reflects what conditions the geologist thought may be encountered and is therefore repeated here. Basalt flows are divided internally into intraflow zones, which are the products of differential cooling rates; and, from place to place, the flows are separated, overlying stratum from underlying stratum, by soil profiles, or alluvial sediments. An interflow zone registers a hiatus in lava extrusion. During that hiatus, the basalt was weathered and eroded, and clastic sediments were deposited in local lake basins and along newly developed stream courses. The youthful terrain and its accompanying soil zone and sediments then were inundated when basalt once again poured across the region.

The basalt intraflow zones usually include, in ascending order:

1) a basal zone of hard, dense, extremely fine-grained, highly and irregularly fractured rock overlain by:

2) a zone of hard, dense, polygonal-columned, vertical-jointed rock referred to descriptively as the colonnade;

3) a zone of platy, horizontal jointed, wavy-columned, often vesicular rock referred to as the entablature; and

4) the uppermost zone of rapidly cooled, scoriaceous, vesicular, cindery material.

Alignment and Profile – The alignment was determined prior to this feasibility design report and no documentation on the specific reasoning for its selection was found. See Drawing No. 222-D-50219 for the plan and profile of Stratford Tunnel.

Comparison to Nearby Tunnels – Bacon Tunnel (first unit) and Bacon Tunnel (second unit) are two parallel tunnels constructed in geology thought to be similar to what would be encountered by Stratford Tunnel. They are located approximately 7.5 miles north and west of the Stratford Tunnel site. The condition and structural performance of these tunnels is unknown at the time of this writing.

Construction – The length of the tunnel is 1,607 feet. It was assumed that excavation by mechanical boring with a tunnel boring machine would not be economically competitive with excavation by drill and blasting for a tunnel of this short length. The perceived efficiency of mechanically boring over drill and blast excavation is offset by the required mobilization and demobilization time when the length of the tunnel is short. It was also assumed that a roadheader could not efficiently excavate competent basalt. Two viable methods of drill and blast excavation were estimated: 1) full face; and 2) top heading.

An excavated horseshoe-shape tunnel was estimated. This shape would be appropriate in hard rock where only light side pressures are anticipated. While a circular shape could also resist lateral pressures, the flatter floor of the horseshoe shape facilitates the contractor’s travel within the tunnel. Excavation quantities were based on B-line dimensions (see Figure 2- 6). The excavated (B-line) diameter was 31.33 feet. The A-line to B-line dimension is 18 inches (Reclamation, 1994). The A-line to B-line dimension allows an estimate of excavation outside of the A-line needed to place the initial support and includes an allowance for overbreak. There would be no negative impact from this size of the tunnel impeding excavation progress.

The average advance rate for full-face excavation of Stratford Tunnel was assumed to be 15 feet per day. This average advance rate was compared to that of Bacon Tunnel (first unit) which was a similar-size tunnel (26.25-foot excavated diameter) and assumed to be excavated full face in similar geology. Reclamation’s tunnel database showed an average advance rate of 15 feet per day for Bacon Tunnel (first unit).

The average advance rate for the top heading method was assumed to be 39 feet per day for the top heading and 62 feet per day for the bench heading. The average advance rate for both headings combined was calculated to be 46 feet per day. The advance rate for the top heading method was based on information gleaned from construction geology reports for Bacon Siphon and Tunnel – Second Unit (Reference to be furnished in final report). It assumed a 6-day workweek for both the top heading and bench heading. The advance rate shown in Reclamation's tunnel database was 49.2 feet per day. For comparison, the SME Mining Engineering Handbook (SME, 1973) estimates the average advance rate for a large tunnel (a face area between 250 ft^2 and 400 ft^2) is 25.7 feet per day in supported sections and 44.5 feet per day in unsupported sections of tunnel.

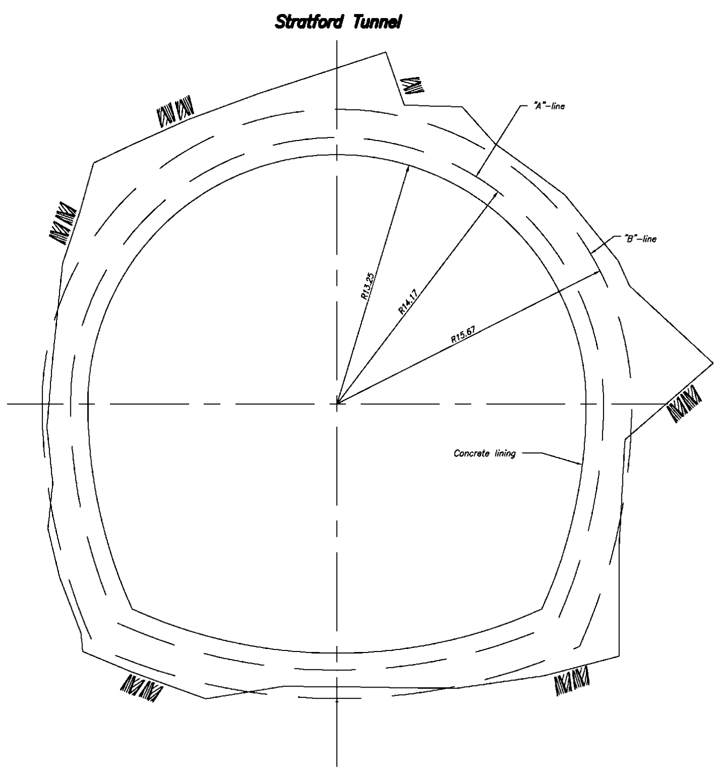

Figure 2- 7. Typical cross-section through Stratford Tunnel

It was assumed that groundwater infiltration into the tunnel during construction would be negligible. It was assumed that large inflows from point sources may occur but would bleed off and be short lived.

Tunnel Design

General – Access roads, contractor staging areas, tunnel muck disposal sites, a wastewater treatment site and open-cut portal excavations were not evaluated. They were assumed unlisted items. From the general geology, it was assumed that Stratford Tunnel would be classified non-gassy.

Hydraulics – The required flow to supply the peak agricultural needs of this portion of the project was estimated to be 5,958 ft^3/sec. Additionally, surface runoff from storms will periodically enter the canal. The design flow was defined to be the combined flow to supply the peak agricultural needs and the flow from the surface runoff entering the canal. The design flow was estimated to be 6,238 ft^3/sec. The tunnel was sized as a free-flow tunnel. The Manning equation was used for hydraulic calculations. A rough (wood formed with an eroded invert) concrete lining surface with a Manning's "n" of 0.014 was assumed for hydraulic calculations. The free-flow depth should be approximately 0.82 times the internal diameter of the tunnel. The velocity would be 12.4 feet per second. The conveyed water was assumed free of sediment. Velocities in excess of 10 feet per second are acceptable if the water is free of silt, sand, or gravel (Reclamation, 1994).

Ground stabilization – Prior to the start of tunnel excavation, it is assumed that horizontal rock reinforcement would be installed around the portal openings. No other pre-tunneling ground stabilization is anticipated.

Initial Support – Bacon Tunnel (first unit) was constructed between 1946 and 1950. It was unsupported for 90 percent of its length and supported by steel sets for 10 percent of its length. Bacon Tunnel (second unit) was constructed from 1976 to 1978 by the top heading method. The top heading was supported for 100 percent of the tunnel length by steel sets with auxiliary rock bolts to support the wall plate ledge. Some shotcrete was also used. Reclamation's tunnel database records that: 1) soil-like zones within hard rock needed additional supports; and 2) a wall plate ledge slid into the tunnel, failing 118 feet of rib-supported tunnel. The two tunnels are only 400 feet apart. It is not understood why the two tunnels had such different support requirements. It can only be speculated that the Bacon Tunnel (second unit) contractor thought that supplying possibly unneeded support would allow a more rapid, but safe, advance of the tunnel and that completing the tunneling in a shorter time would offset the increased cost of the additional support. Consequently, two estimates were generated which reflected the difference in support quantities. One estimate assumed 10 percent of the length of the tunnel would be supported with structural steel sets and the other estimate assumed that 100 percent of the length of the tunnel would be supported with structural steel sets and rockbolts. Initial support consisting mostly of rock reinforcement should be investigated in final design.

The weight of steel supports (pounds per linear foot of tunnel) installed in Bacon Tunnel was used to back-calculate (Reclamation, 1967) that the rock encountered would be classified as moderately blocky and seamy, while the weight of steel supports installed in Bacon Tunnel (second unit) indicated very blocky and seamy rock. Although the difference in rock conditions seems somewhat incongruous for tunnels only 400 feet apart, the estimated steel support sizes for Stratford Tunnel, modified for the smaller tunnel diameter, were based on these assumptions.

Lining – Joints in the rock were assumed to be open to wide (Reclamation, 1998). Regardless of the need for final support, a concrete lining was deemed necessary to prevent egress (loss) of the conveyed water from the tunnel into the surrounding rock.

It is assumed that any water wells are sufficiently far removed from the tunnel that a drop in the water surface above the tunnel will not result in a lower groundwater surface at any wells. It is assumed that there will be no contamination of the conveyed water by water from the rock encompassing the tunnel. Hence, a watertight lining is not required and use of weep holes to minimize external hydrostatic pressure would be acceptable.

A finished horseshoe-shape tunnel was estimated with a diameter of 26.5 feet (see Figure 2- 6). This shape would be appropriate for the following conditions:

1. A tunnel excavated by drill and blast,

2. For a cast-in-place unreinforced lining where lateral rock loadings are anticipated, and

3. For a cast-in-place unreinforced concrete lining where some external hydrostatic pressure on the lining is anticipated. The thickness of the lining is 11 inches (Reclamation, 1994).

Grouting – Following placement of the lining, backfill grouting would be required to fill any voids and ensure contact of the lining with the rock in the crown of the tunnel.

Water Control – Weep holes cast or drilled through the tunnel lining will allow drainage of groundwater into the tunnel and minimize the external hydrostatic pressure against the lining. Weep holes should be placed above normal depth of the design flow if practical.

2.2.13.2. Long Lake Tunnel

Long Lake Tunnel is a part of the East High Canal system of the Columbia Basin Project which will convey agricultural water to farmers in the East Columbia Basin Irrigation District. Long Lake Tunnel is located approximately 5.3 miles north and east of Stratford, Washington. The tunnel, as estimated, is a 1,805 feet long, horseshoe shaped, 26.5 feet finished diameter tunnel.

Geology – Basalt crops out across the top and slope of the hill through which the tunnel would be excavated. However, there are no drill holes into the basalt at the tunnel site and the stratum (or strata) that the tunnel would be excavated through does not outcrop on the adjacent Long Lake Coulee wall, as it is beneath talus rising from the coulee floor. Hence, the nature of the rock that excavation of the tunnel will encounter was inferred from the geologist's knowledge of basalt in general.

The following assumptions are from the geologic report (Reclamation, 1963): 1) the proposed tunnel would be excavated through basalt from portal to portal, unless it intersects a thin interflow layer of alluvial sediments; and 2) the tunnel probably would not intersect any faults.

A general discourse on basalt was included in the geologic report and reflects what conditions the geologist thought may be encountered and is therefore repeated here.

Basalt flows are divided internally into intraflow zones, which are the products of differential cooling rates; and, from place to place, the flows are separated, overlying stratum from underlying stratum, by soil profiles, or alluvial sediments. An interflow zone registers a hiatus in lava extrusion. During that hiatus, the basalt was weathered and eroded, and clastic sediments were deposited in local lake basins and along newly developed stream courses. The youthful terrain and its accompanying soil zone and sediments then were inundated when basalt once again poured across the region.

The basalt intraflow zones usually include, in ascending order:

1) A basal zone of hard, dense, extremely fine-grained, highly and irregularly fractured rock overlain by:

2) A zone of hard, dense, polygonal-columned, vertical-jointed rock referred to descriptively as the colonnade;

3) A zone of platy, horizontal jointed, wavy-columned, often vesicular rock referred to as the entablature; and

4) The uppermost zone of rapidly cooled, scoriaceous, vesicular, cindery material.

Alignment and Profile – The alignment was determined prior to this feasibility design report and no documentation on the specific reasoning for its selection was found. See Drawing No. 222-D-50218 for the plan and profile of Long Lake Tunnel.

Comparison to Nearby Tunnels – Bacon Tunnel (first unit) and Bacon Tunnel (second unit) are two parallel tunnels constructed in geology thought to be similar to what would be encountered by Long Lake Tunnel. They are located approximately 5 miles north and west of the Long Lake Tunnel site. The condition and structural performance of these tunnels is unknown at the time of this writing.

Construction – The length of the tunnel is 1,805 feet. It was assumed that excavation by mechanical boring with a tunnel boring machine would not be economically competitive with excavation by drill and blasting for a tunnel of this short length. The perceived efficiency of mechanically boring over drill and blast excavation is offset by the required mobilization and demobilization time when the length of the tunnel is short. It was also assumed that a roadheader could not efficiently excavate competent basalt. Two viable methods of drill and blast excavation were estimated: 1) full face; and 2) top heading.

An excavated horseshoe-shape tunnel was estimated. This shape would be appropriate in hard rock where only light side pressures are anticipated. While a circular shape could also resist lateral pressures, the flatter floor of the horseshoe-shape facilitates the contractor's travel within the tunnel. Excavation quantities were based on B-line dimensions (see Figure 2- 8). The excavated (B-line) diameter was 31.33 feet. The A-line to B-line dimension is 18 inches (Reclamation, 1994). The A-line to B-line dimension allows an estimate of excavation outside of the A-line needed to place the initial support and includes an allowance for overbreak. There would be no negative impact from this size of the tunnel impeding excavation progress.

The average advance rate for full-face excavation of Long Lake Tunnel was assumed to be 15 feet per day. This average advance rate was compared to that of Bacon Tunnel (first unit) which was a similar-sized tunnel (26.25 feet excavated diameter) and assumed to be excavated full face in similar geology. Reclamation's tunnel database showed an average advance rate of 15 feet per day for Bacon Tunnel (first unit).

The average advance rate for the top heading method was assumed to be 39 feet per day for the top heading and 62 feet per day for the bench heading. The average advance rate for both headings combined was calculated to be 46 feet per day. The advance rate for the top heading method was based on information gleaned from construction geology reports for Bacon Siphon and Tunnel (second unit) (reference to be furnished in final report). It assumed a 6-day work week for both the top heading and bench heading. The advance rate shown in Reclamation's tunnel database was 49.2 feet per day. For comparison, the SME Mining Engineering Handbook (SME, 1973) estimates the average advance rate for a large tunnel (a face area between 250 ft^2 and 400 ft^2) is 25.7 feet per day in supported sections and 44.5 feet per day in unsupported sections of tunnel.

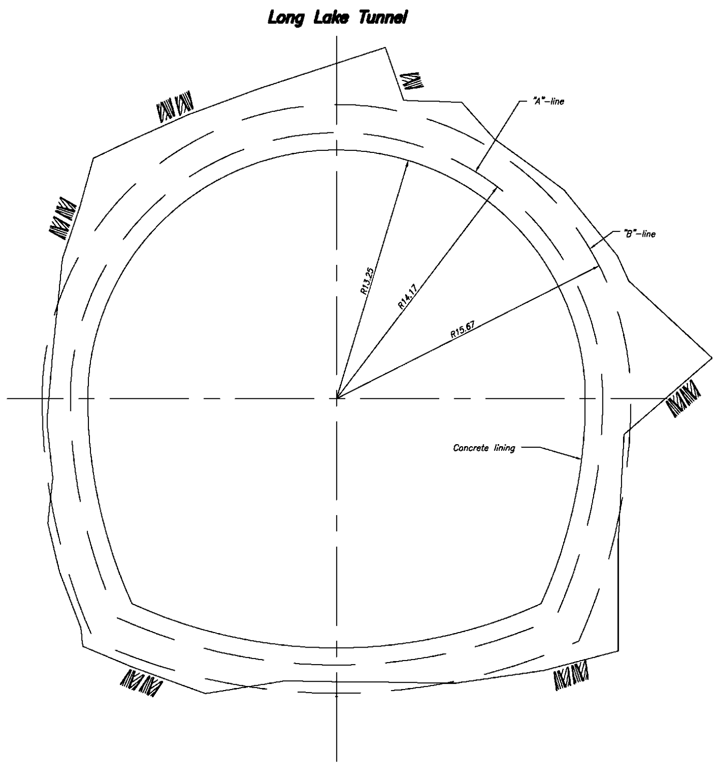

Figure 2- 8. Typical cross-section through Long Lake Tunnel

It was assumed that groundwater infiltration into the tunnel during construction would be negligible. It was assumed that large inflows from point sources may occur but would bleed off and be short lived.

2.2.13.2.1 ***Tunnel Design***

General – Access roads, contractor staging areas, tunnel muck disposal sites, a wastewater treatment site and open-cut portal excavations were not evaluated. They were assumed unlisted items. From the general geology, it was assumed that Long Lake Tunnel would be classified non-gassy.

Hydraulics – The required flow to supply the peak agricultural needs of this portion of the project was estimated to be 5,958 ft^3/sec. Additionally, surface runoff from storms will periodically enter the canal. The design flow was defined to be the combined flow to supply the peak agricultural needs and the flow from the surface runoff entering the canal. The design flow was estimated to be 6,238 ft^3/sec. The tunnel was sized as a free-flow tunnel. The Manning equation was used for hydraulic calculations. A rough (wood formed with an eroded invert) concrete lining surface with a Manning's n of 0.014 was assumed for hydraulic calculations. The free-flow depth should be approximately 0.82 times the internal diameter of the tunnel. The velocity would be 12.4 feet per second. The conveyed water was assumed free of sediment. Velocities in excess of 10 feet per second are acceptable if the water is free of silt, sand, or gravel (Reclamation, 1994).

Ground stabilization – Prior to the start of tunnel excavation, it is assumed that horizontal rock reinforcement would be installed around the portal openings. No other pre-tunneling ground stabilization is anticipated.

Initial Support – Bacon Tunnel was constructed between 1946 and 1950 and was unsupported for 90 percent of its length and was supported by steel sets for 10 percent of its length. Bacon Tunnel (second unit) was constructed from 1976 to 1978 by the top heading method. The top heading was supported for 100 percent of the tunnel length by steel sets with auxiliary rock bolts to support the wall plate ledge. Some shotcrete was also used. Reclamation's tunnel database records that: 1) soil-like zones within hard rock needed additional supports; and 2) a wall plate ledge slid into the tunnel, failing 118 feet of rib-supported tunnel. The two tunnels are only 400 feet apart. It is not understood why the two tunnels had such different support requirements. It can only be speculated that the Bacon Tunnel (second unit) contractor thought that supplying possibly unneeded support would allow a more rapid, but safe, advance of the tunnel and that completing the tunneling in a shorter time would offset the increased cost of the additional support. Consequently, two estimates were generated which reflected the difference in support quantities. One estimate assumed 10 percent of the length of the tunnel would be supported with structural steel sets and the other estimate assumed that 100 percent of the length of the tunnel would be supported with structural steel sets and rockbolts. Initial support consisting mostly of rock reinforcement should be investigated in final design.

The weight of steel supports (pounds per linear foot of tunnel) installed in Bacon Tunnel was used to back-calculate (Reclamation, 1967) that the rock encountered would be classified as moderately blocky and seamy, while the weight of steel supports installed in Bacon Tunnel (second unit) indicated very blocky and seamy rock. Although the difference in rock conditions seems somewhat incongruous for tunnels only 400 feet apart, the estimated steel support sizes for Long Lake Tunnel, modified for the smaller tunnel diameter, were based on these assumptions.

Lining – Joints in the rock were assumed to be open to wide (Reclamation, 1998). Regardless of the need for final support, a concrete lining was deemed necessary to prevent egress (loss) of the conveyed water from the tunnel into the surrounding rock.

It is assumed that any water wells are sufficiently far removed from the tunnel that a drop in the water surface above the tunnel will not result in a lower groundwater surface at any wells. It is assumed that there will be no contamination of the conveyed water by water from the rock encompassing the tunnel. Hence, a watertight lining is not required and use of weep holes to minimize external hydrostatic pressure would be acceptable.

A finished horseshoe-shape tunnel was estimated with a diameter of 26.5 feet (see Figure 2- 8). This shape would be appropriate for the following conditions: 1) a tunnel excavated by drill and blast; 2) for a cast-in-place unreinforced lining where lateral rock loadings are anticipated; and 3) for a cast-in-place unreinforced concrete lining where some external hydrostatic pressure on the lining is anticipated. The thickness of the lining is 11 inches (Reclamation, 1994).

Grouting – Following placement of the lining, backfill grouting will be required to fill any voids and ensure contact of the lining with the rock in the crown of the tunnel.

Water Control – Weep holes cast or drilled through the tunnel lining will allow drainage of groundwater into the tunnel and minimize the external hydrostatic pressure against the lining. Weep holes should be placed above normal depth of the design flow if practical.

2.2.13.3. Moody Tunnel

Moody Tunnel is a part of the East High Canal system of the Columbia Basin Project which will convey agricultural water to farmers in the East Columbia Basin Irrigation District. Moody Tunnel is located approximately 24 miles south and east of Stratford, Washington. The tunnel, as estimated, is a 2,473-foot-long, horseshoe-shaped, 12.0-foot finished diameter tunnel.

Geology – Basalt crops out across the top and slope of the hill through which the tunnel would be excavated. However, there are no drill holes into the basalt at the tunnel site and the stratum (or strata) that the tunnel would be excavated through does not outcrop on the adjacent Long Lake Coulee wall, as it is beneath talus rising from the coulee floor. Hence, the nature of the rock that excavation of the tunnel will encounter was inferred from the geologist’s knowledge of basalt in general.

The following assumptions are from the geologic report (Reclamation, 1963): 1) The proposed tunnel would be excavated through basalt from portal to portal, unless it intersects a thin interflow layer of alluvial sediments; and 2) the tunnel probably would not intersect any faults.

A general discourse on basalt was included in the geologic report and reflects what conditions the geologist thought may be encountered and is therefore repeated here. Basalt flows are divided internally into intraflow zones, which are the products of differential cooling rates; and, from place to place, the flows are separated, overlying stratum from underlying stratum, by soil profiles, or alluvial sediments. An interflow zone registers a hiatus in lava extrusion. During that hiatus, the basalt was weathered and eroded, and clastic sediments were deposited in local lake basins and along newly developed stream courses. The youthful terrain and its accompanying soil zone and sediments then were inundated when basalt once again poured across the region.

The basalt intraflow zones usually include, in ascending order:

1) a basal zone of hard, dense, extremely fine-grained, highly and irregularly fractured rock overlain by:

2) a zone of hard, dense, polygonal-columned, vertical-jointed rock referred to descriptively as the colonnade;

3) a zone of platy, horizontal jointed, wavy-columned, often vesicular rock referred to as the entablature; and

4) the uppermost zone of rapidly cooled, scoriaceous, vesicular, cindery material.

Alignment and Profile – The alignment was determined prior to this feasibility design report and no documentation on the specific reasoning for its selection was found. See Drawing No. 222-D-50220 for the plan and profile of Moody Tunnel.

Comparison to Nearby Tunnels – Bacon Tunnel (first unit) and Bacon Tunnel (second unit) are two parallel tunnels constructed in geology thought to be similar to what would be encountered by Moody Tunnel. They are located approximately 31 miles north and west of the Moody Tunnel site. The condition and structural performance of these tunnels is unknown at the time of this writing.

Construction – The length of the tunnel is 2,473 feet. It was assumed that excavation by mechanical boring with a TBM would not be economically competitive with excavation by drill and blasting for a tunnel of this short length. The perceived efficiency of mechanically boring over drill and blast excavation is offset by the required mobilization and demobilization time when the length of the tunnel is short. It was also assumed that a roadheader could not efficiently excavate competent basalt. Full-face drill and blast excavation was estimated for two different rates of advance and amounts of support.

An excavated horseshoe-shape tunnel was estimated. This shape would be appropriate in hard rock where only light side pressures are anticipated. While a circular shape could also resist lateral pressures, the flatter floor of the horseshoe shape facilitates the contractor’s travel within the tunnel. Excavation quantities were based on B-line dimensions (see Figure 2- 9). The excavated (B-line) diameter was 15.5 feet. The A-line to B-line dimension is 16 inches (Reclamation, 1994). The A-line to B-line dimension allows an estimate of excavation outside of the A-line needed to place the initial support and includes an allowance for overbreak. There would be no negative impact from this size of the tunnel impeding excavation progress.

The average advance rate for full-face excavation of Moody Tunnel was assumed to be 12 and 24 feet per day for the most probable and low estimate, respectively. The Reclamation tunnel database indicates that for drill and blast tunnels with excavated diameters ranging from 14.5 feet to 16.33 feet (23 tunnels total) the lowest advance rate was 12.00 feet per day and the highest advance rate was 35.4 feet per day. The average advance rate was 20.3 feet per day. For comparison, the SME Mining Engineering Handbook (SME, 1973) estimates the average advance rate for a medium tunnel (a face area from 100 ft^2 to 250 ft2) is 35.5 feet per day in supported sections and 64.2 feet per day in unsupported sections of tunnel.

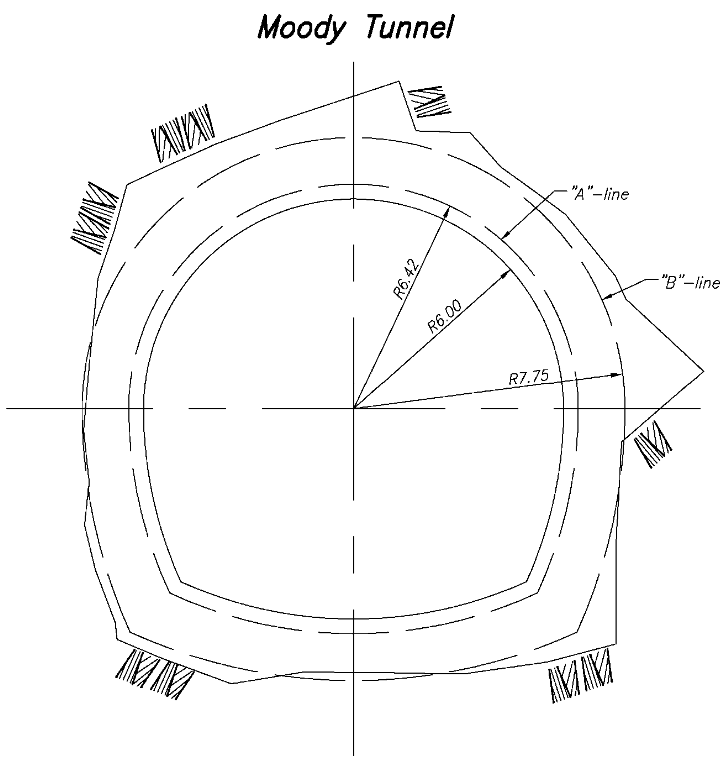

Figure 2- 9. Typical cross-section through Moody Tunnel

It was assumed that groundwater infiltration into the tunnel during construction would be negligible. It was assumed that large inflows from point sources may occur but would bleed-off and be short lived.

2.2.13.3.1 ***Tunnel Design***

General – Access roads, contractor staging areas, tunnel muck disposal sites, a wastewater treatment site, and open-cut portal excavations were not evaluated. They were assumed unlisted items. From the general geology, it was assumed that Moody Tunnel would be classified non-gassy.

Hydraulics – The required flow to supply the peak agricultural needs of this portion of the project was estimated to be 793 ft^3/sec. Additionally, surface runoff from storms will periodically enter the canal. The design flow was defined to be the combined flow to supply the peak agricultural needs and the flow from the surface runoff entering the canal. The design flow was estimated to be 862 ft^3/sec. The tunnel was sized as a free-flow tunnel. The Manning equation was used for hydraulic calculations. A rough (wood formed with an eroded invert) concrete lining surface with a Manning's n of 0.014 was assumed for hydraulic calculations. The free-flow depth should be approximately 0.82 times the internal diameter of the tunnel. The velocity would be 8.0 feet per second.

Ground stabilization – Prior to the start of tunnel excavation, it is assumed that horizontal rock reinforcement would be installed around the portal openings. No other pre-tunneling ground stabilization is anticipated.

Initial Support – Bacon Tunnel was constructed between 1946 and 1950 and was unsupported for 90 percent of its length and was supported by steel sets for 10 percent of its length. Bacon Tunnel (second unit) was constructed from 1976 to 1978 by the top heading method. The top heading was supported for 100 percent of the tunnel length by steel sets with auxiliary rock bolts to support the wall plate ledge. Some shotcrete was also used. Reclamation's tunnel database records that: 1) soil-like zones within hard rock needed additional supports; and 2) a wall plate ledge slid into the tunnel, failing 118 feet of rib-supported tunnel. The two tunnels are only 400 feet apart. It is not understood why the two tunnels had such different support requirements. It can only be speculated that the Bacon Tunnel (second unit) contractor thought that supplying possibly unneeded support would allow a more rapid, but safe, advance of the tunnel and that completing the tunneling in a shorter time would offset the increased cost of the additional support. Consequently, two estimates were generated which reflected the difference in support quantities. One estimate assumed 10 percent of the length of the tunnel would be supported with structural steel sets and the other estimate assumed that 100 percent of the length of the tunnel would be supported with structural steel sets and rockbolts. Initial support consisting mostly of rock reinforcement should be investigated in final design.

The weight of steel supports (pounds per linear foot of tunnel) installed in Bacon Tunnel was used to back-calculate (Reclamation, 1967) that the rock encountered would be classified as moderately blocky and seamy, while the weight of steel supports installed in Bacon Tunnel (second unit) indicated very blocky and seamy rock. Although the difference in rock conditions seems somewhat incongruous for tunnels only 400 feet apart, the estimated steel support sizes for Moody Tunnel, modified for the smaller tunnel diameter, were based on these assumptions.

Lining – Joints in the rock were assumed to be open to wide (Reclamation, 1998). Regardless of the need for final support, a concrete lining was deemed necessary to prevent egress (loss) of the conveyed water from the tunnel into the surrounding rock.

It is assumed that any water wells are sufficiently far removed from the tunnel that a drop in the water surface above the tunnel will not result in a lower groundwater surface at any wells. It is assumed that there will be no contamination of the conveyed water by water from the rock encompassing the tunnel. Hence, a watertight lining is not required and use of weep holes to minimize external hydrostatic pressure would be acceptable.

A finished horseshoe-shape tunnel was estimated with a diameter of 12.0 feet (see Figure 2- 9). This shape would be appropriate for the following conditions: 1) a tunnel excavated by drill and blast; and 2) for a cast-in-place unreinforced lining where lateral rock loadings are anticipated; and 3) for a cast-in-place unreinforced concrete lining where some external hydrostatic pressure on the lining is anticipated. The thickness of the lining is 5 inches (Reclamation, 1994).

Grouting – Following placement of the lining, backfill grouting will be required to fill any voids and ensure contact of the lining with the rock in the crown of the tunnel.

Water Control – Weep holes cast or drilled through the tunnel lining will allow drainage of groundwater into the tunnel and minimize the external hydrostatic pressure against the lining. Weep holes should be placed above normal depth of the design flow if practical.

2.2.14 **Bridges and Relocated Roads**

2.2.14.1. General

A bridge is a structure, including supports, erected over a depression or an obstruction, such as a water barrier, highway, or railway, and having a track or passageway for carrying traffic or other moving loads, and also having an opening measured along the center of the passageway of more than 20 feet between undercopings of abutments or spring lines of arches, or extreme ends of openings for multiple boxes. A bridge may also include multiple pipes, where the clear distance between openings is less than half of the smaller contiguous opening [from the American Association of Highway Transportation Officials (AASHTO) glossary].

A bridge should be classified according to its use or the type of facility it crosses:

- Operational structures such as spillways
- Farm access and local access roads
- County roads
- State or interstate highways
- Railroads
- Pedestrian walkways (covered in separate AASHTO guide specifications)

2.2.14.2. Feasibility Design Criteria

Fifty-nine locations have been identified that will require bridges or road relocations upon completion of the East High Canal. Existing county roads will require bridges to cross the new canal. Roadway relocations are a cost-effective alternative to constructing bridges. A total of 7.15 miles of roadway is planned to address 29 of the 59 locations. In general, the roadway relocations will parallel the canal alignment. In a few cases, the road relocation will parallel the old road to avoid a series of road/canal crossings. Roadway widths and surfacing will match existing conditions. Paving surfaces are composed of 4 inches of aggregate base course and 3 inches of asphalt. Striping will also be replaced in kind. Gravel surfacing is placed at a depth of 6 inches. The quantities for bridges required in this study include 30 bridges and 7.15 miles of roadway relocations.

- Bridge information
    - Name of feature being crossed – East High Canal
    - See Appendix D for detailed list of bridges, locations, and other pertinent data.
        - In accordance with the Reclamation Manual FAC 07-01 (*Bridge Inventory and Inspection Program*), a Type 1 bridge is any Reclamation-owned bridge that is located on a publicly or privately maintained road open to public travel. All bridges in this estimate are considered to be Type 1 bridges. During final design, a determination will have to be made as to whether they would be transferred to the local county entity or retained by the Government.
- Bridge Design Summary
    - The following sections of canal are included in the East High Canal estimate and Appendix D:
        - East High Canal 1 (EHC 1)
        - East High Canal 2 Reach 1 (EHC 2-1)
        - East High Canal 2 Reach 2 (ECH 2-2)
        - Black Rock Branch Canal Reach 1 (BRC 1)
        - Black Rock Branch Canal Reach 2 (BRC 2).
    - Aerial photography was reviewed to determine where the proposed canal crosses the existing roadways. Driving surface (gravel or paved) and roadway width (1- or 2-lane) were also determined by the photos.
    - The width and invert of the proposed canal section was used to determine the bridge span lengths and abutment height/elevations. The length of the approach roads and earthwork (cut/fill) was calculated based on a 2-percent to 5-percent slope from the top of abutment to existing ground.

- Earthwork quantities underneath the bridge superstructure are included in the overall canal excavation quantity and are not part of this section. For excavation, 25 percent is assumed as rock and 75 percent as common.
- Guardrail on the bridge consists of a 1.5-foot-wide concrete Jersey shape barrier. On the approaches, guardrail details are taken from Washington State Department of Transportation (WSDOT) standard plans. These WSDOT details include the length, railing type, post size/spacing, flare angle, and terminal section. Four corners of the bridge require metal W-beam guardrail transitions with a slight flare and end terminal sections.
- Bridges are designed in accordance to AASHTO Load and Resistance Factor Design (LRFD) standards. Girder selection ranges from AASHTO Type I through Type III standard beams. Bridges will accommodate the HL-93 live load. Single span and 8-inch cast-in-place decks are assumed for all bridges. Cast-in-place concrete abutments with spread footing versus piles are considered sufficient since shallow bedrock is assumed.
- At final design, a utility survey will be required to identify any utilities such as power lines, telephone lines, and water lines that may require installation on the bridge superstructure.

2.2.15 Field Drainage Systems

Extensive feasibility-level investigations, designs, and cost estimating of drainage facilities for the Columbia Basin Project were conducted in the 1960s (Reclamation, 1966a) and 1970s (Reclamation, 1972b; 1976) by Reclamation's Pacific Northwest Region. Based on the data available at that time, significant drainage facilities and associated costs were determined to be necessary for the irrigated lands serviced by the East High and East Low Canals. Because of budget and schedule constraints placed on the Odessa Subarea Special Study, it was decided to use the drainage information and costs presented in these earlier studies modified for information developed from over 35 years of irrigation. The following procedures were used to develop the cost estimates for the drainage facilities presented in this report:

1. Costs contained in the *Supporting Data for Development of Costs for Initial Plan Formulation Studies* Report (Reclamation, 1972b) were reviewed and revised to remove unlisted items (design contingencies), rights-of-way (ROW) costs, construction contingencies, and indirect (noncontract) costs, as these costs will be added separately to the estimates.

2. A weighted average cost of drainage per acre for the eight drainage areas that cover the lands to be irrigated by Alternatives 2 and 3 was computed and the January 1970 costs were indexed to January 1972 costs using the index factor (1.0294) recommended by the November 1972 Report (Reclamation, 1972b).

3. The January 1972 cost was indexed to October 2009 price levels using the general construction cost index factor (5.31). (Note: Reclamation cost guidelines do not

support the use of cost indexes to reflect a current price level which is more than 5 years from the original estimate.)

4. The indexed cost per acre was multiplied by number of acres to be irrigated within each of the eight drainage areas under Alternatives 2 and 3.

5. For the Monte Carlo analysis of costs, no drainage facilities were assumed to be required for the most probable low (MPL) field cost estimates for Alternatives 2 and 3. For the most probable (MP) field cost estimate, a percentage of the drainage requirement estimated in the 1966-1972 studies was used to account for present day conditions. For Alternative 2, this percentage was estimated to be 33 percent, and for Alternative 3, this percentage was estimated to be 50 percent. The reduced level for Alternative 2 was skewed towards no facilities because no new open carriage facilities would be constructed. The reduced level for Alternative 3 was assumed neutral because of the construction of a new major canal. For the most probable high (MPH) field cost estimates, the full drainage requirement estimated in the 1966-1972 studies was assumed to be required for both Alternatives 2 and 3.

6. To account for the fact that drainage facilities would not be constructed at the same time as the initial carriage and distribution systems, costs for drain construction were delayed 5 years and distributed across 15 years in accordance with the cost distribution curve shown on page 34 of the November 1972 Report (Reclamation, 1972b).

The following factors were considered in the determination of the appropriate percentages to be used to modify the 1966-1972 indexed costs for the MP field-cost estimates.

Factors for using a lower percentage of the 1966-1972 drainage requirements:

- Reclamation has provided drainage for about 119,000 acres of 670,000 acres of irrigated project land, or 18 percent. All of these lands are west of the ELC. The bulk of these drains were constructed between 1960 and 1978 and the last drains were constructed in 1994.

- Thirty-five years of irrigation have occurred in the Subarea without significant drainage issues and no drains have been constructed east of the ELC. Based on the 1966-1972 studies, drainage facilities should have been constructed east of the ELC within 20 years of operation.

- Farm units west of ELC have a mixture of rill (gravity), handline, wheeline and drip but predominately center pivot. Legislation mandates sprinkler irrigation for the farm units east of ELC which is more efficient than gravity irrigation. Drainage projections in the 1966 Drainage Appendix (Reclamation, 1966a) assumed rill (gravity) irrigation and production of high-water-use crops.

- Farm units west of ELC were laid out for gravity irrigation which is denser than the farm units east of ELC. The dispersed farming east of ELC will reduce the concentration of irrigation water and lessen the potential for subsurface saturation. Center pivot irrigation also leaves nonirrigated corners.

- Subarea soils east of ELC tend to be deeper and less rocky than some of the developed block acreage west of ELC which should reduce soil saturation.

- The main carriage system (ELC) is existing and any seepage or bank storage has already taken place.

- The proposed irrigation distribution system from the ELC is piped and will not contribute to need for drainage. The same drainage costs in the November 1972 Report (Reclamation, 1972b) were used for both open lateral and pressure pipe delivery plans, which are conservative.

- Annual average carriage in the ELC is 1,287,505 acre-feet. An additional 174,000 acre-feet (14 percent) for Alternative 2 would not significantly increase seepage rate or saturate additional acreage.

- There is no evidence of drainage limitations in the Odessa Subarea under current conditions (CH2M Hill, 2010).

Factors for using a higher percentage of the 1966-1972 drainage requirements:

- Drainage costs in the November 1972 Report (Reclamation, 1972b) are based on the 1966 *Columbia Basin Project, Completion of Irrigation Facilities, Feasibility Grade Study, Appendix Volume III, Drainage* (Reclamation, 1966a). This study prepared detailed drain layouts and cost estimates for a sample area that included areas where drainage was not deemed necessary. Drainage costs for these sample areas were calculated and then divided by total irrigable acres in the sample area to come up with a drainage cost per irrigable acre; i.e., reduction for lands not requiring drainage has already been accounted for in the November 1972 Report.

- The 1966 Drainage Appendix (Reclamation, 1966a) based drainage needs on topography and soil conditions in sample areas representative of the eight drainage areas.

- Actual historical irrigation application rate is unknown but is believed to be less than 3 feet/acre.

- Where four pivots meet, some farmers have been placing a small pivot to irrigate this interior area.

- Natural subsurface drainage could be a limitation if increased water supply reliability encouraged more widespread irrigation and irrigation of more water intensive crops (CH2M Hill, 2010).

As stated in the Odessa DEIS, the estimated costs are based on 20- to 30-year-old CBP design assumptions, which included new irrigation development, and were based on platted, concentrated farms using gravity flow and rill irrigation. These assumptions are no longer valid, because the current farms in the Study Area are spaced widely and use pressurized delivery systems. Although project design has not progressed to the point of addressing irrigation water drainage in detail, estimates of drainage system costs using the original CBP assumptions are included to ensure complete and conservative cost estimates. The proposed action alternatives being considered in this report would simply replace current groundwater with surface water. No new land would be irrigated, and field application would not exceed historical water use. Further, under current conditions, no significant drainage issues or problems are evident in the Study Area. Given these factors, no substantial change in irrigation water drainage conditions is anticipated. Final designs and cost estimates will be updated as appropriate.

2.2.16 **Operation and Maintenance Facilities**

Operation and maintenance facilities were initially designed and estimated for north and south of I-90. Upon further review, the facilities were eliminated as a cost-savings measure. However, operation and maintenance facilities were evaluated in the Odessa DEIS to account for environmental impacts.

2.2.17 **Wildlife Enhancements**

At the request of the Washington State Department of Fish and Wildlife (WDFW), Reclamation agreed to incorporate wildlife enhancements into the feasibility design of all new canals to improve the ability of wildlife to travel from one side of the new canal to the other side. Features specifically incorporated into the feasibility design are wildlife crossing bridges and canal under passages.

2.2.17.1. Wildlife Crossing Bridges

A total of eleven wildlife crossing bridges were included in the feasibility design. State officials provided locations for these bridges based on surveys completed by them. Each bridge would be covered with a thick layer of soil which would be planted with native grasses and small bushes. Small boulders would be randomly placed on the bridge to provide cover for smaller animals. These bridges would also serve as O&M bridges so one side of the bridge would be left clear of boulders so that vehicles would be able to drive across the bridge. Refer to Drawing No. 222-D-50118.

2.2.17.2. Ramps

For animals that accidentally fall into the canal, the feasibility designs include escape ramps constructed into the sides of the proposed canals. The design criteria for the ramps are summarized below:

- A ramp will be located 500 feet upstream of all intakes for siphons, tunnels, and check structures. Intakes for turnouts and pumping plants do not require a ramp 500 feet upstream of the structure because these intakes are protected by trashracks, which would prevent animals from being drawn into the intake.
- For concrete-lined canal sections or sections constructed through rock, ramps would be constructed every mile into the side of the canal. Ramps would alternate between each side of the canal.
- For canals constructed through soil, ramps are not required. It is assumed that the flatter slopes of these sections of canals, plus the fact that the slopes are soil, animals would have adequate footing to make their escape from the canal.

Ramps will be constructed with a 4:1 slope and would be surfaced with unreinforced concrete. The concrete surface would be roughened to provide a nonslip surface for adequate footing. A visual and audible barrier would be suspended across the canal immediately downstream of the ramp and angled upstream so that the animals would be directed to the ramp. Refer to Drawing No. 222-D-50119.

2.2.17.3. Wildlife Underpasses

For this feasibility study, it was assumed that the cross drainages designed for draining up-gradient lands would serve the dual purpose of wildlife under passages. It was also assumed that smaller cross drainages would be enlarged if needed to meet State design criteria. In addition, the inlets and outlets to these cross drainages would be shaped per State criteria to safely guide wildlife into and out of the passages.

Chapter 3: Black Rock Coulee Re-Regulation Facility

The proposed Black Rock Coulee Re-Regulation Facility is an off-stream storage reservoir capable of storing 4,819 acre-feet of Columbia River water for use during the irrigation season. The reservoir would be formed by an embankment dam constructed across Black Rock Coulee approximately 9 miles southeast of the town of Wilson Creek, Washington. The facility is comprised of a zoned earthfill embankment dam, a gated spillway, a low-level reservoir outlet works, a reservoir inlet check structure, a reservoir outlet check structure, a pumping plant and switchyard, and a discharge pipeline. Data describing facility types, sizes, and capacities are shown in Table 3- 1.

For the location plan of this facility, refer to Drawing No. 222-D-50061.

Table 3- 1. Black Rock Coulee Re-Regulation facility data

Facility/Characteristic	Size/Quantity
Reservoir	
Active storage capacity @ El. 1463.3	4,819 acre-feet
Maximum water surface elevation	1 ,464 feet (Probable Maximum Flood)
100-year flood water surface elevation	1 463.3 feet
Top of active storage water surface elevation	1 463.3 feet
Minimum water surface elevation	1 409.6 feet
Dam	
Type	Zoned earthfill embankment
Crest elevation	1 470.0 feet
Crest width	24 feet
Crest length	2,455 feet
Low level outlet works flow capacity	110 ft^3/s at top of active storage El. 1463.3
Pumping Plant	
Pumping unit type Pumping units 1 and 12 flow capacity Pumping units 2 and 11 flow capacity Pumping units 3 thru 10 flow capacity Number of pumping units	Vertical turbine 11.14 ft^3/s each (includes 3% wear factor) 22.28 ft^3/s each (includes 3% wear factor) 44.57 ft^3/s each (includes 3% wear factor) 12
Plant flow capacity (rated)	423.40 ft^3/s (includes 3% wear factor)
Pumping unit lift (rated)	204 feet @ 190,000 gpm (includes 3% wear factor)

3.1. Black Rock Coulee Dike

3.1.1 Engineering Geology

3.1.1.1. Regional Geology

The Black Rock Coulee Dike site is located in the Palouse Subprovince near the northern edge of the Columbia Plateau Physiographic Province in central Washington. Relatively flat surfaces have been locally deeply incised by scouring during Pleistocene flooding from glacial Lake Missoula. The channels, some of which host the present-day drainages, trend southerly and westerly into the Snake and Columbia Rivers. The flood channels in this area, known as channeled scablands, are the dominate landforms in the otherwise generally flat terrain (Reclamation, 1966b; 2008b).

Bedrock consists chiefly of Miocene age, tholeitic basalt of the Columbia River Basalt Group (CRBG) except at the northern margin of the Columbia Plateau. The basalts attain an aggregate thickness of over 15,000 feet in the central portion of the basin. The CRBG consists of the Imnaha Basalt Formation (oldest), the Grande Ronde Basalt Formation, the Wanapum Basalt Formation, and the Saddle Mountains Basalt Formation (youngest). These formations interline locally with sedimentary members of the Ellensburg Formation, although no sedimentary units are mapped in the vicinity of the dike site. Each of these formations is composed of one or more separate flows. Basalts mapped in the vicinity of Black Rock Coulee Dike are chiefly flow units of the Wanapum Basalt (Reclamation, 1966b; 2008b).

The basaltic bedrock in the Palouse Subprovince is overlain by Quaternary deposits of loess in tracts of land that have not been scoured during the Pleistocene flooding. Within the channeled scabland tracts, the basaltic bedrock is overlain by flood deposits varying in gradation from fluviolacustrine clays, silts, and sands, to gravel, cobble, and boulder deposits (Reclamation, 1966b; 2008b).

3.1.1.2. Site Geology

The dike site is about 17 miles northeast of Moses Lake, in Grant County, Washington. The dike is located across Black Rock Coulee, which trends in a southwesterly direction. The coulee floor is occupied by an intermittent creek that connects a series of six relatively small lakes. These lakes appear to have formed behind resistant basalt ledges dipping southward. The dike is located just downstream of one of these lakes, on one of the basalt ledges. At the dike site, basalt bedrock is very shallow or exposed and is thinly veneered by silt, sand, or basalt detritus derived locally. Twelve power auger holes were attempted along the dam axis in 1960. Four of these holes were deleted because the basalt was outcropping at the surface at their location. Only three of the remaining eight holes encountered more than 18 inches of soil. Auger hole AP 2 near the left end of the dike encountered 3 feet of silt; AP 3, also on the left end of the dike, encountered fine to coarse grained sand to a depth of 5 feet; and AP 9, near the bottom of the right abutment, penetrated silt to a depth of 2.5 feet and then basaltic gravel (slopewash and in-place detritus) to a depth of 4 feet. The remaining five holes had only 4 to 18 inches of silt, sand or gravel overlying the basalt bedrock (Reclamation, 1966b; 2008b).

The basalt is the Wanapum Basalt Formation, which is divided into the Priest Rapids (Tp), Roza (Trz), and Frenchman Springs (Tf) Members. The Frenchman Springs (Tf) Member was encountered in exploration drill holes in the area of Black Rock Coulee Dike site. The Frenchman Springs Basalt (Tf) forms the foundation at Black Rock Coulee Dike. It is black to gray, fine grained, very hard, and dense to slightly vesicular. Drill hole exploration at a sight about 1/3-mile upstream indicates that the rock is from moderately to slightly fractured. Pressure permeability tests, performed at the upstream site, indicate this bedrock unit has low hydraulic conductivity. The upper few feet of the basalt is generally moderately to highly weathered. The unit consists of four flows and is about 200 feet thick in the northwest part of the study area. Its thickness at the dike site is not known (Reclamation, 1966b; 2008b).

3.1.1.3. Seismicity

The dike is located in the Odessa subarea in the northeast Columbia Plateau. The site is a relatively stable area dominated by generally underformed Tertiary rocks and Quaternary sediments. Seismic loadings for the seismic study included earthquakes associated with blind faults and folds of the Yakima Fold Belt (YFB) as well as background or random events. The YFB is believed capable of producing large magnitude earthquakes (moment magnitude [M] > 6.5). The controlling event at peak horizontal ground acceleration (PGA) and 0.2 sec spectral acceleration (SA) is a moderate-size random earthquake, M 5.8, at about 15± 2 km. At longer return periods, the larger events begin to control both the short- and long-period hazard. In this case, an M 6.9 event at 40 km distance may be possible. At a return period of 10,000 years, the PGA would be about 0.29 g and at a return period of 50,000 years, the PGA would be about 0.45 g (Reclamation, 2008b; 2009b).

3.1.2 Diversion and Care of Water

The project is located in a semiarid zone and the Black Rock Coulee has only intermittent flow. The damsite will be dry most of the time. Diversion and care of water should be a simple matter. It may be possible for the contractor to just build the dam to a required height during the drier season of the year to provide adequate downstream protection and to protect the worksite from flooding. At the most, diversion of water would only require a gap to be left at one abutment that can be closed quickly with embankment fill after the flood season has ended. The outlet works construction may require a small cofferdam to protect it from flooding.

3.1.3 **Foundation Excavation and Treatment**

3.1.3.1. Foundation Excavation

Foundation excavation at Black Rock Coulee Dike will be fairly simple. The foundation will be stripped of loose rock and soil materials to the top of in-place rock beneath the entire embankment. Beneath the impervious zone and chimney drain, the rock will be excavated to sound basalt bedrock, which is estimated to be between 2 and 4 feet deep. The Most Probable Low cost option (low option), the Most Probable cost option (probable option), and the Most Probable High cost option (high option) for the dike would have 2, 3, and 4 feet of excavation, respectively, beneath the impervious zone and chimney drain. Outside of the impervious zone and chimney drain, the rock surface will only require removal of any loose surface soil and rock.

3.1.3.2. Dewatering

Dewatering will be relatively easy at this site because of the rock foundation and shallow excavation. The water table is likely below any required excavation. If any dewatering is required, it will likely be in the form of a few sump pumps with collection ditches leading to the sumps.

3.1.3.3. Foundation Treatment

3.1.3.3.1 ***Foundation Grouting***

Based on preliminary investigations, a grout curtain may not be necessary at Black Rock Coulee Dike because the basalt appears to be a fairly impermeable formation and the height of the dike is comparatively small. Maximum hydraulic head will be about 50 feet. For this design, it has been assumed that the probable option will require a single line grout curtain beneath the dike abutments only. The low option would have no grout curtain and the high option would have a grout curtain for the entire length of the dike. The grout curtain would be 60 feet deep and the grout holes would be on 10-foot centers with provisions to split space holes in more permeable zones until grouting closure is obtained. The grout holes should be drilled at an angle that will intercept an optimal number of vertical fractures.

3.1.3.3.2 ***Foundation Cleanup***

After the foundation grouting has been completed, the rock surface beneath the impervious zone will have to be meticulously cleaned by hand tools and the use of high pressure air and water, mixed. All dirt, debris, and loose rock should be removed so that the surface grouting, dental concrete, and impervious fill can be placed on a clean, sound rock surface. A second cleanup of the impervious fill foundation will likely be required just prior to fill placement. Outside the impervious foundation, the foundation surface only needs to be cleaned of loose rock, soil and debris.

3.1.3.3.3 ***Surface Grouting (slush grouting)***

After the first foundation cleanup, the entire rock surface beneath the impervious foundation should be examined for cracks and fractures wider than 1/8-inch. These should be treated by troweling a sand-cement grout into the fractures.

3.1.3.3.4 ***Dental Concrete***

The basalt will likely form a fairly irregular and rough surface upon excavation. Treatment with dental concrete will probably be necessary in some areas. For this design, it is assumed that the low option will require treatment by a 2-foot thickness of dental concrete over 10 percent of the area of the impervious foundation. The most probable option would require this treatment over 20 percent of the area of the impervious foundation and the high option would require this treatment over 30 percent of the area of the impervious foundation.

3.1.4 **Embankment Design and Construction**

3.1.4.1. Dam Embankment

The embankment will be constructed of earthfill. The crest of the dam will be at elevation 1,468.0 feet for the low option, 1,470.0 feet for the probable option, and 1,472 feet for the high option. The central portion of the embankment (the impervious core) will be constructed of impervious soil called Zone 1. The top of the core will be at elevation 1,465.0 for all three options and will be 8, 10, or 12 feet wide for the low, probable, and high option, respectively. The side slopes of the core will be 2 vertical on 1 horizontal for all three options. The impervious core will have a chimney drain constructed of processed sand and gravel called Zone 2 on its downstream face that extends via a drainage blanket, over the basalt foundation, to the downstream toe of the dam. The Zone 2 chimney and drainage blanket will be drained by a perforated high density polyethylene (HDPE) toe drain pipe surrounded by a processed gravel drain material called Zone 4. The width of the chimney will be 2, 4, or 6 feet wide for the low, probable, and high options, respectively. The thickness of the drainage blanket will be 2, 3, and 4 feet thick for the low, probable, and high options, respectively. The purpose of the Zone 2 is to provide for collection of seepage through the foundation and Zone 1, and to protect against internal erosion of the silty Zone 1, both for static and earthquake loading conditions. It will be capable of performing this function for reservoir loading conditions up to the maximum water surface and above, if necessary. The core and chimney drain will be buttressed up- and downstream by Zone 3 embankment. The Zone 3 will be mixtures of sand, silt, and gravel from required excavation or borrow. The Zone 3 will extend from the top of the Zone 1 core to the top of the dam and form the crest of the embankment. The crest width will be 20, 24, or 28 feet, depending on the cost option of low, probable, or high, respectively. The upstream slope of the embankment will vary, depending on the option. The low option will be 1 vertical on 2.5 horizontal from crest to rock surface; the high option, 1 vertical on 3 horizontal from crest to rock surface; and the probable option will break at elevation 1,450.0 feet from

1 vertical on 2.5 horizontal to 1 vertical on 3 horizontal to the rock surface. The downstream slope will be 1 vertical on 2 horizontal from crest to rock surface for the low option; 1 vertical on 2.5 horizontal from crest to rock surface for the high option; and 1 vertical on 2 horizontal to elevation 1,450 feet; and then 1 vertical on 2.5 horizontal to the rock surface for the probable option.

3.1.4.2. Slope Protection

The upstream slope will be protected by 2 feet of riprap on 1 foot of gravel bedding material above elevation 1,450.0 feet for all three options. The downstream slope will be protected by 1 foot of cobbles on 6 inches of gravel bedding material from the crest to the rock surface for all three options. Sound durable rock for riprap and cobbles is readily available near the damsite; however, it may be more cost effective to purchase the material from an existing quarry rather than open up a specific quarry. In either case, the riprap should be available in the near vicinity of the damsite. Gravel bedding may have to come from a commercial source. It should be clean, well graded sand and gravel material. There is no known borrow source for this type of material near the dike site.

3.1.4.3. Embankment Materials

3.1.4.3.1 ***General***

There has been no specific investigation for construction materials for the embankment. However, based on reconnaissance and exploration at this damsite and other nearby structures, it appears that there is an abundance of silt and sandy silt, as well as some sand and gravel material nearby. It is obvious that there is abundant basalt rock material nearby. However, for this feasibility design, rockfill is not considered for use in the embankment because the economy of opening a rock quarry for this small dike is questionable. The feasibility and economy of rockfill should be investigated during final design of the dam.

3.1.4.3.2 ***Zone 1***

Zone 1 will be silts, silty sands, and sandy silts from nearby borrow sources. It will be required to have at least 30 percent, by weight, of the material passing a standard No. 200 sieve. These materials are abundant in the area and should be available within ½-mile of the dike. The fill will be placed in 9-inch loose lifts, near optimum water content, and compacted by eight passes of a tamping type roller to a dry density of 95 to 100 percent of standard dry density. Prior to compaction the loose lift will be disked by a 36-inch disk plow to break up clods, mix the soil, distribute moisture, and scarify the surface of the previous lift. Pre-wetting of this material will be required in the borrow area to bring the soil to the required moisture content. On the embankment, sprinkling of the fill with water will likely be necessary to maintain proper soil moisture during placing, disking, and compacting.

3.1.4.3.3 *Zone 2*

Zone 2 will be both a filter to prevent internal erosion of the Zone 1 and a drain that will carry any seepage water out of the embankment and into the toe drain. It will be a very clean medium-to-coarse sand and fine gravel material. The actual gradation will be a designed gradation based on Reclamation filter criteria. It does not appear that material from which Zone 2 can be manufactured is readily available from nearby borrow. It will likely have to be purchased from a commercial source. The cities of Ephrata or Moses Lake are the most likely sources of this type material.

The zone 2 material will be placed in 12-inch loose lifts and compacted by 2 to 4 passes of a vibratory roller to about 75% to 80% relative density. It is likely that the vibrator will be turned off for the last two passes. Moisture content will be to the degree that will facilitate compaction.

3.1.4.3.4 *Zone 3*

Zone 3 will be silts, sands, and gravels, or mixtures thereof, from nearby borrow. It is anticipated that it will be available within ½-mile of the dam. It will be placed in 12-inch-thick lifts, at moisture content between optimum and 3-percent dry of optimum, and compacted to a minimum of 95 percent standard dry density by an appropriate roller. A minimum of 4 passes of the roller will be required.

3.1.4.3.5 *Zone 4*

Zone 4 materials should be designed in accordance with Reclamation filter criteria. It will be a very clean, graded, coarse-sand-to-gravel. It will likely have to be purchased from a commercial source such as at Ephrata or Moses Lake.

3.1.4.4. Instrumentation

This is a small dike with an excellent foundation. No special instrumentation will be required. Only three to four lines of surface points to monitor settlement of the embankment seepage weirs at the toe drain exits, and possibly some porous tube piezometers at the maximum section, are required.

3.1.5 **Future Considerations**

3.1.5.1. Exploration

3.1.5.1.1 *Foundation*

There has been very little exploration for Black Rock Coulee Dike**.** Twelve power auger holes were attempted along the dike alignment, but only 8 were completed because basalt outcrops at, or very near, the surface. None of the auger holes penetrated the basalt. For final design, 3 to 4 drill holes that penetrate the basalt 50 to 100 feet should be drilled along the alignment of the dam so that the integrity of the basalt foundation can be better assessed. At least one of these holes should be angled to intersect maximum vertical rock fractures and water tested to determine permeability so that a more informed decision can be made about the need for grouting.

3.1.5.1.2 *Borrow Material*

The only information about the available borrow materials is based on reconnaissance and three holes that were drilled on an upstream, alternative alignment. It appears that most available materials are silt to sandy silts. There does not appear to be any sand and gravel, suitable for filter and drain material, available in the vicinity of the dike. The available materials are suitable for general embankment construction, but volume and location need to be verified by borrow area investigation.

3.1.5.1.3 *Rockfill*

Basalt rock near the ground surface is abundant in the area. This material is suitable for slope protection or rockfill. The economics of constructing a rockfill dike versus an earthfill dike should be investigated. It was not considered during this feasibility design because of the relatively small volume of the dike. It was thought that, for this small volume, earthfill would be more economical. This should be verified.

3.1.5.1.4 *Laboratory testing*

No laboratory analysis of available material has been conducted. The embankment materials should be tested for shear strength, consolidation characteristics, and permeability. These tests should be conducted on samples that have been prepared to imitate compaction and moisture of in-place embankment. Gradation and permeability testing should be conducted on material available for filter and drain material, whether it be from borrow or commercial source.

3.1.5.1.5 *Analyses*

No analyses have been conducted for the dike. During final design, the usual set of analyses to ensure embankment stability and fine-tune the embankment geometry, assess consolidation and any necessary camber, and to predict seepage and verify adequate protection against internal erosion should be conducted.

3.1.6 Gated Spillway

The proposed Gated Spillway (refer to Drawing No. 222-D-50104) is capable of making releases to the existing Black Rock Coulee. The spillway would be constructed through the left abutment of the dike embankment. The spillway is a two-bay reinforced concrete structure that houses two 20-foot-wide by 14-foot-tall radial gates, each capable of passing 3,610 ft^3/s each, for a total capacity of 7,220 ft^3/s. The structure is approximately 55 feet wide and 102 feet long (including upstream entrance and downstream outlet slabs). Each bay is 20 feet wide. The approach and exit slabs are set at about elevation 1,446.00 feet. The top of the gate hoist bridge is set at elevation 1,482.50 feet and the top of the spillway bridge is set at elevation 1,472.50 feet. The top of the spillway crest is set at elevation 1,451.50. The unlined inlet and outlet channels are excavated rock present at the site.

Electrical systems would be provided to supply power for operation of the equipment and lighting as needed. SCADA equipment would be provided to connect this facility to the overall project monitoring and control system located in Ephrata, Washington.

3.1.7 **Reservoir Low-Level Outlet Works**

The proposed Reservoir Low-Level Outlet Works (refer to Drawing No. 222-D-50098, 222-D-50099, and 222-D-50102) is capable of making releases to the existing Black Rock Coulee. The outlet works would be constructed through the dike embankment near the center of the coulee. Features of the outlet works include:

- Reinforced concrete, trashrack-protected, bellmouthed intake structure,
- 36-inch-diameter steel-lined reinforced concrete upstream conduit,
- Reinforced concrete gate chamber located at the centerline of the dam that houses a 3-foot by 3-foot hydraulically operated guard gate,
- Downstream 36-inch-diameter steel pipe inside a 8-foot-diameter reinforced concrete horseshoe-shaped adit (tunnel),
- Reinforced concrete outlet control structure that houses a 2-foot, 6-inch by 2-foot, 6-inch hydraulically operated regulating gate,
- Reinforced concrete chute, and
- Reinforced concrete stilling basin.

The intake structure (222-D-50099) is an 8-foot, 4-inch square, reinforced concrete box with two sets of 2-foot, 8-inch-wide trashracks on each of the three sides and on the top of the structure. The purpose of the trashracks is to prevent debris from entering the structure and potentially plugging or damaging the outlet work's gates or conduits. Where this structure ties into the steel-lined conduit, a streamlined bellmouthed intake is formed into the concrete to minimize hydraulic losses. Immediately downstream of the bellmouthed intake, bulkhead gate seats and guides are provided to allow for inspection and maintenance of the upstream conduit and equipment. This Study does not include costs for a bulkhead gate; it was assumed that if this structure were constructed, a bulkhead gate would be procured sometime in the future for inspection and maintenance of the conduit and gates. The future bulkhead gate should be designed for the maximum differential reservoir head.

Approximately 190 feet downstream of the intake structure, a reinforced concrete gate chamber would be constructed within the dam embankment to house a 2-foot, 6-inch by

2-foot, 6-inch hydraulically-operated guard gate. The gate chamber is connected to the downstream face of the dam via an 8-foot-diameter reinforced concrete horseshoe-shaped adit (tunnel). A 36-inch-diameter exposed steel pipe would convey the flow from the guard gate within the gate chamber through the adit to the control structure that is located approximately 175 feet downstream of the gate chamber. The control structure houses a 2-foot, 6-inch by 2-foot, 6-inch hydraulically-operated regulating gate which controls the release of water from the reservoir to the existing Black Rock Coulee. The outlet works discharges into a reinforced concrete chute and stilling basin before the water is discharged into the coulee. The purpose of the chute and stilling basin is to dissipate the energy of the water as it exits the outlet works to prevent downstream damage to the coulee.

A diesel engine-generator (EG) set would be provided at the control structure as a standby (backup) power source for essential equipment in the event primary commercial power is unavailable. An automatic transfer switch would be provided with the EG set.

A ventilation system would be provided to supply fresh air to the gate chamber and access adit. A heating and ventilation system would be provided to supply fresh air to the control structure.

Electrical systems would be provided to supply power for operation of the equipment and lighting as needed. SCADA equipment would be provided to connect this facility to the overall project monitoring and control system located in Ephrata, Washington.

The capacity of the outlet works is 110 ft^3/s when the reservoir water surface is at the top of active storage which corresponds to a reservoir water surface elevation of 1,463.3 feet.

3.1.8 Reservoir Inlet Check Structure

On the north shore of the reservoir where the proposed East High Canal discharges into the reservoir, an inlet check structure would be constructed to control the canal flows as they enter the reservoir. The reinforced concrete structure was sized to have an ultimate discharge capacity of 5,652 ft^3/s.

The structure is divided into four bays. The center two bays would have 9-foot, 6-inch-wide by 25-foot-tall radial gates installed in them, each capable of discharging 499.5 ft^3/s, for a total discharge capacity of 999 ft^3/s. The two 19-foot-wide side bays would have precast concrete stoplogs installed within each bay. If the East High Canal is enlarged at some future date, these stoplogs would be removed and two additional radial gates would be installed.

The structure is approximately 130 feet wide, 289 feet long, and 26 feet deep. Each side of the structure incorporates 10-foot-wide by 26-foot-deep overflow spillways into the structure which would permit water to flow into the reservoir when the radial gates close during power outages. The overflow crest of each spillway is 100 feet long. The purpose of this feature is to protect the upstream canal from overtopping during power outages as the water already flowing in the canal continues to flow toward the reservoir. A vehicle bridge has been incorporated into the structure to permit access to both sides of the structure by O&M vehicles.

Electrical systems would be provided to supply power for operation of the equipment and lighting as needed. SCADA equipment would be provided to connect this facility to the overall project monitoring and control system located in Ephrata, Washington.

3.1.9 Reservoir Outlet Check Structure

On the south shore of the reservoir an outlet check structure would be constructed through the left abutment of the dike embankment to serve as an inlet structure for the continuation of the proposed East High Canal. The reinforced concrete structure was sized to have an ultimate discharge capacity of 3,875 ft^3/s.

The structure is divided into four bays. The center two bays would have 7-foot-wide by 30-foot tall radial gates installed in them, each capable of discharging 260 ft^3/s, for a total discharge capacity of 520 ft^3/s. The two 14-foot-wide side bays would have precast concrete stoplogs installed. If the East High Canal is enlarged at some future date, these stoplogs would be removed and two additional radial gates would be installed.

The structure is approximately 111feet wide, 256 feet long, and 30 feet deep. A vehicle bridge has been incorporated into the structure to permit access to both sides of the structure by O&M vehicles.

Electrical systems would be provided to supply power for operation of the equipment and lighting as needed. SCADA equipment would be provided to connect this facility to the overall project monitoring and control system located in Ephrata, Washington.

3.1.10 Black Rock Coulee Pumping Plant No. 1

The purpose of the proposed Black Rock Coulee Pumping Plant No. 1 is to lift water from the proposed Black Rock Coulee Re-Regulation Reservoir to the proposed Black Rock Branch Canal. The pumping plant is a wet sump plant that houses 12 vertical pumping units that range in capacity from 11.14 ft^3/s to 44.57 ft^3/s. The plant has a total rated capacity of 423.40 ft^3/s at a rated hydraulic head of 204 feet.

The pumping plant (refer to Drawing No. 222-D-50062 through 222-D-50069) would be located on the southeast shore of the reservoir. The layout of the pumping plant service yard is based on the existing site topography, the submergence requirements of the pumping units, the alignment of the steel pipe from the reservoir low-level outlet works, equipment space requirements for the pumping plant and switchyard, access into and around the pumping plant and switchyard for construction vehicles, and access into and around the pumping plant and switchyard for maintenance vehicles. The service yard was excavated into the hillside to accommodate the hydraulic needs of the pumps and to allow for proper drainage purposes. The high point of the service yard was set at elevation 1,465.5 feet.

The layout of the pumping plant is governed by the number, type, and size of the selected pumping units and equipment, the relationship between the electrical and mechanical systems, required clearances to maintain a safe work environment for the operation and maintenance personnel, and handling requirements for the various pieces of equipment during initial installation and subsequent maintenance operations. The pumping plant is separated into two distinct areas--the Unit Bay and the Service Bay. The Unit Bay is that portion of the plant that houses the main pumping units and associated manifold piping, gates, and valves. The Service Bay is located at the end of the plant and provides an area for maintenance of electrical and mechanical equipment as well as access to the plant for maintenance vehicles. An electrical equipment gallery room would run the length of the plant and would house motor control equipment, motor starters, and switchgear. At the end of the plant, space has been allocated for an office, electrical closet, janitor closet, restroom, and kitchen/breakroom.

The depth of the sump was established based on the minimum water surface elevation in the reservoir and the required pump submergence that is needed to ensure that the proper suction head is provided so that the pumps operate efficiently. Based on these design parameters, the top of the base slab of the sump was set at elevation 1,446.42 feet.

The pump deck slab of the pumping plant was established based on the reservoir water surface corresponding to the 100-year flood event, plus 3 feet of freeboard. Based on these design parameters, the top of the pump deck slab of the sump was set at elevation 1466.00 feet.

The length and width of the plant is based on the size and arrangement of the pumping units and the required clearances for operation and maintenance of the plant. The electrical equipment gallery was determined by the size and arrangement of the electrical equipment. HVAC equipment for the plant was located outside of the superstructure. The discharge manifold was located away from the rest of the plant so as to allow space for installation of flowmeters between the pumping units and the discharge manifold. The client requested that individual flowmeters be placed immediately downstream of each pump, which resulted in the plant being wider than is normal to accommodate 12 individual flowmeters and their associated clearances (10 pipe diameters upstream of the flowmeter and 5 pipe diameters downstream of the flowmeter). Normal Reclamation layout procedures call for a single flowmeter located on the discharge pipeline in a separate structure in the service yard.

The pumping plant has a reinforced concrete substructure approximately 146 feet long by 92 feet wide, and a 188-foot-long by 57-foot-wide structural steel superstructure to house the pumps and crane, with a standing seam metal roof. Handling requirements for the units controlled the building height and the 10-ton overhead bridge crane elevation. The structural steel superstructure framing supports the crane loads and distributes them to the reinforced concrete substructure.

A channel would be excavated from the pumping plant to the reservoir to insure that water can flow into the sump when the reservoir is at lower elevations. At the inlet to the plant, the sump is protected by a trashrack to guard against trash being drawn into the structure and potentially being drawn into the pump bell. The design calls for a trash rake and trash conveyor system to provide for cleaning trashracks if excessive debris builds up on the trashracks.

3.1.10.1. Pumps

Twelve vertical-turbine pumping units were selected for the pumping plant. Pumping unit data is provided in Table 3- 2.

For this Study, the design does not include an installed spare pumping unit or spare set of bowls and impellers. The design assumed pump materials are ductile or cast iron casings and bronze impellers. Water quality was assumed to be adequate for the selected pump materials.

All pumps were analyzed for run-out conditions with the assumption that only one pump would be running in this condition. The pumps were selected providing a range of operation near the best efficiency point. Any pumping configuration that required a wide total dynamic head range was selected referencing the Hydraulic Institute standard *Centrifugal and Vertical Pumps for Allowable Operating Range* (ANSI/HI, 1997) of 70 percent to 120 percent of the best efficiency point flow for pumps with specific speeds less than or equal to 4,500 (US Units).

Table 3- 2. Black Rock Coulee Pumping Plant No. 1 unit data

Type and Number of Units:	Twelve vertical turbine pumping units
Discharge Capacity @ TDH_{Max} of 199 feet	
• Units 1 and 12 (11.14 ft^3/s each)	22.28 ft^3/s (includes 3% wear factor)
• Units 2 and 11 (22.28 ft^3/s each)	44.56 ft^3/s (includes 3% wear factor)
• Units 3 thru 10 (44.56 ft^3/s each)	356.48 ft^3/s (includes 3% wear factor)
Total	**423.32 ft^3/s (includes 3% wear factor)**
Motors	
• Units 1 and 12	350 hp @ 1,800 rpm
• Units 2 and 11	700 hp @ 1,200 rpm
• Units 3 thru 10	1,251 hp @ 900 rpm
Discharge Manifold Diameter	96-inch
Guard Valve (Discharge)	
• Units 1 and 12	16-inch butterfly
• Units 2 and 11	24-inch butterfly
• Units 3 thru 10	30-inch butterfly
Check Valve (Discharge)	
• Units 1 and 12	16-inch tilting disc
• Units 2 and 11	24-inch tilting disc
• Units 3 thru 10	30-inch tilting disc

3.1.10.2. Steel Piping and Valves

Steel pipe and valves were selected for the discharge manifold and discharge pipeline. The individual steel pipe branches and manifold are sized to limit the flow velocity and minimize friction loss.

Downstream of each pump, the individual pump discharge pipes connect into a single 96-inch-diameter steel discharge manifold. The 96-inch-diameter discharge manifold connects to the 96-inch-diameter steel discharge pipe. The steel discharge pipe continues up the hillside where it ties into the intake structure of the Black Rock Branch Canal. Steel piping was designed in accordance with American Water Works Association (AWWA) Manual M11 (AWWA, 2004) and the American Society of Civil Engineers (ASCE) Manuals and Reports on Engineering Practice No. 79 (ASCE, 1993). The minimum plate thickness for handling is calculated in accordance with AWWA recommendations. This minimum thickness is the lesser of d/288 and (d+20)/400 where d is pipe diameter in inches. After fabrication, all piping would be hydrostatically tested to 1.5 times the design pressure. Steel plate used for the manifolds and discharge pipes conforms to American Society for Testing and Materials (ASTM) A36. This steel has good weldability and resistance to brittle fracture.

Each individual pump discharge line is provided with a check valve and a motor-operated discharge butterfly valve. The check valve is utilized during the startup procedure of the pumps and will prevent reverse flow through the pumps during a power outage. The motor-operated maintenance butterfly valve is only to be closed for maintenance on the pump and the check valve.

3.1.10.3. Valves

Each individual pump discharge line is provided with a check valve, air valve, and a motor-operated discharge butterfly valve.

3.1.10.4. Butterfly Valves for Intake and Discharge Manifolds

The motor-operated butterfly valves are commercially available and manufactured in accordance with AWWA C504 (AWWA, 1994) and are suitable for pressures up to 150 pounds per square inch gauge (psig). They are intended to serve as shutoff valves for preventing flow during maintenance on the pumps and check valves.

3.1.10.5. Check Valves

Check valves will prevent reverse flow through the pumps during a power outage. The check valves are equipped with a hydraulically operated dampening device. Upon pump unit shutdown, these valves freely close the first 90 percent of travel; the final 10 percent of travel is controlled by a hydraulic dampening device. The hydraulic dampening device has an adjustable dashpot that can be used to control the time for the last 10 percent of closure. The check valves are rated for 250 psig cold water service.

3.1.10.6. Air Valves

Air valve assemblies are provided on the pump discharge lines. The air valves are combination-type which both release air and admit air. Air-vacuum valve assemblies are provided on the pump discharge lines immediately downstream from the pumps.

3.1.10.7. Auxiliary Mechanical Systems

The auxiliary mechanical systems in the pumping plant consist of a gravity drainage system, fire suppression system, compressed air system, service water system, domestic water and sanitary waste system.

The gravity drainage system consists of floor drains around the perimeter of the plant interior and in floor areas throughout the building where leakage of water can be expected. Sloped cast iron hub and spigot soil pipe will collect waste water from the floor drains and will convey the water by gravity to a buried septic tank with integral separators.

<u>Assumption</u>: Minimal quantities of oil will be washed down the gravity drainage system.

The fire suppression system consists of:

- A wet pipe sprinkler system in the office and kitchen as well as hose stations in the pumping unit area. A deluge system will be included if flammable materials are stored in the building.
- Portable multipurpose class ABC wall-mounted dry chemical fire extinguishers located at every exterior door and at a maximum distance of 75 feet, and a wheeled dry chemical fire extinguisher to extinguish fires in flammable materials and equipment fires in the plant.
- A clean agent suppression system for the Switchgear Room which houses the switchgear and transformers.

<u>Assumptions</u>:

- Low risk of fire when not operating equipment.
- Fire protection system operable during pumping season only. Allows tapping a line off discharge manifold to use as fire water for hoses and sprinklers.

A compressed air system will be provided in the interior of the pumping plant for use by plant personnel for the operation of pneumatic tools during maintenance activities. The system consists of a stationary vertical receiver tank, two rotary screw air compressors, an air dryer, and steel distribution piping.

A nonpotable service water system is provided for plant maintenance operations and to supply water to the plant evaporative coolers. The plant will be supplied from a tap in the discharge manifold. The water will be directed to a self-cleaning strainer. Corrosion-resistant copper tubing will supply water to the service water outlets and the evaporative coolers. The service water outlets will be supplied with quick connects distributed throughout the interior of the plant.

Assumption:

- Service water will not be required when the pumps are not operational.

The domestic water and sanitary waste system consists of copper tubing, PVC vent pipe and fittings, and cast iron wastewater pipe and fittings with cleanouts. An electric instant hot water heater will provide necessary water to the lavatory and utility sinks. A wall-hung efficient autoflush toilet will be provided for the bathroom.

The heating, ventilating, and air conditioning (HVAC) systems for the Black Rock Coulee Pumping Plant are designed for both equipment protection purposes and occupancy requirements. The following systems will be used:

3.1.10.7.1 *Unit Bay and Service Bay*

The HVAC systems for the Unit Bay and Service Bay areas are designed for equipment protection purposes, not for occupancy. Indoor design temperatures are assumed to be 95^{o} F and 55^{o} F for summer and winter, respectively.

Cooling is provided by evaporative cooling systems. Sizing of equipment is based upon maximum outdoor climatic conditions and indoor electrical equipment cooling requirements. Pump motors are assumed to be air cooled.

Heating will be provided by electric type duct heaters. Localized heating is provided by wall mounted electric unit heaters.

3.1.10.7.2 *Office and Break Room*

Comfort cooling and heating is provided by split-system heat pumps. Indoor design temperatures are assumed to be 75^{o} F and 68^{o} F for summer and winter, respectively.

3.1.10.7.3 *Electrical Control Room*

The HVAC systems for the Motor Control Room and Switchgear Room are designed for equipment protection purposes and limited occupancy. Indoor design temperatures are assumed to be 104^{o} F and 55^{o} F for summer and winter, respectively, during unoccupied periods. During occupancy, temperatures will be maintained at 75^{o} F and 65^{o} F for summer and winter, respectively.

Cooling is provided by evaporative cooling systems. Sizing of equipment is based upon maximum outdoor climatic conditions and indoor electrical equipment cooling requirements.

Heating will be provided by electric type duct heaters.

3.1.10.7.4 ***SCADA/Phone Room***

The HVAC systems for the SCADA and Phone Room are designed for equipment protection purposes and limited occupancy. Indoor design temperatures are assumed to be 75^{o} F year round.

Cooling is provided by split-system heat pump system. Sizing of equipment is based upon maximum outdoor climatic conditions and indoor electrical equipment cooling requirements.

Heating will be provided by electric-type duct heaters.

SCADA management of plant HVAC systems is provided to remotely monitor and control the plant HVAC system.

An overhead traveling bridge crane will be provided in the interior of the plant for use during maintenance operations. The crane capacity is based on handling the motor and pump separately. The hoist, trolley, and bridge are electric powered with radio controls.

An 8-path ultrasonic flowmeter will be provided in each pumping unit discharge line to measure the flow of water from each individual pump.

A diesel engine-generator (EG) set is provided as a standby (backup) power source for essential equipment in the event the primary commercial power is unavailable. An automatic transfer switch is provided with the EG set. The EG set is not sized to operate the pumping plant's main pumps.

Assumption: Since essential equipment could not be determined at this feasibility level, the EG set size was selected in the size range used at other pumping plants.

Trashracks will be provided on the inlet to the pumping plant to prevent trash such as logs and debris from entering the plant sump and damaging the main pumping units.

The trashracks will be cleaned by an automatic trash rake. A conveyor will be provided to carry the raked debris away from the intake where it is elevated and dumped.

Stoplog seats and guides in the inlet to the pumping plant sump downstream of the trashracks will be provided to allow the sump to be dewatered using future stoplogs during maintenance activities (Note: Stoplogs are not provided as part of this feasibility study).

3.1.10.8. Air Chamber

A 22-foot-diameter spherical air chamber is included in the design to provide protection to the buried discharge pipeline against hydraulic transients caused by unforeseen shut downs of the pumping units.

3.1.10.9. Electrical Equipment

A 4.16-kV motor bus provides power for the twelve squirrel-cage induction pump motors. Motors 1 and 12 are driven with a variable frequency drive. Motors 2 and 11 are started full-voltage across-the-line with medium-voltage vacuum contactors with power factor correction capacitors.

A double-ended, 480-volt station service switchboard provides power for large low-voltage loads such as motor control centers, distribution panelboards, and distribution transformers.

Auxiliary loads, which require 3-phase, 480-volt power, such as air compressors, sump pumps, HVAC equipment, valve or gate motor-operators, etc., are fed from a 480-volt motor control center.

Lighting loads including 120-volt convenience outlets are fed from 208Y/120-volt dry-type transformers and lighting panel boards.

SCADA equipment would be provided to connect the discharge structure to the overall project monitoring and control system located in Ephrata, Washington.

3.1.10.10. Substation and Transmission Line

The plant is planned to be supplied from a separately developed (fenced) open-air, low-profile, 115-kV substation consisting of a 16 MVA, 115 – 4.16-kV oil-filled power transformer, high-side SF-6 (sulfur hexafluoride) gas-insulated power circuit breaker, and associated electrical equipment (disconnect switches, instrument transformers, etc.), steel structures, foundations, and oil containment system.

It was assumed that power for the plant would come from a local utility out of Moses Lake, Washington, which is located approximately 14.2 miles southwest of the plant site. A new transmission line would be constructed to the site. This study assumes that 115-kV service would be available. If 115-kV service is not available at Moses Lake, a 115-kV source of power could come from Grand Coulee, Washington, or of another voltage (e.g., 69-kV) from a local source.

Chapter 4: Canal-Side and Re-lift (Booster) Pumping Plants

A requirement of each water delivery alternative is for the construction of canal-side pumping plants at numerous locations along the existing and proposed canals associated with this feasibility study. The purpose of these plants is to pump water from the canal into the proposed buried pipe laterals which, in turn, deliver the water to the irrigated fields that are part of this feasibility study. In addition, because of the terrain present in the project area and the distance of the fields from the canals, some re-lift (booster) pumping plants are required on several of the proposed pipe laterals. The purpose of these plants is to provide additional lift to the water to get to fields that are located at higher elevations within the project boundary or at greater distances from the canal. To meet the requirements of water delivery, Alternative 2 would require construction of 6 canal-side pumping plants and 5 re-lift pumping plants. To meet the requirements of water delivery, Alternative 3 would require construction of 21 canal-side pumping plants and 8 re-lift pumping plants.

4.1. Canal-side Pumping Plant General Description

Each canal-side pumping plant is a wet sump plant comprised of a reinforced concrete substructure that supports and protects the pumping units and the electrical and mechanical equipment that is necessary to make the plant functional. The layout of the pumping plant is governed by the number, type, and size of the selected pumping units and equipment, the relationship between the electrical and mechanical systems, required clearances to maintain a safe work environment for the operation and maintenance personnel, and handling requirements for the various pieces of equipment during initial installation and subsequent maintenance operations. The intake for the plant is constructed into the side of the canal and is protected by a trashrack to prevent trash and debris from entering the sump. Water screens will be provided between the intake and the pumping units to screen out small debris and moss that might be drawn into the sump. The water screens will be provided with a trash conveyor system which will convey the small debris and moss to a location adjacent to the plant for accumulation prior to transfer to a disposal site.

The layout of the pumping plant service yard is based on the existing site topography, the submergence requirements of the pumping units, the alignment of the discharge pipelines, equipment space requirements for the pumping plant and switchyard, access to and around the pumping plant and switchyard for construction vehicles, and access to and around the pumping plant and switchyard for maintenance vehicles. The service yard was built up to accommodate the hydraulic needs of the pumps and to allow for proper drainage purposes.

4.1.1 **Pumps**

All canal-side pumping plants draw water from a canal invert and discharge to an elevated tank a distance away. Static head and friction head for the discharge piping from the plant to the tank mean water surface elevation were provided by Reclamation's Water Conveyance Group in Denver, Colorado. The unit piping losses were calculated and then added to the friction and static heads calculated by the Water Conveyance Group to come up with the total dynamic head. The pumps were selected using this total dynamic head.

All pumps were analyzed for run-out conditions with the assumption that only one pump would be running in this condition. The pumps were selected providing a range of operation near the best efficiency point. Any pumping configuration that required a wide total dynamic head range was selected referencing the Hydraulic Institute standard *Centrifugal and Vertical Pumps for Allowable Operating Range* (ANSI/HI, 1997) of 70 percent to 120 percent of the best efficiency point flow for pumps with specific speeds less than or equal to 4,500 (US Units).

A minimum of four pumps were selected for each pumping plant. Using no less than four pumps in a pumping plant allows for minimum of three quarters flow in the event a pumping unit being out of service. All pumps were selected based on providing a range of operation near the best efficiency point. The pumps at the East Low Canal, East High Canal, and Black Rock Branch Canal pumping plants are identical. None of the pumping plants have an installed spare or spare set of bowls and impellers.

It was assumed that variable-frequency drives would not be needed since multiple pumps will be available at every pumping plant. Pumping water to tanks as well as energizing and de-energizing of individual pumps should provide sufficient control of flow capacity.

Vertical turbine pumps were selected for all canal-side pumping plants. Alternate types of pumps were not evaluated, since previous evaluations of pumping plants have shown that the vertical turbine pumps are the most cost-effective pump selection for canal-side pumping plants.

For this feasibility study, the selected materials for the pumps are cast iron or coated cast iron bowls and steel, bronze and/or aluminum bronze impellers. Water quality was assumed to be adequate for the selected standard pump materials.

Steel pipe and valves were selected for pumping plants with discharge manifolds. The individual steel pipe branches and the main manifold are sized to limit the flow velocity and minimize friction loss. The steel piping will connect the individual pump discharge lines into a single steel discharge manifold. The manifold will extend from the pumping plant to where it connects to the line pipe. Steel piping was designed in accordance with American Water Works Association (AWWA) M11 (AWWA, 2004) and American Society of Civil Engineers (ASCE) Manuals and Reports on Engineering Practice No. 79 (ASCE, 1993).

The minimum pipe wall thickness was calculated in accordance with AWWA recommendations. This included checking the thickness required due to the system pressure and the minimum thickness required for handling. After fabrication, all piping would be hydrostatically tested to 1.5 times the design pressure.

Steel plate used for the manifolds and discharge pipes conforms to the American Society for Testing and Materials (ASTM) A36 Steel. This steel has good weldability and resistance to brittle fracture.

4.1.2 Valves

Each individual pump discharge line is provided with a check valve, air valves, and a motor-operated discharge butterfly valve.

4.1.3 Butterfly Valves

The motor-operated butterfly valves are commercially available and manufactured in accordance with AWWA C504 (AWWA, 1994) and are suitable for pressures up to 150 pounds per square inch gauge (psig). They are intended to serve as shutoff valves for preventing reverse flow during maintenance on the pumps and check valves.

4.1.4 Check Valves

Check valves will prevent reverse flow through the pumps during a power outage. The check valves are equipped with a hydraulically operated dampening device. Upon pump unit shutdown, these valves freely close the first 90 percent of travel; the final 10 percent of travel is controlled by a hydraulic dampening device. The hydraulic dampening device has an adjustable dashpot that can be used to control the time for the last 10 percent of closure. The check valves are rated for pressures up to 250 psig cold water service.

4.1.5 Air Valves

Air valve assemblies are provided on the pump discharge lines at all the pumping plants. The air valves are combination-type which both release air and admit air. Air-vacuum valve assemblies are provided on the pump discharge lines immediately downstream from the pumps.

Table 4- 1. Canal-side pumping plant unit data

Required for Water Delivery Alternative 2	Required for Water Delivery Alternative 3	Pumping Plant Name	Number of Pumping Units[1]	Total Dynamic Head (feet)	Total Plant Flow Capacity (ft^3/s)[2]	Motor Rated Horsepower per Pump
East Low Canal (ELC)						
X	X	EL47	5	234	117.15	800
X	X	EL53	5	317	173.95	1,500
X	X	EL68	7	346.5	355.46	2,500
X	X	EL75	4	299	51.00	600
X	X	EL80	4	237	130.00	1,250
X	X	EL85	4	247	55.32	500
East High Canal (EHC)						
----	X	EH4	4	324.14	15.88	200
----	X	EH11	4	224.13	48.36	400
----	X	EH19	5	243	62.20	450
----	X	EH29	4	181	48.88	350
----	X	EH33	4	332.53	90.84	1,250
----	X	EH35	6	132.78	116.58	400
----	X	EH42	4	171.29	111.68	700
----	X	EH47	4	68.52	22.56	60
Black Rock Branch Canal (BRBC)						
----	X	BRB2	4	101.23	27.96	100
----	X	BRB7	4	198.71	95.68	700
----	X	BRB11	4	158	78.60	450
----	X	BRB17	4	191	1.56	15
----	X	BRB18	4	161	3.92	25
----	X	BRB27	4	68.5	77.32	200
----	X	BRB28	4	143.5	12.56	75

[1]Vertical turbine pumping units

[2]Includes 3% wear factor.

4.2. Re-lift (Booster) Pumping Plant General Description

Each re-lift pumping plant is comprised of a reinforced concrete slab on grade that supports and protects the pumping units and the electrical and mechanical equipment that is necessary to make the plant functional. The layout of the pumping plant is governed by the number, type, and size of the selected pumping units and equipment, the relationship between the electrical and mechanical systems, required clearances to maintain a safe work environment for the operation and maintenance personnel, and handling requirements for the various pieces of equipment during initial installation and subsequent maintenance operations. Water is delivered to the intake side of the pumping units via a buried intake manifold pipe. Water is discharged by the pumping units into a buried discharge pipeline which, in turn, delivers the water to the fields.

The layout of the pumping plant service yard is based on the existing site topography, the submergence requirements of the pumping units, the alignment of the intake and discharge pipelines, equipment space requirements for the pumping plant and switchyard, access to and around the pumping plant and switchyard for construction vehicles, and access to and around the pumping plant and switchyard for maintenance vehicles. The service yard was built up to accommodate the hydraulic needs of the pumps and to allow for proper drainage purposes.

4.2.1 Pumps

All re-lift pumping plants receive water from the main discharge line of the first elevated tank a distance away. Static head and friction head for the discharge piping from the plant to the tank mean water surface elevation were provided. The unit piping losses were calculated and then added to the friction and static heads calculated by Water Conveyance to determine the total dynamic head. The pumps were selected using this total dynamic head.

With the above configuration of a tank supplying the water for each re-lift pump, the total dynamic head of the plant will need to kept at a high level in the spring and summer months. If the head is allowed to drop in the fall and winter, the head range for the pump would be too wide and the pump would be inoperable at lower total dynamic heads. This can be prevented by operating the re-lift plants only when the elevated storage tank is at a certain level. Future studies could investigate different size pumps in each plant to accommodate the head and flow ranges that could be experienced throughout the year.

Table 4- 2. Re-lift (Booster) Pumping Plant unit data

Required for Water Delivery Alternative 2	Required for Water Delivery Alternative 3	Pumping Plant Name	Number of Pumping Units[1]	Total Dynamic Head (feet)	Pumping Total Capacity (ft^3/s)[2]	Motor Rated Horsepower per Pump
East Low Canal (ELC)						
X	X	EL47R	5	219	47.05	300
X	X	EL53R	5	132	78.90	350
X	X	EL68R	8	227	179.84	700
X	X	EL80R	4	149	52.52	300
X	X	EL89R2	5	127	2.70	15
East High Canal (EHC)						
----	X	EH19R	5	243	62.20	450
----	X	EH50R	4	101	7.76	30
Black Rock Branch Canal (BRBC)						
----	X	BRB7R	4	177	71.84	600

[1]Horizontal split-case centrifugal pumping units

[2]Includes 3% wear factor.

The elevated tank provides a direct supply to the pump; there is no storage tank before the re-lift plants for control of flow and pressure. Another factor to be considered with the above configuration is the necessity of high-pressure seals on the suction side of the pump. Pump manufacturers were contacted about the pumps receiving a higher suction pressure and potential problems with the pump shaft seal. The shaft seals can be equipped with the high-pressure seals if necessary when the pumps are being specified.

All pumps were also analyzed for run-out conditions with the assumption that only one pump would be running in this condition. The pumps were selected to provide a range of operation near the best efficiency point. In certain configurations in which the pump had a wide operating range, many changes had to be made to the discharge pipe diameters and the length of pipe to the regulating tank to limit the amount of friction head to which the pumps were subject. Any pumping configuration that required a wide total dynamic head range was selected referencing the Hydraulic Institute standard *Centrifugal and Vertical Pumps for Allowable Operating Range* (ANSI/HI, 1997) of 70 percent to 120 percent of the best efficiency point flow for pumps with specific speeds less than or equal to 4500 (US Units).

A minimum of four pumps were selected for each pumping plant. Using no less than four pumps in a pumping plant allows for minimum of three quarters flow in the event a pumping unit being out of service. All pumps were selected based on providing a range of operation near the best efficiency point. At any pumping plant, the pumps are identical. None of the pumping plants have an installed spare or spare set of volutes and impellers.

It was assumed that variable-frequency drives would not be needed since multiple pumps will be available at every pumping plant. Pumping water to tanks as well as energizing and de-energizing of individual pumps will provide some control of flow capacity.

All of the re-lift pumping plants have a variable range of positive suction pressure and are supplied with water from an elevated tank. Horizontal split-case pumps were selected for all re-lift pumping plants. Vertical turbine can-style pumps were also considered, but horizontal split-case pumps were a better selection.

For this feasibility study, pump materials selected are ductile or cast iron casings and bronze impellers. Water quality was assumed to be adequate for the selected standard pump materials.

Steel pipe and valves were selected for pumping plants with suction and discharge manifolds. The individual steel pipe branches and the main manifold are sized to limit the flow velocity and minimize friction loss.

For each pumping plant, a suction manifold is connected from a regulating tank to the pumping plant. At the pumping plant structure, the pipe manifolds into the individual pump intake lines that feed the pumping units.

Downstream of each pump, the individual pump discharge pipes connect into a single steel discharge manifold. The discharge manifold extends from the pumping plant structure to the discharge line pipe.

Steel piping was designed in accordance with AWWA M11 (AWWA, 2004) and ASCE Manuals and Reports on Engineering Practice No. 79 (ASCE, 1993). The minimum pipe wall thickness was calculated in accordance with AWWA recommendations. This included checking the thickness required due to the system pressure and the minimum thickness required for handling. After fabrication, all piping would be hydrostatically tested to 1.5 times the design pressure.

Steel plate used for the manifolds and discharge pipes conforms to ASTM A36. This steel has good weldability and resistance to brittle fracture.

4.2.2 Valves

Each individual pump suction line is provided with a motor operated butterfly valve. It is only to be closed for maintenance on the pump. Each individual pump discharge line is provided with a check valve and a motor-operated discharge butterfly valve.

4.2.3 Butterfly Valves for Suction and Discharge Manifolds

The motor butterfly valves are commercially available and manufactured in accordance with AWWA C504 (AWWA, 1994) and suitable for pressures up to 150 psig. They are intended to serve as shutoff valves for preventing flow during maintenance on the pumps and check valves.

4.2.4 Check Valves

Check valves will prevent reverse flow through the pumps during a power outage. The check valves are equipped with a hydraulically operated dampening device. Upon pump unit shutdown, these valves freely close the first 90 percent of travel; the final 10 percent of travel is controlled by a hydraulic dampening device. The hydraulic dampening device has an adjustable dashpot that can be used to control the time for the last 10 percent of closure. The check valves are rated for 250 psig cold water service.

4.2.5 Air Valves

There is a combination-type air valve mounted on each individual pump suction pipe, the top of each pumping unit, and on each individual pump discharge pipe. The combination air valves release air at the high points of the piping and pumps. They will also continuously permit the release of air during pump operations. All air valves are rated for 300 psig cold water service. The ball valve provided below each air valve and manifold is for isolation of the air valve to permit air valve maintenance.

4.3. Electrical Design Considerations

4.3.1 Motor Bus Voltage Selection

Motor bus voltage selection is based on the maximum horsepower of individual pump motors and the total kVA load of the pumping plant. For motors not greater than 300 horsepower and total plant load less than 2000 kVA, the preferred motor bus voltage is

480 V. For larger motors and greater plant loads, a motor bus voltage of either 4.16 kV or 6.9 kV is selected.

4.3.2 Motor Type Selection

Motors rated 1250 horsepower or less are squirrel-cage induction type. This includes all motors in the feasibility study except the 2500 horsepower main pump motors at EL 68 Pumping Plant. These motors are the synchronous type with brushless exciters.

4.3.3 Motor Enclosure Selection

The enclosures for motor 700 horsepower and less are TEFC (totally-enclosed fan-cooled). Motors rated 800 to 1000 horsepower have WP1 (weather protected 1) enclosures. Motors rated 1250 horsepower and greater are TEWAC (totally-enclosed water-to-air cooled).

4.3.4 Motor Starting

Motors are started full-voltage, across-the-line with magnetic motor contactors. Motor contactors for the 4.16- and 6.9-kV motors are the medium-voltage vacuum type.

Connection between the switchyard or unit substation and the pumping plant motor bus is with non-segregated phase bus.

4.3.5 Pumping Plant Auxiliary Loads

Auxiliary loads, which require 3-phase, 480-volt power, such as air compressors, sump pumps, HVAC fans, valve or gate motor-operators, etc., are fed from a 600-volt motor control center.

4.3.6 Pumping Plant Lighting Loads

Lighting loads including 120-volt convenience outlets are fed from 208Y/12- volt dry-type transformers and lighting panelboard.

4.3.7 Substations and Switchyards

4.3.7.1. East Low Canal Pumping Plant Switchyards

All plants will be supplied from 34.5-kV, outdoor, metal-enclosed, single-ended, unit substations except EL 68 Pumping Plant, which will be supplied from a separately developed (fenced) 34.5-kV open-air switchyard.

Each unit substation will consist of a step-down power transformer, high-side disconnect switch and fuse, and low-side circuit breaker. For the larger plants with unit substations (low side greater than 480V), the transformers will be liquid-filled (natural ester based for better fire performance and for environmental considerations), forced-cooled, and the smaller units will be self-cooled dry type. Low-side voltages range from 4.16-kV to 6.9-kV for the larger plants. Each unit substation can be placed within the fenced plant service yard.

The open air switchyard for EL 68 Pumping Plant will consist of a 25 MVA, 34.5 – 6.9-kV oil-filled power transformer, high-side SF6-gas power circuit breaker with air-type disconnect switches.

Transformer sizes were somewhat standardized and were based on available standard sizes.

4.3.7.2. East High Canal and Black Rock Branch Pumping Plants

All plants will be supplied from 34.5-kV, metal-enclosed, single-ended, unit substations.

Each unit substation will consist of a step-down power transformer, high-side disconnect switch and fuse, and low-side circuit breaker. For the larger plants with unit substations (low side greater than 480V), the transformers with be liquid filled (natural ester based for better fire performance and for environmental considerations), forced-cooled, and the smaller units will be self-cooled dry type. Low-side voltages range from 4.16-kV to 6.9-kV for the larger plants. (Note: At the time of writing this document, exact sizes of the Black Rock Branch Canal pumping plants were unknown and equipment sizes were estimated based on average of the other plants.)

Transformer sizes were somewhat standardized and were based on available standard sizes.

4.3.8 **Transmission Lines**

4.3.8.1. East Low Canal Transmission Line

It is anticipated that power for the East Low Canal plants will come from a local utility out of Moses Lake. This study assumes that 34.5-kV service will be available. If not, a 115-kV source could come from Grand Coulee.

4.3.8.2. East High Canal and Black Rock Branch Canal Transmission Lines

It is anticipated that power for the East High Canal and Black Rock Branch Canal pumping plants will come from a local utility out of Grand Coulee. This study assumes that 115-kV service should be used due to the distance involved. This will require a substation to step the voltage down to 34.5-kV for distribution to the plants. The substation would consist of a 50-MVA power transformer and three (3) distribution transmission lines in addition to the main transmission line (4 total).

Chapter 5: Rocky Coulee Storage Facility

The proposed Rocky Coulee Storage Facility is an offstream storage reservoir capable of storing 109,315 acre-feet of Columbia River water for use during the irrigation season. The reservoir would be formed by an embankment dam constructed across Rocky Coulee approximately 8 miles northeast of the town of Moses Lake, Washington. The facility is comprised of a zoned earthfill embankment dam, an inlet/outlet channel constructed from the existing East Low Canal to the reservoir, a reservoir inlet structure, a pumping plant and switchyard, a pump discharge structure, and a low-level reservoir outlet works. Data describing facility types, sizes, and capacities are shown in Table 3-1. For the location plan of this facility refer to Drawing No. 222-D-50046.

Table 5- 1. Rocky Coulee Storage Facility data

Facility/Characteristic	Size/Quantity
Reservoir	
Surface area at full pool	2,812 acres
Active storage capacity	109,315 acre-feet
Maximum water surface elevation	1300.8 feet (Probable Maximum Flood)
100-year flood water surface elevation	1291.1 feet
Top of active storage water surface elevation	1,291.0 feet
Minimum water surface elevation	1215.0 feet
Dam	
Type	Zoned earthfill embankment
Crest elevation	1,305 feet
Crest width	30 feet
Crest length	3,100 feet
Inlet/outlet channel length and flow capacity	1.27 miles; 1,060 ft^3/s
Low level outlet works flow capacity	2,020 ft^3/s at top of active storage El. 1291.0
Pumping Plant	
Pumping unit type Individual pumping unit flow capacity Number of pumping units	Horizontal split-case centrifugal 91.9 ft^3/s (includes 3% wear factor) 8
Plant flow capacity (rated)	735.4 ft^3/s (includes 3% wear factor)
Pumping unit lift (rated)	88 feet @ 41,240 gpm

Each year the reservoir would be filled when excess water is available in the Columbia River (September and October). This water would be conveyed to the reservoir via gravity through an inlet/outlet channel constructed from the existing East Low Canal to the reservoir. Where the channel intersects the dam, a reservoir inlet structure would be constructed through the dam to permit the water to safely pass through the dam and discharge into the reservoir. It takes approximately 60 calendar days to fill the reservoir. The water would be stored in the reservoir during the winter months for use during the following irrigation season (March through August). During the irrigation season, the stored water would be pumped from the reservoir via the proposed Rocky Coulee Pumping Plant to the pump discharge structure located in the side of the inlet/outlet channel. Discharged water would flow via gravity from the pump discharge structure to the East Low Canal through the inlet/outlet channel. Pumping would continue until the reservoir is essentially empty. It takes approximately 85 calendar days to empty the reservoir. This method of operation would occur each year.

5.1. Rocky Coulee Dam

5.1.1 Overview

One of the proposed sources of water for the feasibility study is a new storage reservoir and dam at Rocky Coulee. The Rocky Coulee damsite has been studied during this feasibility design process and design concepts have been developed and estimated. Design considerations for this embankment are discussed below, along with a detailed description of the design concepts.

5.1.2 Design Considerations

There are several key design considerations associated with the construction of a dam at Rocky Coulee. In general, these considerations are typical of many embankment damsites, and are not viewed to be indicative of any “fatal flaws” that would indicate a dam at this site is not technically feasible. Rather, it is judged that a safe embankment can be designed and constructed at Rocky Coulee, without any particularly unusual measures or features beyond what are typically considered for a major embankment dam. The key design considerations affecting the embankment are listed below.

5.1.2.1. Reservoir Size

During the appraisal study, reservoir storage sizes for various alternatives considered such factors as availability of Columbia River water, canal availability and capacity, and water demands (which were based on crop distribution, annual on-farm allotments, and canal efficiencies). Given the relatively small size of the Rocky Coulee site, the reservoir was simply sized at the maximum capacity (limited by the feeder canal invert), or about 126,000 acre-feet as determined by preliminary area-capacity estimates.

For the feasibility design, new topography was developed from aerial surveys, and a new area-capacity table was developed. Based on these new data, the estimated storage capacity of Rocky Coulee Reservoir is 109,315 acre-feet, corresponding to a top of active conservation capacity elevation of approximately 1,291 feet.

5.1.2.2. Hydrologic Loading Assumptions

For the feasibility study, a Probable Maximum Flood (PMF) study was conducted, which led to the development of a proposed PMF for use in designing the dam and appurtenant structures at Rocky Coulee Reservoir. From this study, the PMF for Rocky Coulee Dam consists of a December rain-on-snow storm with a peak flow of 18,760 ft^3/s and a 5-day volume of 45,930 acre-feet. This PMF was then routed through the reservoir, using only the proposed outlet works for releases since there is no service spillway. The routing assumed the initial reservoir water surface was at the top of active conservation (elevation 1,291 feet) and utilized the full capacity of the outlet works (2,202 ft^3/s at elevation 1,291 feet). The resulting reservoir maximum water surface due to the PMF is estimated to be elevation 1,300.8 feet.

5.1.2.3. Seismic Loading Assumptions

A feasibility-level probabilistic seismic hazard analysis (PSHA) was performed for the Odessa subarea. There are a number of potential seismic sources that may impact the area, including random (background) earthquakes; faults associated with the Yakima fold belt, a prominent group of mostly east-west striking folds; and the Cascadia Subduction Zone which is capable of producing very large magnitude earthquakes. Based on this study, the peak horizontal ground acceleration for a 10,000-year return period is estimated to be 0.29g, and 0.46g for a 50,000-year return period.

This level of earthquake shaking leads to the possibility that lower density embankment or foundation saturated soils may experience liquefaction, which is essentially a loss of strength that can result in large slope failures. To mitigate this concern, it is critical that all potentially liquefiable foundation soils are removed and that all embankment materials are compacted to high densities, which can be routinely accomplished through the use of large rollers.

Another potential concern is that earthquake shaking, if severe and of sufficiently long duration, can induce slope failures in an embankment. This concern can be addressed by carefully analyzing the dam for potential deformations from the expected earthquake load, and designing crest dimensions, zoning, and embankment slopes to ensure stability, as well as selecting strong materials and keeping the phreatic surface (water level) in the embankment as low as possible. Seismic loading can additionally lead to cracking, sometimes associated with deformations. To address this concern, a chimney filter will be included in the embankment to ensure that any seepage through seismically induced cracks is filtered and thus unable to lead to internal erosion.

5.1.2.4. Potential Faulting

An additional concern in areas subject to earthquake loading is the possibility of fault displacements within the footprint of the embankments. Based on the limited preliminary geologic characterization of the site, there is no evidence to indicate that a potentially active fault exists within the Rocky Coulee dam or reservoir area.

5.1.2.5. Varying Rock Quality

The bedrock at the site consists of an interbedded sequence of volcanic and sedimentary rocks of the Columbia River Basalt Group. In essence, these are a series of basalt flows that were extruded and flowed over the Columbia Basin between 18 and 6 million years ago. Individual flows were up to 100 feet thick, and the time periods between sequential flows were from hundreds to tens of thousands of years, which allowed for sedimentation deposition between basalt flows. As a result the bedrock stratigraphy consists of a number of different basalt flows with sedimentary interbeds separating some of these flows. In addition, due to nature of the flow deposition, the basalts may contain sediments that are “rafted” within the basalt or contain “pillow” structures that also contain pods of fine sediment and fractured basalt. It is not unusual to see “interflow zones” of higher permeability at the top or bottom of flows due to shearing and intermixing during deposition or resulting from differences in cooling of the flows.

As the bedrock surface is excavated during construction, it would be expected that rock quality could vary significantly as different areas of one flow or different flows are uncovered. This is by no means a significant detriment for an embankment foundation, but does mean some flexibility will be needed during construction to ensure a suitable foundation is reached. Considerable onsite presence will thus be needed to determine the adequacy of the bedrock and the degree of foundation treatment measures such as additional excavation, slush grouting, and filter placement.

In addition, the varying bedrock composition and quality will require additional investigations during advanced design phases to better understand the bedrock permeability (fracture density, openness, infilling characteristics, etc.) and to develop a foundation grouting program to explore foundation conditions and to potentially reduce bedrock seepage. The feasibility-level drilling program conducted for this feasibility study included three drill holes along the axis of the proposed embankment. The degree of weathering and fracturing varied, and included some areas in the upper portion of the bedrock that were described as intensely fractured. Flow contacts and interflow zones, as well as vertical joints associated with columnar basalt in the Roza member, were encountered in the drilling. Permeability testing of the bedrock revealed hydraulic conductivity values typical of the vesicular basalt in the area. Grouting is assumed to be necessary.

5.1.2.6. Construction Material Availability

A key consideration for the design of any embankment dam is utilization of available materials. The nature and availability of construction materials is important for both technical and economic reasons. The feasibility-level geologic investigations included seven drill holes in the reservoir area of Rocky Coulee to obtain information on potential borrow materials. The reservoir sediments primarily consist of silt, sandy silt, and silty sand. Water levels are fairly shallow, on the order of 14 to 18 feet, so some dewatering of the reservoir borrow areas may be required.

Material surveys conducted during appraisal studies indicated several sand and gravel pits in the general area, and estimated that a 10-mile haul from the Moses Lake area might be a reasonable assumption for the source of sand, gravel, and cobbles for the filters and drains.

5.1.2.7. Selection of Dam Type

Given these types of design considerations listed above, an initial step in the design process is to select the appropriate type or types of embankment dam to consider for this damsite. At Rocky Coulee, the embankment will be relatively small, both in terms of length and height. There appears to be sufficient borrow material in the vicinity to construct the size of dam anticipated at this site. Therefore, a conventional earthfill embankment appears to be an economical option at Rocky Coulee.

Rocky Coulee Dam is in an area of moderate seismicity. To address any seismic concerns at the smaller Rocky Coulee Dam, complete overburden removal and well designed filters will be included in the dam design.

5.1.3 **Embankment Design Concepts**

5.1.3.1. General Design Concepts

A zoned earthfill dam was selected based on the relatively small size of the embankment, absence of very deep overburden, and apparent availability of various embankment materials in the immediate vicinity. This traditional design, shown on Drawing No. (Drawing No. to be provided in final report), features a fairly wide central core consisting of impervious materials. Immediately downstream of the earth core is a zone 2 filter zone, consisting of clean sand and gravel designed to be filter compatible with the zone 1 core, thus preventing erosion of the core materials in the event of a crack. Downstream of the zone 2 filter is a clean gravel-and-cobble drainage zone (zone 3) to safely control and convey any seepage resulting from cracks in the core. These filter and drain zones extend to along the base of the foundation excavation to prevent piping of core material into foundation alluvium. The zone 2 and zone 3 also extend to the downstream toe and tie into a toe drain to allow any seepage to be safely directed to a filter-controlled exit. The embankment shells will consist of zone 4 material, primarily obtained from required excavation and local borrow areas.

A more detailed description of the various embankment zones, including expected material descriptions and properties and construction procedures, are included later in the subparagraph entitled “Embankment Zoning.”

5.1.3.2. Crest Elevation

For the Most Probable (MP) and Most Probable Low (MPL) cost alternatives, the crest elevation of Rocky Coulee Dam was simply set at approximately 4 feet above the maximum water surface (elevation 1,300.8 feet) resulting from storing the PMF flood volume, which results in a crest elevation of 1,305 feet. This is a simplified means of setting the crest elevation to allow for 4 feet of freeboard during the PMF. This crest elevation results in approximately 14 feet of freeboard with the reservoir at the top of active conservation level. For the Most Probable High (MPH) cost alternative, the crest elevation was assumed to be 6 feet above the PMF reservoir water surface, which would be a crest elevation of 1,307 feet.

During final designs, it may be worth looking into more detail at the potential for optimizing dam height by considering a spillway. In addition, wave run-up considerations will be looked at in later designs.

5.1.3.3. Embankment Slopes

The geometry of the outer embankment configuration will be the same for each of the MP, MPL, and MPH cost alternatives. The crest width of Rocky Coulee Dam will be 30 feet, a typical width for this type and size of embankment. The downstream slope will be set at 2 horizontal to 1 vertical, while the upstream slope will be 2.5 horizontal to 1 vertical. These are fairly typical slopes for earthfill embankments. Flatter slopes are not judged necessary due to the large amount of foundation excavation; the embankment will largely be founded on bedrock. Steeper slopes are not judged appropriate at this level of design considering the potential for earthquake shaking.

5.1.3.4. Embankment Zoning

5.1.3.4.1 ***Zone 1***

For this zoned earthfill embankment, the zone 1 serves as the core, or water barrier, for the dam. Although a plastic clay or clayey gravel would be the ideal core material, this type of material does not appear to be available in the general area. Therefore, the core is envisioned to be comprised of the silts that are plentiful throughout the reservoir area. Although some zone 1 may be available from the required excavations, it is unlikely that a sufficient quantity of good quality and consistent material to form the entire core will be available from required excavation. It is envisioned that additional zone 1 materials will need to be borrowed from reservoir locations in relatively close proximity to the dam. The following table shows the assumed percentage of usable required excavation and the borrow haul distances for each alternative.

Table 5- 2. Sources for zone 1 core materials

Alternative Cost Estimate	Assumed Percentage of Usable Required Excavation	Assumed One-Way Haul Distance from Borrow to Dam
MPL	40%	2 miles
MP	30%	3 miles
MPH	20%	5 miles

The zone 1 materials will be placed to 6-inch (possibly 9-inch) lifts and compacted to a dense state by tamping rollers. The moisture content of these soils will be carefully controlled to ensure that optimum properties for the core are achieved.

5.1.3.4.2 ***Zone 2***

This is a processed, clean sand-and-gravel zone that serves as a critical filter for the zone 1 core. This feature is particularly important given the low plasticity and erodible nature of the zone 1 core. Typically, this material will consist of a sand-and-gravel mixture with a maximum particle size of 3 inches and contain 0 to 3 percent fines. Because the zone serves as a filter, it is important that the material is as cohesionless as possible. This means that fines will be minimized, plastic fines not permitted, and any materials that display even a slight tendency toward cementation will be rejected. Depending upon availability and cost considerations, these materials may be processed onsite or obtained from commercial sources. For the MP cost estimate, it is assumed that the materials will be obtained from commercial sources in Moses Lake, about 10 miles from the damsite. For the MPL and MPH cost estimates, the assumed one-way haul distances will be 7 miles and 12 miles, respectively. Zone 2 materials will be compacted in 12-inch lifts by vibratory rollers. Within the horizontal blanket portion of this zone, the zone 2 will be 3 feet thick for all three cost options. The horizontal width of the filter comprising the chimney portion of the zone 2 is assumed to be 8 feet for the MPL option, 10 feet for the MP option, and 15 feet for the MPH option.

5.1.3.4.3 ***Zone 3***

This is a processed, clean gravel-and-cobble zone, placed immediately downstream of the zone 2 in both the MP and MPH alternatives so as to provide a two-stage filter and drain system. Zone 3 serves as a transition zone between the zone 2 and the downstream shell, and also as a drainage element to control any flows that pass through the zone 1 and zone 2. For the MPL alternative, it is assumed that a single stage filter/drain consisting solely of zone 2 materials will be sufficient; thus, there is no zone 3 in the MPL alternative. Depending upon availability and cost considerations, these materials may be processed onsite or obtained from commercial sources. For the MP cost estimate, it is assumed that the materials will be obtained from commercial sources in Moses Lake, about 10 miles from the damsite. For the MPH cost estimates, the assumed one-way haul distance will be 12 miles. This zone will also be compacted in 12-inch lifts by vibratory rollers. Within the horizontal blanket portion of this zone, the zone 3 will be 3 feet thick. The horizontal width of the drain comprising the chimney portion of the zone 3 is assumed to be 10 feet for the MP option and 15 feet for the MPH option.

5.1.3.4.4 ***Zone 4***

This zone comprises the upstream and downstream shells of the earthfill embankment. These materials will be obtained from the foundation excavation or from borrow areas in the immediate vicinity of the dam. The overburden soils comprising the zone 4 will likely consist of silts, sandy silts, silty sands, and sands. Some weathered rockfill materials from required excavation may also be included in this zone. For the MP cost estimate, it is assumed that the bulk of the materials will be obtained from the reservoir borrow areas with an average one-way haul distance from the damsite of 3 miles. For the MPL and MPH cost estimates, the assumed one-way haul distances will be 2 miles and 5 miles, respectively. Zone 4 materials will be placed in approximate 18-inch layers and compacted to a dense state by large vibratory rollers.

5.1.3.4.5 ***Slope Protection***

Upstream and downstream slope protection will consist of rockfill (riprap) quarried from the reservoir rims. The thickness of these layers will be 3 feet (normal to the slope) on the upstream slope and 2 feet on the downstream slope. An 18-inch bedding layer will be placed beneath the upstream riprap to ensure wave action does not remove embankment materials. No bedding is judged necessary on the downstream slope. For the MP cost estimate, it is assumed that the materials will be obtained within the reservoir area with an average one-way haul distance from the damsite of 3 miles. For the MPL and MPH cost estimates, the assumed one-way haul distances will be 2 miles and 5 miles, respectively.

5.1.4 **Foundation Excavation and Treatment**

5.1.4.1. Overburden Excavation

As discussed under “Design Considerations,” a key design consideration for the dam and dike is to prevent the potential for foundation liquefaction. Thus, for this feasibility study, complete excavation to bedrock beneath the majority of the footprint of the embankment is assumed. This will positively reduce all uncertainties of foundation liquefaction, and may support the use of steeper slopes in later designs.

At this level of design, with a limited number of drill holes along the dam axis, there is considerable uncertainty regarding the depth to bedrock. The 3 drill holes from the feasibility-level geologic investigations, as well as water well drill logs, test pit logs, and any other available data from the vicinity of the damsite, were used to estimate the amount of overburden. From these data, the maximum depth of overburden beneath Rocky Coulee Dam is estimated to be approximately 80 feet, and thins considerably on the abutments.

For this feasibility study, an estimated bedrock contour surface was developed based on the available data, and MP excavation quantities were based on the difference between original ground and the assumed bedrock surface. For the MPL foundation excavation, the bedrock surface was assumed to be 3 feet higher than in the MP model. For the MPH foundation excavation, the bedrock surface was assumed to be 7 feet lower than in the MP model. Gathering additional data in further levels of design will be critical to better define top of rock and the liquefaction potential of the overburden soils, and thus better estimate the amount of foundation excavation.

5.1.4.1.1 ***Localized Over-excavation of Rock***

Different basalt flows, as well as sedimentary interbeds and interflow zones, may be encountered during foundation excavation. The quality of rock at the contacts of these various flows is expected to be poor, and localized over-excavation to remove poor quality rock is anticipated at each site. In addition, there will likely be other areas, particularly under the dam impervious cores, where the rock quality is suspect and not ideally competent to support the impervious barrier, or localized irregularities in the rock, depending on the size, may create concerns for differential settlement or stress concentrations. In such areas, additional rock excavation, sometimes requiring drilling and blasting, may be required; however, this quantity of rock excavation is not judged to be particularly large. The uncertainty in this excavation is reflected by varying the quantities assumed in each of the 3 alternatives (1,000 yd^3 for MPL; 2,500 yd^3 for MP; and 5,000 yd^3 for MPH).

5.1.4.2. Treatment Beneath Earth Core

Because the zone 1 core is the key component comprising the water barrier of the dam, foundation treatment will be concentrated beneath the core. Foundation treatment beneath the remainder of an earthfill dam is less important, except in areas of highly weathered rock or fault zones where seepage/piping or displacement concerns exist. That type of special foundation treatment is discussed later in section 5.1.4.3. , "Miscellaneous Bedrock Treatment." The amount of foundation treatment required beneath the core will depend in large part on the quality of rock encountered. To minimize the potential for stress concentrations and differential cracking, rock excavation and dental concrete will be used to shape the bedrock surface so as to minimize abrupt changes, overhangs, etc. In addition, slush grouting may be needed in areas where the foundation is highly fractured or jointed and poses a risk of the zone 1 piping into such discontinuities. The uncertainties in the amount of these types of foundation treatment measures is addressed by varying the quantities in the 3 cost estimates, as shown in the following table.

Table 5- 3. Amounts of assumed foundation treatment

Alternative Cost Estimate	Assumed Amount of Slush Grouting	Assumed Amount of Dental Concrete
MPL	95,000 ft^2	1,000 yd^3
MP	160,000 ft^2	2,500 yd^3
MPH	250,000 ft^2	5,000 yd^3

In the bedrock beneath the core, foundation grouting will be a combination of blanket (consolidation) and curtain grouting to improve rock strength and create a low permeability zone beneath the core. Given the presence of fracturing in the basalts and areas of poor rock quality, extensive grouting is envisioned in certain areas. For this feasibility estimate, both blanket grouting and curtain grouting has been assumed for each of the 3 cost estimates. However, details of each grouting program are assumed to vary as shown on the following table.

Table 5- 4. Grouting program assumptions

Item	MPL	MP	MPH
Blanket Grouting			
Avg. hole centers	10 ft	10 ft	7.5 ft
Avg. hole depth	25 ft	30 ft	30 ft
Avg. grout take	1 ft^3/lineal ft	2 ft^3/lineal ft	2.5 ft^3/lineal ft
Curtain Grouting			
Number of rows	1	2	3
Avg. hole centers	10 ft	10 ft	7.5 ft
Avg. hole depth	90 ft	90 ft	90 ft
Avg. grout take	1 ft^3/lineal ft	2 ft^3/lineal ft	2.5 ft^3/lineal ft

5.1.4.3. Miscellaneous Bedrock Treatment

Special foundation treatment in some areas upstream of the zone 1 core of the embankment may be required in areas of particularly poor rock quality, which may include highly fractured rock, highly weathered or altered rock, or in areas of faulting. (Note that this type of protection will be afforded beneath all of the downstream shell at Rocky Coulee Dam given that the zone 2/zone 3 internal drainage system is located on the top of the rock surface.) When needed, the upstream foundation filter would consist of a 3-foot thickness of zone 2 material. The uncertainty in the amount of this type of treatment is reflected by varying the quantities assumed in each of the 3 alternatives (none assumed for MPL; 2700 yd^3 for MP; and 5300 yd^3 for MPH).

In the event that a foundation fault or highly fractured rock is unexpectedly encountered, additional upstream treatment might include locally increasing the width of the core, performing additional grouting, or even placing an impervious blanket for a distance upstream of the core.

5.1.5 **Diversion and Dewatering**

5.1.5.1. Diversion

Although it does not have an appreciable watercourse running through it, Rocky Coulee may have intermittent flows. In general, diversion does not appear to be a significant concern. Flows at the relatively small Rocky Coulee could probably be handled with a small cofferdam and pumping scheme. For these reasons, it appears the diversion at all sites will be relatively simple and not require any unusual or expensive efforts. Given this assumption, it appears reasonable to simply assume that the costs of diversion will be included in the design contingencies.

5.1.5.2. Dewatering

At this level of design, with limited drilling information and no pump test data in the overburden, it is difficult to assess the issue of foundation dewatering. Water wells in the general vicinity of the damsite indicate that groundwater will be encountered in the foundation beneath the dam. Conceptually, the dam foundation may be able to be dewatered by a relatively routine application of deep wells, perhaps with supplementary well points and with some sumping. For this feasibility design, the dewatering scheme was priced out as deep wells, with additional well points and sumps as needed. The following assumptions were made for the deep wells for each of the alternatives. The uncertainties in the dewatering work are addressed by varying the quantities in the three cost estimates, as shown in the following table.

Table 5- 5. Dewatering deep well layout assumptions

Alternative Cost Estimate	Well Spacing	Average Well Depth
MPL	150 ft	50 ft
MP	100 ft	50 ft
MPH	75 ft	70 ft

5.1.6 Reservoir Low-Level Outlet Works

The proposed Reservoir Low-Level Outlet Works (refer to Drawing No. 222-D-50090 through 222-D-50092) is capable of making releases to the existing Rocky Coulee Wasteway and the proposed Rocky Coulee Pumping Plant. The outlet works would be constructed through the dam embankment near the center of the coulee. Features of the outlet works include:

- reinforced concrete trashrack-protected bellmouthed intake structure,
- 108-inch-diameter steel-lined reinforced concrete upstream conduit,
- reinforced concrete gate chamber located at the centerline of the dam that houses a 7-foot, 6-inch by 9-foot hydraulically-operated guard gate,
- downstream 108-inch-diameter steel pipe inside a 15-foot-diameter reinforced concrete horseshoe-shaped adit (tunnel),
- reinforced concrete outlet control structure that houses a 7-foot, 6-inch by 9-foot hydraulically-operated regulating gate,
- reinforced concrete chute, and
- reinforced concrete stilling basin.

The intake structure (Drawing No. 222-D-50091) is a 25-foot-square, reinforced concrete box with two sets of 8-foot-wide trashracks on each of the three sides and on the top of the structure. The purpose of the trashracks is to prevent debris from entering the structure and potentially plugging or damaging the outlet work's gates or conduits. Where this structure ties into the steel-lined conduit, a streamlined bellmouthed intake is formed into the concrete to minimize hydraulic losses. Immediately downstream of the bellmouthed intake, bulkhead gate seats and guides are provided to allow for inspection and maintenance of the upstream conduit and equipment. This feasibility study does not include costs for a bulkhead gate; it was assumed that if this structure were constructed, a bulkhead gate would be procured sometime in the future for inspection and maintenance of the conduit and gates. The future bulkhead gate should be designed for the maximum differential reservoir head.

Approximately 270 feet downstream of the intake structure, a reinforced concrete gate chamber would be constructed within the dam embankment to house a 7-foot, 6-inch by 9-foot hydraulically-operated guard gate. The gate chamber is connected to the downstream face of the dam via a 15-foot-diameter reinforced concrete horseshoe-shaped adit (tunnel). A 108-inch-diameter exposed steel pipe would convey the flow from the guard gate within the gate chamber through the adit to the control structure that is located approximately 320 feet downstream of the gate chamber. The control structure houses a 7-foot, 6-inch by 9-foot hydraulically-operated regulating gate which controls the release of water from the reservoir to the existing Rocky Coulee Wasteway. Approximately 300 feet downstream of the gate chamber, the 108-inch-diameter steel pipe bifurcates so that water may be diverted to the proposed Rocky Coulee Pumping Plant for use during the irrigation season. The pipe connecting the outlet works and the pumping plant is a 108-inch-diameter buried steel pipe. Where this pipe ties into the pumping plant, a 7-foot, 6-inch by 9-foot hydraulically-operated gate would be provided to isolate the pumping plant from the outlet works. The outlet works discharges into a reinforced concrete chute and stilling basin before the water is discharged into the existing wasteway. The purpose of the chute and stilling basin is to dissipate the energy of the water as it exits the outlet works to prevent damage to the waterway.

A diesel engine-generator (EG) set would be provided at the control structure as a standby (backup) power source for essential equipment in the event primary commercial power is unavailable. An automatic transfer switch would be provided with the EG set.

A ventilation system would be provided to supply fresh air to the gate chamber and access adit. A heating and ventilation system would be provided to supply fresh air to the control structure.

Electrical systems would be provided to supply power for operation of the equipment and lighting as needed. SCADA equipment would be provided to connect this facility to the overall project monitoring and control system located in Ephrata, Washington.

The capacity of the outlet works is 2,020 ft^3/s when the reservoir water surface is at the top of active storage which corresponds to a reservoir water surface elevation of 1,291.0 feet.

5.1.7 **Reservoir Inlet/Outlet Channel**

The proposed Reservoir Inlet/Outlet Channel (refer to Drawing No. 222-D-50046 and 222-D-50057) is an unreinforced concrete-lined channel constructed from the existing East Low Canal to the proposed Rocky Coulee Dam and Reservoir. The purpose of the channel is to convey Columbia River water from the East Low Canal to the reservoir for storage during the nonirrigation season. During the irrigation season, the channel would convey the stored water back to the East Low Canal for use by the District. During reservoir filling, the channel is capable of conveying water at a flow rate of 1,060 ft^3/s. At this flow rate, the reservoir would fill in approximately 60 calendar days. During the irrigation season, water is pumped from the reservoir via the proposed Rocky Coulee Pumping Plant (refer to Drawing No. 222-D-50046) which discharges into the channel via the pump discharge structure (refer to Drawing No. 222-D-50054) constructed in the side of the channel. Water from the discharge structure would flow via gravity to the East Low Canal. During the irrigation season, the channel would convey the water at 735 ft^3/s. At this flow rate, it takes approximately 85 calendar days to empty the reservoir. The channel is approximately 74 feet wide by 17 feet deep with side slopes of 1.5 horizontal to 1 vertical. The channel is approximately 6,715 feet long.

Where the channel intersects the dam, a reservoir inlet structure (refer to Drawing No. 222-D-50059) would be constructed to safely pass the flows through the dam to the reservoir. This structure would discharge into a reinforced concrete and rock-lined discharge channel (refer to Drawing No. 222-D-50101) that conveys the water into the reservoir while protecting the right abutment of the dam from erosion as the reservoir is filled. The structure is a reinforced concrete structure that houses two 6-foot by 7-foot motor-operated slide gates that control flows into the reservoir. Once the reservoir is filled, these gates would be closed to prevent water from flowing back into the channel.

Electrical systems would be provided to supply power for operation of the equipment and lighting as needed. SCADA equipment would be provided to connect the inlet structure to the overall project monitoring and control system located in Ephrata, Washington.

5.1.8 **Rocky Coulee Pumping Plant, Substation, and Transmission Line**

The proposed Rocky Coulee Pumping Plant is an eight-unit, 735- ft^3/s pumping plant. It consists of a pre-engineered metal superstructure and a shallow reinforced concrete “bathtub” type substructure (refer to Drawing No. 222-D-50047 through 222-D-50053). The pumping plant is located immediately downstream of the proposed Rocky Coulee Dam on the right side of the coulee looking downstream (refer to Drawing No. 222-D-50046). The layout of the pumping plant service yard is based on the existing site topography, the submergence requirements of the pumping units, the alignment of the steel pipe from the reservoir low-level outlet works, equipment space requirements for the pumping plant and switchyard, access into and around the pumping plant and switchyard for construction vehicles, and access into and around the pumping plant and switchyard for maintenance vehicles. The service yard was built up to accommodate the hydraulic needs of the pumps and to allow for proper drainage purposes. The high point of the service yard was set at elevation 1,212.1 feet. Access to the service yard would be via a new access road constructed from County Road 7 NE.

The layout of the pumping plant is governed by the number, type, and size of the selected pumping units and equipment, the relationship between the electrical and mechanical systems, required clearances to maintain a safe work environment for the operation and maintenance personnel, and handling requirements for the various pieces of equipment during initial installation and subsequent maintenance operations. The pumping plant is separated into two distinct areas—the Unit Bay and the Service Bay. The Unit Bay is that portion of the plant that houses the main pumping units and associated manifold piping, gates, and valves. The Service Bay is located at the end of the plant and provides an area for maintenance of electrical and mechanical equipment as well as access to the plant for maintenance vehicles. The upper Service Bay area has been left clear for these maintenance activities. The lower floor level of the Service Bay is divided into rooms to provide space for miscellaneous equipment, storage for tools and spare parts, and access to the plant sump which is located beneath this level. An electrical equipment gallery room would run the length of the unit bay and would house motor control equipment, motor starters, and switchgear.

The elevation of the bottom floor of the pumping plant was established based on the minimum water surface elevations in the proposed Rocky Coulee Reservoir, the hydraulic losses that occur within the intake pipe, and the required pump submergence that is needed to ensure that the proper suction head is provided so that the pumps operate efficiently. Based on these design parameters, the centerline of the pumps were set at elevation 1,202.08 feet.

The length and width of the Unit Bay is based on the size and arrangement of the pumping units and the required clearances for operation and maintenance of the plant. To minimize the width of the plant, the intake manifold was located beneath the exterior side wall and is encased in reinforced concrete, which forms the base of the side wall. The length and width of the Service Bay is based on the unit sizes and handling requirements between the unit bay and service bay. The electrical equipment gallery was determined by the size and arrangement of the electrical equipment. A mechanical room was placed on the north end of the plant to house the HVAC equipment for the plant based on the size of the required HVAC equipment. The discharge manifold had to be located away from the rest of the plant so as to allow space for installation of flowmeters within the plant. The client requested that individual flowmeters be placed immediately downstream of each pump which resulted in the plant being wider than is normal to accommodate eight individual flowmeters and their associated clearances (10 pipe diameters upstream of the flowmeter and 5 pipe diameters downstream of the flowmeter). Normal Reclamation layout procedures call for a single flowmeter located on the discharge pipeline in a separate structure in the service yard.

The pumping plant has a reinforced concrete substructure approximately 300 feet long by 200 feet wide, and a 300-foot-long by 120-foot-wide structural steel superstructure to house the pumps and crane, with a standing seam metal roof. Handling requirements for the units controlled the building height and the 20-ton overhead bridge crane elevation. The structural steel superstructure framing supports the crane loads and distributes them to the reinforced concrete substructure.

5.1.8.1. Mechanical Equipment

5.1.8.1.1 ***Pumps***

Eight horizontal centrifugal pumps, each rated for 91.9 ft^3/s (41,240 gpm) at 101 feet total head were selected for the pumping plant. The same size pumps were chosen to minimize the spare parts needed. Horizontal synchronous motors rated 1,250 hp at 505 rpm each would be used to drive the pumps. The pumping units operate over reservoir water surface elevations that vary between elevations 1,215.0 and 1,291.0 feet.

Variable frequency drive pumping units were not selected for this plant. The eight fixed-speed units provide sufficient flexibility to control the flow of water into the East Low Canal to meet water demands during the irrigation season.

For this feasibility study, the design does not include an installed spare pumping unit or a spare set of impellers. The design assumed pump materials are ductile or cast iron casings and bronze impellers. Water quality was assumed to be adequate for the selected pump materials.

Table 5- 6. Rocky Coulee pumping unit data

Type and Number of Units	Eight Horizontal Centrifugal (split-case) Pumping Units
Discharge Capacity: At TDH_{Max} of 101 feet	735 ft^3/s (includes 3% wear factor)
Minimum Submergence	18.3 feet
Motors	1,250-hp @ 505 rpm
Intake Manifold Diameter	108-inch
Guard Valve (Intake)	48-inch butterfly
Discharge Manifold Diameter	108-inch
Guard Valve (Discharge)	48-inch butterfly
Check Valve (Discharge)	48-inch tilting disc

Operation of the storage facility involves emptying the proposed Rocky Coulee Reservoir each year to supply additional water to the existing East Low Canal when needed. During the irrigation season, water would be pumped from the reservoir to the pump discharge structure located in the side of the inlet/outlet channel. Discharged water would then flow via gravity from the pump discharge structure to the East Low Canal through the inlet/outlet channel. Pumping would continue until the reservoir is essentially empty. It takes approximately 85 calendar days to empty the reservoir. During the nonirrigation season, the reservoir would be refilled by water supplied from the East Low Canal. This water would flow via gravity through the reservoir inlet/outlet channel into the reservoir. This method of operation would occur each year.

A unique aspect of this pumping facility is the procedure for startup and operation of the pumping units during the irrigation season. At the beginning of the season, the water surface in the existing East Low Canal matches the water surface in the reservoir. Normally, the discharge point for a pumping system is at a higher elevation than the elevation of the water source so that the pumps have a differential pressure against which to pump. In the case of this facility, initially there is no differential head against which to pump. For this reason, the design of the pumping system includes a means of artificially raising the head of the discharge point. This is accomplished via a regulating gate installed within the pumping plant discharge structure. When pumping starts, the regulating gate is opened slightly which results in the buildup of back pressure within the discharge pipeline. As the water surface in the reservoir begins to lower, the regulating gate is gradually opened to maintain the appropriate head differential needed to effectively operate the pumps. This method of operation continues until the regulating gate reaches full open. When the minimum water surface is obtained in the reservoir, the regulating gate would be closed to prevent backflow into the discharge pipeline and the pumps would be shut off. Once the regulating is closed, stoplogs should be installed in the pumping plant discharge structure to relieve any back pressure on the regulating gate due to water in the reservoir inlet/outlet channel.

During final design it is recommended that the design team complete further analysis of the startup procedures of the pumps and the operation of the valves in order to ensure the pump is not outside of its preferable operating region. For this pumping configuration with a potentially wide total dynamic head range at startup, this analysis will need to conform to the Hydraulic Institute standard *Centrifugal and Vertical Pumps for Allowable Operating Range* (ANSI/HI, 1997) of 70 percent to 120 percent of the best efficiency point flow for pumps with specific speeds less than or equal to 4500 (US Units).

5.1.9 Steel Piping and Valves

Steel pipe and valves were selected for the suction and discharge manifolds. The individual steel pipe branches and the main manifold are sized to limit the flow velocity and minimize friction loss.

The suction manifold is a 108-inch-diameter steel manifold that connects to a 108-inch-diameter steel pipe from the reservoir outlet structure with an insulating flanged joint located at the downstream end of the reservoir outlet structure. The 108-inch-diameter steel suction manifold continues into the pumping plant structure where it manifolds into the individual pump suction lines that feed the eight pumping units. Downstream of each pump, the individual pump discharge pipes connect into the single 108-inch-diameter steel discharge manifold. The 108-inch-diameter discharge manifold extends from the pumping plant structure through an insulating flanged joint, where it connects to the 108-inch-diameter steel discharge pipe at another insulating flanged joint. The steel discharge pipe continues up the side of the coulee where it ties into the Pumping Plant Discharge Structure. Steel piping was designed in accordance with American Water Works

Association (AWWA) Manual M11 (AWWA, 2004) and the American Society of Civil Engineers (ASCE) Manuals and Reports on Engineering Practice No. 79 (ASCE, 1993). The minimum plate thickness for handling is calculated in accordance with AWWA recommendations. This minimum thickness is the lesser of d/288 and (d+20)/400 where d is pipe diameter in inches. After fabrication, all piping would be hydrostatically tested to 1.5 times the design pressure. Steel plate used for the manifolds and discharge pipes conforms to ASTM A36. This steel has good weldability and resistance to brittle fracture.

Each individual pump intake line is provided with a motor-operated butterfly valve, which is only to be closed for maintenance on the pump. Each individual pump discharge line is provided with a check valve and a motor-operated discharge butterfly valve. The check valve is utilized during the startup procedure of the pumps and will prevent reverse flow through the pumps during a power outage. The motor-operated maintenance butterfly valve is only to be closed for maintenance on the pump and the check valve.

5.1.9.1. Valves

Each individual pump intake line is provided with a motor-operated butterfly valve. It is only to be closed for maintenance on the pump. Each individual pump discharge line is provided with a check valve and a motor-operated discharge butterfly valve.

5.1.9.1.1 ***Butterfly Valves for Intake and Discharge Manifolds***

The motor butterfly valves are commercially available and manufactured in accordance with AWWA C504 (AWWA, 1994) and are suitable for pressures up to 150 psig. They are intended to serve as shut off valves for preventing flow during maintenance on the pumps and check valves.

5.1.9.1.2 ***Check Valves***

Check valves will prevent reverse flow through the pumps during a power outage. The check valves are equipped with a hydraulically operated dampening device. Upon pump unit shutdown, these valves freely close the first 90 percent of travel; the final 10 percent of travel is controlled by a hydraulic dampening device. The hydraulic dampening device has an adjustable dashpot that can be used to control the time for the last 10 percent of closure. The check valves are rated for 250 psig cold water service.

5.1.9.1.3 ***Air Valves***

There is a combination-type air valve mounted on each individual pump intake pipe, the top of each pumping unit, and on each individual pump discharge pipe. The combination air valves release air at the high points of the piping and pumps. They will also continuously permit the release of air during pump operations. All air valves are rated for 300 psig cold water service. The ball valve provided below each air valve and manifold is for isolation of the air valve to permit air valve maintenance.

5.1.9.2. Auxiliary Mechanical Systems

The auxiliary mechanical systems in the pumping plant consist of a gravity drainage system, fire suppression system, compressed air system, nonpotable service water system, sanitary waste system, cooling water system, and HVAC system.

The gravity drainage system consists of floor drains around the perimeter of the plant interior and in floor areas throughout the building where leakage of water can be expected. Sloped cast iron hub and spigot soil pipe will collect waste water from the floor drains and will convey the water by gravity to the plant sump.

A sump waste oil skimmer assembly with collection drum will be provided to skim any oil sheen from the plant sump water surface to prevent environmental contamination during pumping operations. Vertical turbine sump pumps will be provided to remove accumulated water within the plant sump. It is assumed that the sump pump discharge will be to the service yard drainage trenches without further treatment.

The fire suppression system consists of:

- A wet pipe sprinkler system including fire pump, hose stations in the pumping unit area, and a deluge system for various rooms.

- Portable multipurpose class ABC wall-mounted dry chemical fire extinguishers located at every exterior door and at a maximum distance of 75 feet, and a wheeled dry chemical fire extinguisher, to extinguish fires in flammable materials and equipment fires in the plant.

- A clean agent suppression system for the Electrical Equipment Gallery Room that houses the motor control equipment, motor starters, switchgear and transformers.

Assumptions:

- Water will always be available in the suction tube for the fire protection system.

A compressed air system will be provided in the interior of the pumping plant for use by plant personnel for the operation of pneumatic tools during maintenance activities. The system consists of a stationary vertical receiver tank, two rotary screw air compressors, an air dryer, and steel distribution piping.

A nonpotable service water system is provided for plant maintenance operations and to supply water to the plant evaporative coolers. The plant will be supplied with two self-cleaning strainers and a hydropneumatic tank which is pressurized by an automatic pressure switch controlled small service water pump. Corrosion-resistant copper tubing will supply water to the service water outlets. The service water outlets will be supplied with quick connects distributed throughout the interior of the plant.

The sanitary waste system consists of an incinerating toilet(s) to be provided in the plant.

<u>Assumptions</u>:

- The plant will be an unmanned facility.

A cooling water system will be provided to supply cooling water to the pumping unit motors. The cooling water system consists of water supply pumps, self-cleaning strainers (see Service Water System), exposed noncorrosive copper distribution piping, and embedded ductile iron piping. It is assumed that mechanical seals will not be used on any pumps installed within the plant.

The HVAC system for the plant is designed for equipment protection purposes only, not for occupancy, since this plant is an unmanned facility. Indoor temperatures are assumed to be 95° F and 55° F for summer and winter, respectively.

Plant cooling is via evaporative cooling systems. Sizing of equipment is based upon maximum outdoor climatic conditions and indoor electrical equipment cooling requirements. Pump motors are assumed to be water cooled.

Plant heating will be via electric-type duct heaters. Localized heating is provided throughout the plant via wall-mounted electric unit heaters.

SCADA management of the plant HVAC system is provided to remotely monitor and control the plant HVAC system.

An overhead traveling bridge crane will be provided in the interior of the plant for use during maintenance operations. The crane capacity is based on handling the motor and pump separately. The hoist, trolley, and bridge are electric powered with radio controls.

An 8-path ultrasonic flowmeter will be provided in each pumping unit discharge line to measure the flow of water from each individual pump.

A diesel engine-generator (EG) set is provided as a standby (backup) power source for essential equipment in the event the primary commercial power is unavailable. An automatic transfer switch is provided with the EG set. The EG set is not sized to operate the pumping plant's main pumping units.

5.1.9.3. Air Chamber

Based on hydraulic transient analysis, it was determined that an air chamber would not be required on the discharge pipeline.

5.1.9.4. Electrical Equipment

A 6.9-kV motor bus provides power for the eight squirrel-cage induction motors. The motors are started full-voltage, across-the-line with medium-voltage vacuum-type magnetic motor contactors.

Auxiliary loads, which require 3-phase, 480-volt power, such as air compressors, sump pumps, HVAC equipment, valve or gate motor-operators, etc., are fed from a 600-volt motor control center.

Lighting loads including 120-volt convenience outlets are fed from 208Y/120-volt dry-type transformers and lighting panel boards.

SCADA equipment would be provided to connect the discharge structure to the overall project monitoring and control system located in Ephrata, Washington.

5.1.9.5. Substation and Transmission Line

This plant will be supplied from a separately developed (fenced) open-air, 115-kV substation consisting of a 16 MVA, 115 – 4.16-kV oil-filled power transformer, high-side SF-6 (sulfur hexafluoride) gas-insulated power circuit breaker, and associated disconnect switches.

It was assumed that power for the plant would come from a local utility out of Moses Lake, Washington, which is located approximately 5.6 miles southwest of the plant site. A new transmission line would be constructed to the site. This study assumes that 115-kV service would be available. If 115-kV service is not available at Moses Lake, a 115-kV source of power could come from Grand Coulee, Washington.

5.1.10 **Pumping Plant Discharge Structure**

The proposed Pumping Plant Discharge Structure (refer to Drawing No. 222-D-50054 through 222-D-50056) is a reinforced concrete structure constructed in the side of the Reservoir Inlet/Outlet Channel. The purpose of the structure is to safely discharge the water pumped by the Rocky Coulee Pumping Plant into the channel so that the water can be conveyed via gravity to the East Low Canal during the irrigation season. The structure houses a single 9-foot by 9-foot slide gate which controls the flow of water from the structure into the channel. The outlet of the structure where it ties into the side of the inlet/outlet channel is protected by a trashrack which prevents trash, animals, and people from entering the structure. The top of the structure has been designed to permit vehicles to drive across, since this structure crosses the O&M access road that runs along the side of the channel.

Electrical systems would be provided to supply power for operation of the equipment and lighting as needed. SCADA equipment would be provided to connect the discharge structure to the overall project monitoring and control system located in Ephrata, Washington.

Chapter 6: Cost Estimates

Feasibility-level total project cost estimates (field cost estimates plus noncontract cost estimates) were prepared for the various water delivery and water supply alternatives associated with this Study. This section discusses the various components of these estimates and presents the cost estimates that are based on the feasibility-level designs. Also presented in this section are estimates of the annual Operation, Maintenance, and Replacement (OM&R) costs and annual Power costs for the various alternatives.

6.1. Field Cost Estimates

Field cost estimates for the Study include itemized pay items (includes an allowance for escalation during construction), mobilization, an allowance for design contingencies, an allowance for procurement strategies, and an allowance for construction contingencies. Field cost estimates do not include noncontract costs (e.g., environmental studies, site investigations, design, construction management, legal, security, etc.). Field cost estimates also do not include land acquisition, relocation, or right-of-way costs that may be significant and are required for construction of the project features. Operation, maintenance, and replacement costs are also not included in field cost estimates.

The feasibility-level field cost estimates for construction of the features associated with this feasibility study are summarized in Table 6- 1, Table 6- 2, and Table 6- 3. These field cost estimates are in October 2009 price-level dollars and include the following:

- **Mobilization.** Mobilization costs include mobilizing contractor personnel and equipment to the project site during initial project startup. The assumed 5 percent (+/-) of the subtotal cost used in the cost estimates is based on past experience and bid abstract percentages on similar projects. The mobilization line item is a rounded value per Reclamation rounding criteria which may cause the dollar value to slightly deviate from the actual percentage shown.

- **Design Contingencies.** Design contingencies are intended to account for three types of uncertainties inherent as a project advances from the planning stage through final design which directly affects the estimated cost of the project. These include: (i) unlisted items, (ii) design and scope changes, and (iii) cost estimating refinements. According to the Reclamation Cost Estimating Handbook guidelines, the allowance for feasibility-level design contingencies (formerly referred to as unlisted items) varies between 2 percent and 15 percent. The design contingencies line item is a rounded value per Reclamation rounding criteria which may cause the dollar value to slightly deviate from the actual percentage shown.

- **Allowance for Procurement Strategies (APS).** A line item allowance for procurement strategies (considerations) may be included in an estimate to account for additional costs when solicitations for construction are advertized and awarded under other than full and open competition. These include solicitations that will be set aside under socioeconomic programs, along with solicitations that may limit competition or allow award to other than the lowest bid or proposal. The Study estimates assume full and open competition, receipt of sealed bids, with award to the lowest responsive and responsible bidder, except for the Most Probable High estimate, which includes a 1-percent allowance for APS.

- **Construction Contingencies**. Construction contingencies are considered funds to be used, if needed, after construction starts. These funds are not for design changes made during project planning. The percentage allowance is intended to cover minor differences in actual and estimated quantities, unforeseeable difficulties at the site, changed site conditions, possible minor changes in plans and other uncertainties. The Reclamation Cost Estimating Handbook guidelines suggest approximately 20 percent be added for construction contingencies to feasibility-level cost estimates. The construction contingency line item is a rounded value per Reclamation rounding criteria which may cause the dollar value to slightly deviate from the actual percentage shown.

The designs and feasibility level cost estimates are based on the best available design data provided. The amount of data collected to adequately define major cost drivers and technical adequacy is considered to be at the level required for a feasibility-level assessment of project features. Design data collected for future studies may change future cost estimates significantly from the feasibility-level cost estimates presented in this report.

6.2. Noncontract Costs

Noncontract costs refer to work or services provided in support of the project and other work which is of such a broad nonspecific nature that it can only be attributed to the project as a whole. These costs generally originate for work or services provided by agency personnel (or contractor personnel used to augment agency resources) or land or right-of-way acquisitions to facilitate project development.

- **Land Cost, Rights, and Realty.** Included in this category is the purchase of land, rights-of-way, easements, etc., that are required for the construction of the project.

- **Service Facilities.** Service facilities are those items intended primarily for use in the construction of permanent properties. Camps, construction roads and trails, utility systems, transportation equipment, and most costs of temporary plant used during construction are included under this category.

- **Studies, Investigations, and Design Data Collection.** Included in this category are all appropriate studies/investigations (environmental impact, cultural, archeological, mitigation, etc.), topographic surveys, and design data collection.

- **Engineering Design.** Included in this category are the preparation and review of final designs, construction drawings, specifications, construction cost estimates; procurement activities and similar or related activity, including prevalidation of funds estimates, and Independent Government Cost Estimates. Expenses within this category generally occur before contract award.

- **Construction Management and Contract Administration.** Included in this category are construction management, contract administration, construction inspection, construction surveys, laboratory services (concrete, soil, and other materials testing), construction safety engineering, etc. Expenses within this category generally occur after contract award.

- **Other Costs.** Included in this category are the general expenses incurred after appropriation of funds for construction, not readily identified within studies, surveys, designs and specifications, or construction management, including general office salaries, general office supplies, general office expenses (e.g., rent and utility services), general transportation expenses, security, environmental oversight, mitigation/cultural resources services, legal services, etc.

6.3. Cost-Risk Modeling (Monte Carlo Simulation)

To adequately define the costs required to construct the water delivery alternatives and Rocky Coulee Storage Facility, Reclamation will conduct a Monte Carlo cost-risk simulation to identify cost risk and critical cost drivers for those alternatives.

To prepare the Monte Carlo cost-risk simulation, Reclamation developed the most probable cost estimates for the water delivery alternatives and the Rocky Coulee Storage Facility which are summarized in Table 6- 1, Table 6- 2, and Table 6- 3. Most Probable Low and Most Probable High cost estimates will be prepared for the final Special Study report. These three costs estimates, which include quantity and unit price ranges, help define the potential cost risk associated with possible difficulties with construction due to uncertainty with local conditions. Monte Carlo simulation techniques are applied to the three cost estimates to evaluate the cost risk associated with each line item. The Monte Carlo cost-risk simulation is a statistical analysis randomly generating total costs based on the sum of the distributions of quantities and unit prices generated from the most probable low, most probable, and most probable high cost estimates. The Monte Carlo cost estimates provide a confidence level that a certain project cost estimate will cover the costs associated with construction. The information generated by this analysis will be used by project stakeholders to assess which parts of the project may pose a significant risk to project costs allowing them to focus project resources to mitigate those potential risks identified by the analysis.

The final feasibility report will display the Monte Carlo 0-percent, most probable, and Monte Carlo 100-percent cost estimates generated during the analysis. The Monte Carlo 0-percent cost estimate has a 0-percent probability of not being exceeded, and the Monte Carlo 100-percent cost estimate has a 100-percent probability of not being exceeded. These values are typically between the most probable low and most probable high values summarized in Table 6- 1, Table 6- 2, and Table 6- 3 because the probability that all pay items and allowances will be at their minimums (or maximums) is highly unlikely.

Table 6- 1. Feasibility cost estimates--Water Delivery Alternative 2, Partial Groundwater Irrigation Replacement

	Most Probable Low Cost Estimate	Most Probable Cost Estimate	Most Probable High Cost Estimate
Field Cost Estimates – East Low Canal Enlargement and Extension			
Weber Coulee Siphon Outlet to Lind Coulee Siphon No. 1 Inlet	To Be Determined	$ 123,351,831.00	To Be Determined
Lind Coulee Siphon No. 1 Inlet to Mile Post 72 Check Structure		$ 172,824,900.10	
Mile Post 72 Check Structure to Kansas Prairie Siphon No. 2 Inlet		$ 66,014,070.60	
Kansas Prairie Siphon No. 2 Inlet to Scootenay plus Extension		$ 59,714,308.00	
Subtotal		$ 421,905,109.70	
Mobilization (MPL ~ 5%, MP ~ 5%, MPH ~ 5%)		$ 21,130,000.00	
Escalation to Notice-to-Proceed (None included)		$ 0.00	
Design Contingencies (MPL ~ 8%, MP ~ 10%, MPH ~ 13%)		$ 45,164,890.30	
Allowance for Procurement Strategy (MPL ~ 0%, MP ~ 0%, MPH ~ 1%)		$ 0.00	
Construction Contingencies (MPL ~ 20%, MP ~ 20%, MPH ~ 25%)		$ 99,400,000.00	
Subtotal		$ 165,694,890.30	
Field Cost Total		$ 587,600,000.00	
Noncontract Cost Estimates			
Land Cost, Rights, and Realty	To Be Determined	$ 5,073,172.00	To Be Determined
Service Facilities		$ 5,480,000.00	
Studies, Investigations, and Design Data Collection		$ 16,440,000.00	
Engineering Design		$ 42,470,000.00	
Construction Management and Contract Administration		$ 38,360,000.00	
Other Costs		$ 32,880,000.00	
Noncontract Cost Total		$ 140,703,172.00	
Total project cost estimate (field cost plus non-contract cost estimates)	To Be Determined	$ 728,303,172.00	To Be Determined

Table 6- 2. Feasibility cost estimates--Water Delivery Alternative 3, Full Groundwater Irrigation Replacement

	Most Probable Low Cost Estimate	Most Probable Cost Estimate	Most Probable High Cost Estimate
Field Cost Estimates – East Low Area			
Weber Branch Siphon Outlet to Lind Coulee Siphon No. 1 Inlet	To Be Determined	$ 123,351,831.00	To Be Determined
Lind Coulee Siphon No. 1 Outlet to Mile Post 72		$ 172,824,900.10	
Mile Post 72 to Kansas Prairie Siphon No. 2 Inlet		$ 66,014,070.60	
Kansas Prairie Siphon No. 2 Outlet to Scootenay plus Extension		$ 59,714,308.00	
Subtotal – East Low Area		$ 421,905,109.70	
Field Cost Estimates – East High Area			
East High Canal – Main Canal to Black Rock Coulee Re-Reg Reservoir		$ 485,255,636.30	
East High Canal – Black Rock Coulee to Rocky Branch Siphon Inlet		$ 192,064,328.70	
East High Canal – Rocky Branch Siphon Inlet to Weber Coulee Wasteway		$ 140,586,052.30	
Black Rock Branch Canal – Black Rock Coulee to Weber Coulee Siphon		$ 165,514,816.90	
Black Rock Branch Canal – Weber Coulee Siphon to Farrier Coulee		$ 68,741,315.00	
Wasteway		$ 15,863,241.00	
Black Rock Coulee Dike		$ 79,150.00	
Black Rock Coulee Flood Control Facility @ East Low Canal			
		$ 1,068,104,540.20	
Subtotal – East High Area			
		$ 74,864,000.00	
Mobilization (MPL ~ 5%, MP ~ 5%, MPH ~ 5%)		$ 0.00	
Escalation to Notice-to-Proceed (None included)		$ 155,617,350.10	
Design Contingencies (MPL ~ 8%, MP ~ 10%, MPH ~ 13%)		$ 0.00	
Allowance for Procurement Strategy (MPL ~ 0%, MP ~ 0%, MPH ~ 1%)		$ 351,219,000.00	
Construction Contingencies (MPL ~ 20%, MP ~ 20%, MPH ~ 25%)			
		$ 2,071,710,000.00	
Field Cost Total			
Non-Contract Cost Estimates			
Land Cost, Rights, and Realty	To Be Determined	$ 28,046,356.00	To Be Determined
Service Facilities		$ 19,501,100.00	

Table 6- 2. Feasibility cost estimates--Water Delivery Alternative 3, Full Groundwater Irrigation Replacement

	Most Probable Low Cost Estimate	Most Probable Cost Estimate	Most Probable High Cost Estimate
Studies, Investigations, and Design Data Collection		$ 58,503,300.00	
Engineering Design		$ 151,133,525.00	
		$ 136,507,700.00	
Construction Management and Contract Administration		$ 117,006,600.00	
Other Costs			
Non-Contract Cost Total		$ 510,698,581.00	
Total project cost estimate (field cost plus non-contract cost estimates)	To Be Determined	$ 2,582,408,581.00	To Be Determined

Table 6- 3. Feasibility cost estimates--Water Supply Alternative, Rocky Coulee Storage Facility

	Most Probable Low Cost Estimate	Most Probable Cost Estimate	Most Probable High Cost Estimate
Field Cost Estimates			
Rocky Coulee Dam	To Be Determined	$ 82,690,400.00	To Be Determined
Reservoir Low Level Outlet Structure		$ 13,045,945.00	
Reservoir Inlet/Outlet Channel		$ 4,118,920.00	
Pumping Plant and Discharge Structure		$ 34,509,663.90	
Switchyard and Transmission Lines		$ 3,193,920.00	
Roads and Road Relocations		$ 1,037,112.50	
SCADA System		$ 265,000.00	
Subtotal		$ 138,860,961.40	
Mobilization (MPL ~ 5%, MP ~ 5%, MPH ~ 5%)		$ 6,900,000.00	
Escalation to Notice-to-Proceed (None included)		$ 0.00	
Design Contingencies (MPL ~ 8%, MP ~ 10%, MPH ~ 13%)		$ 14,239,038.60	
Allowance for Procurement Strategy (MPL ~ 0%, MP ~ 0%, MPH ~ 1%)		$ 0.00	
Construction Contingencies (MPL ~ 20%, MP ~ 20%, MPH ~ 25%)		$ 30,000,000.00	
Field Cost Total		$ 190,000,000.00	
Non-Contract Cost Estimates			
Land Cost, Rights, and Realty	To Be Determined	$ 40,586,850.00	To Be Determined
Service Facilities		$ 1,900,000.00	
Studies, Investigations, and Design Data Collection		$ 5,700,000.00	
Engineering Design		$ 13,300,000.00	
Construction Management and Contract Administration		$ 13,300,000.00	
Other Costs		$ 11,400,000.00	
Non-Contract Cost Total		$ 86,186,850.00	
Total project cost estimate (field cost plus non-contract cost estimates)	To Be Determined	$ 276,186,850.00	To Be Determined

6.4. Annual Operations, Maintenance, and Replacement (OM&R) Costs

Feasibility-level annual operations, maintenance, and replacement (OM&R) costs were prepared for the various water delivery and water supply alternatives associated with this Study. The operation and maintenance costs are based on historical data for project lands (Reclamation, 2003) currently served by the East Columbia Basin Irrigation District and represent anticipated annual O&M costs expected for the proposed new project features. Current annual O&M costs are not included in the costs presented in Table 6- 4 below.

The estimated annual replacement costs are based on the most probable field-cost estimates developed for this Study for the proposed project features with appropriate depreciation rates applied. The depreciation rates were taken from the document *Replacements – Units, Service Lives, and Factors*, prepared by the U.S. Department of Energy, Western Area Power Administration and the U.S. Department of the Interior, Bureau of Reclamation (DOE/DOI, 2006).

Table 6- 4. Annual operation, maintenance, and replacement costs

	Annual O&M Costs	Annual Replacement Costs	Total Annual OM&R Costs
East Low Canal – Sta. 1640+00 to Sta. 2903+86.89 (18,790 Acres Served)	$ 1,040,111	$ 947,190	$ 1,987,301
East Low Canal – Sta. 2903+86.89 to Sta. 3778+00.00 (23,272 Acres Served)	$ 1,288,211	$ 1,254,606	$ 2,542,817
East Low Canal – Sta. 3778+00.00 to Sta. 4475+85.95 (11,283 Acres Served)	$ 624,567	$ 572,505	$ 1,197,072
East Low Canal – Sta. 4475+85.95 to Sta. 4713+61.00 (10,961 Acres Served)	$ 606,746	$ 308,982	$ 915,728
East High Canal – Sta. 0+00 to Sta.1311+00 (7,403 Acres Served)	$ 459,144	$ 1,827,740	$ 2,286,884
East High Canal – Sta. 1333+00 to Sta. 2055+08 (16,186 Acres Served)	$ 1,003,878	$ 986,046	$ 1,989,924
East High Canal – Sta. 2055+08 to Sta. 2670+57 (11,413 Acres Served)	$ 707,850	$ 598,306	$ 1,306,156
Black Rock Branch Canal – Sta. 0+00 to Sta. 988+26 (13,012 Acres Served)	$ 807,022	$ 1,527,608	$ 2,334,630
Black Rock Branch Canal – Sta. 988+26 to Sta. 1525+70 (7,887 Acres Served)	$ 489,163	$ 443,182	$ 932,345
Rocky Coulee Storage Facility	$ 200,000	$ 794,718	$ 994,718

6.5. Annual Power Costs

Feasibility-level annual power costs were prepared for the various water delivery and water supply alternatives associated with this feasibility study. These power costs are based on estimates of power usage for the various pumping plants, buildings, canal structures, and dam facilities that comprise the proposed project features. These estimates assume that project power would be the source of power for these features and at the time of preparation these costs were set at 2.2 mills per information provided by Reclamation's Ephrata Field Office, Ephrata, Washington.

Table 6- 5. Annual power costs

	Annual Power Costs
East Low Canal – Sta. 1640+00 to Sta. 2903+86.89 (18,790 Acres Served)	$ 65,756
East Low Canal – Sta. 2903+86.89 to Sta. 3778+00.00 (23,272 Acres Served)	$ 143,814
East Low Canal – Sta. 3778+00.00 to Sta. 4475+85.95 (11,283 Acres Served)	$ 36,702
East Low Canal – Sta. 4475+85.95 to Sta. 4713+61.00 (10,961 Acres Served)	$ 8,817
East High Canal – Sta. 0+00 to Sta.1311+00 (7,403 Acres Served)	$ 30,286
East High Canal – Sta. 1333+00 to Sta. 2055+08 (16,186 Acres Served)	$ 36,766
East High Canal – Sta. 2055+08 to Sta. 2670+57 (11,413 Acres Served)	$ 14,729
Black Rock Branch Canal – Sta. 0+00 to Sta. 988+26 (13,012 Acres Served)	$ 95,498
Black Rock Branch Canal – Sta. 988+26 to Sta. 1525+70 (7,887 Acres Served)	$ 4,519
Black Rock Coulee Pumping Plant No. 1	$ 66,179
Rocky Coulee Storage Facility	$ 31,355

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Drawings

Appendix A - General Canal Design Flowchart

GENERAL CANAL DESIGN FLOW CHARTS (Sheet 1 of 4)

CANALS AND LATERALS (ENGLISH)

LINING

COMPACTED EARTH

Lining Material
P.I.=12 MIN., L.L.=45 Max

Use only for Q>50 cfs

THICKNESS OF EARTH LINING

d	t_1	t_2
2.0 or less	1.0	3.0
2.0 to 4.0	1.5	4.0
4.0 to 6.0	2.0	6.0
6.0 to 20.0	2.0	8.0
> 20.0	3.0	8.0

FREEBOARD FOR EARTH LINING

Q	H
≤ 200	0.5
200 to 300	(0.45 log Q)−0.50
300 to 20,000	(0.83 log Q)−1.48

For Explanation of Symbols see Figure 4

Note: These charts may be used for general layout and/or design, but engineering judgement should be used if unique or special conditions arise

UNREINFORCED CONCRETE

THICKNESS OF CONCRETE LINING

Q	t
0 to 500	2½
500 to 1500	3
1500 to 3500	3½
3500 to 7500	4
7500 to 20,000	4½

CONTRACTION JOINT SPACING

Transverse		Longitudinal	
t	C−C	P	C−C
2½	10'	< 20'	None
3	12'	≥ 20'	Maximum spacing same as Transv. C−C
3½	14'		
4	15'		
4½	15'		

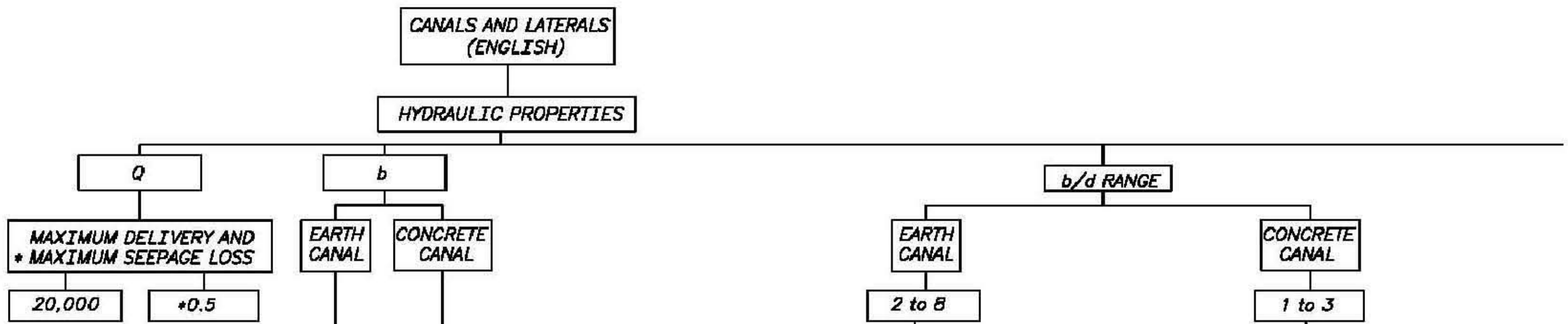
WIDTH OF LIP (l)

Q	L
0 to 30	4
30 to 300	6
>300	8

FREEBOARD FOR CONCRETE LINING

Q	H
≤ 30	0.5
30 to 100	0.34 log Q
100 to 400	(0.78 log Q)−0.90
400 to 20,000	(1.15 log Q)−1.85

GENERAL CANAL DESIGN FLOW CHARTS (Sheet 2 & 3 of 4)



Q	b
50 to 100	6.64(log Q)−5.3
100 to 250	10.1(log Q)−12.2
250 to 1000	23.3(log Q)−43.9
1000 to 2500	47.7(log Q)−117.1
2500 to 5000	66.4(log Q)−180.8
5000 to 7500	114(log Q)−357
7500 to 10,000	120(log Q)−380
10,000 to 15,000	199(log Q)−696
15,000 to 20,000	280(log Q)−1035

Q	b
≦ 4	1.0
4 to 15	2.0
15 to 30	3.0
30 to 60	4.0
60 to 100	5.0
100 to 250	6.0
250 to 800	3.96(log Q)−3.5
800 to 2500	14.1(log Q)−32.9
2500 to 4500	39.2(log Q)−118.3
4500 to 7500	67.6(log Q)−222
7500 to 10,000	80(log Q)−270
10,000 to 15,000	142(log Q)−518
15,000 to 20,000	200(log Q)−760

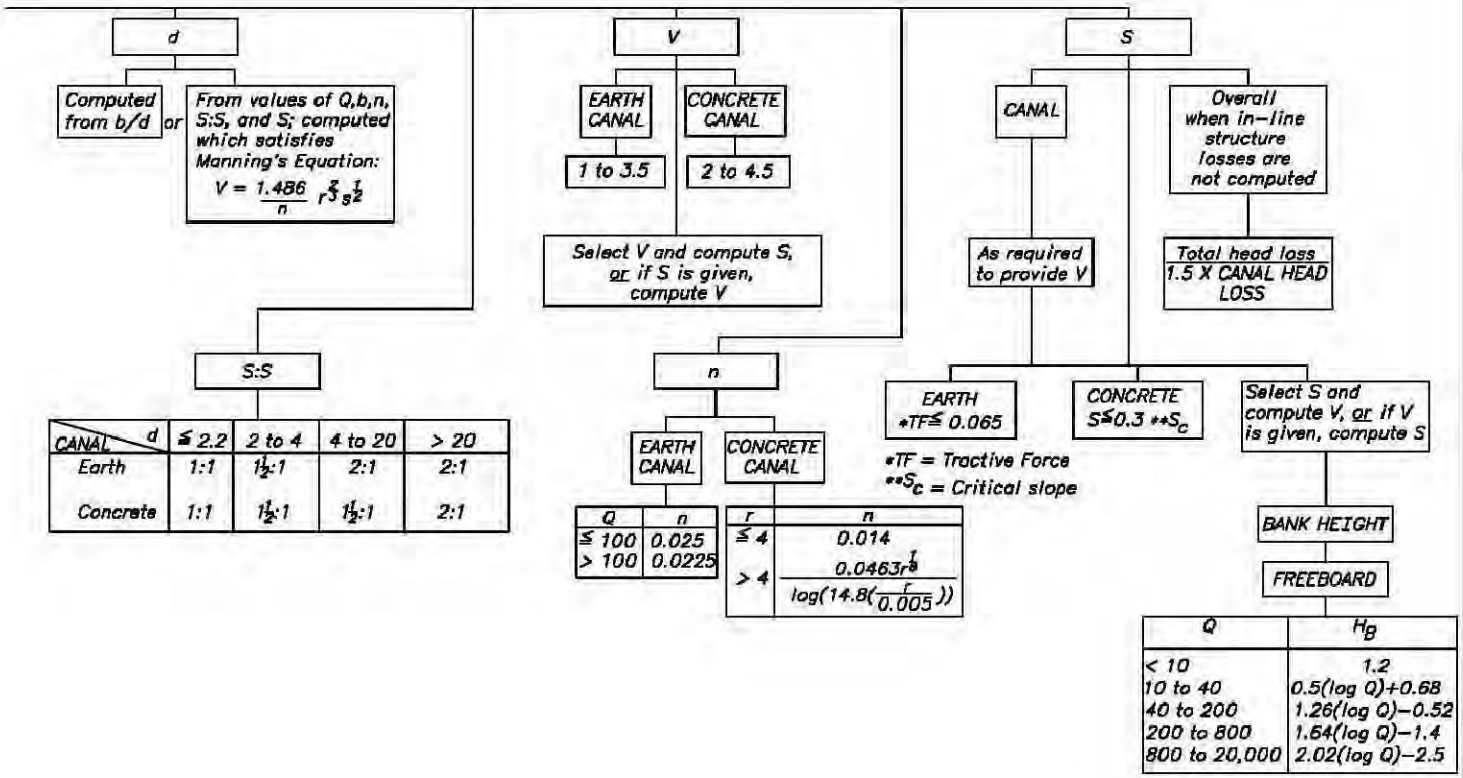
Q	b/d Minimum
50 to 100	2
100 to 500	2.5
500 to 1000	3
1000 to 2500	3.5
2500 to 5000	4
5000 to 7500	5
7500 to 10,000	6
10,000 to 15,000	7
15,000 to 20,000	8

For Explanation of Symbols see Figure 4

Q	b/d Minimum
0.5 to 25	1.0
25 to 500	1.2
500 to 2500	1.3
2500 to 5000	1.5
5000 to 7500	1.8
7500 to 10,000	2.0
10,000 to 15,000	2.5
15,000 to 20,000	3.0

*Maximum allowable seepage loss before lining is required (cfs/sq. ft/day)

Note: These charts may be used for general layout and/or design, but engineering judgement should be used if unique or special conditions arise


d
Computed from b/d
or
From values of Q,b,n, S:S, and S; computed which satisfies Manning's Equation:
$V = \frac{1.486}{n} r^{\frac{2}{3}} s^{\frac{1}{2}}$
S:S
CANAL
d
≤ 2.2
2 to 4
4 to 20
> 20
Earth
1:1
$1\frac{1}{2}$:1
2:1
2:1
Concrete
1:1
$1\frac{1}{2}$:1
$1\frac{1}{2}$:1
2:1
V
EARTH CANAL
CONCRETE CANAL
1 to 3.5
2 to 4.5
Select V and compute S, or if S is given, compute V
n
EARTH CANAL
CONCRETE CANAL
Q
n
≤ 100
0.025
> 100
0.0225
r
n
≤ 4
0.014
> 4
$\frac{0.0463r^{\frac{1}{6}}}{\log(14.8(\frac{r}{0.005}))}$
S
CANAL
Overall when in-line structure losses are not computed
As required to provide V
Total head loss 1.5 X CANAL HEAD LOSS
EARTH *TF≤ 0.065
CONCRETE S≤0.3 **S_c
Select S and compute V, or if V is given, compute S
*TF = Tractive Force
**S_c = Critical slope
BANK HEIGHT
FREEBOARD
Q
H_B
< 10
10 to 40
40 to 200
200 to 800
800 to 20,000
1.2
0.5(log Q)+0.68
1.26(log Q)−0.52
1.64(log Q)−1.4
2.02(log Q)−2.5

GENERAL CANAL DESIGN FLOW CHARTS (Sheet 4 of 4)

CANALS AND LATERALS (ENGLISH)

O & M ROADS

NUMBER

Q	* Number
≤ 100	1
> 100	2

*The number of roads usually increases to 2 when cleaning and maintenance can no longer be accomplished from one side.

WIDTH

Q	W
< 100	12
100 to 500	14
500 to 1000	16
1000 to 1500	18
1500 to 7500	20
> 7500	24

SURFACING

**Type	t_s
Gravel	6

**If acceptable no special surfacing need be applied. On a large canal, it may be desirable to pave the road on the operations side of the canal.

Explanation of Symbols:

- d = Normal depth (ft)
- NWS = Normal water surface
- t = Thickness of concrete lining (inches)
- t_1 = Vertical thickness of earth lining at invert (ft)
- t_2 = Horizontal thickness of earth lining along sideslopes (ft)
- b = Canal bottom width (ft)
- Q = Canal design capacity (cfs)
- V = Canal velocity (ft/sec)
- r = Hydraulic radius (ft)
- n = Mannings "n"
- S = Energy slope
- S_c = Critical engergy slope
- S:S = Canal sideslopes (Horizontal/Vertical)
- H_E = Height of earth lining above NWS (ft)
- H_C = Height of concrete lining above NWS (ft)
- H_B = Height of bank above NWS (ft)
- L = Width of lining lip (ft)
- P = Perimeter of concrete lining (ft)
- C–C = Center to center spacing of contraction joints
- TF = Tractive Force (62.4)(d)(s)
- W = Width of O and M Road (ft)
- t_s = Thickness of gravel surfacing (inches)
- L.L. = Liquid Limit
- P.I. = Plasticity Index

Appendix B - Field Unit Delivery Data

East Low Canal Lateral Deliveries

East Low - Pipe Lateral 47					
FIELD (number)	***Irrigation Category***	***Field (Acres)***	***Field Q (ft^3/s)***	***Delivery Q (ft^3/s)***	***Delivery Name***
765	G	118.0	2.2		
764	G	44.4	0.8		
763	G	76.1	1.4		
830	G	18.7	0.4		
829	G	13.1	0.2		
769	G	10.0	0.2		
768	G	91.6	1.7	7.04	EL47A-D7
767	G	143.2	2.7		
766	G	207.4	3.9		
770	G	40.4	0.8		
890	G	36.4	0.7	8.09	EL47A-D6
889	G	149.4	2.8		
787	G	181.4	3.4		
790	G	63.4	1.2		
789	G	9.8	0.2		
788	G	61.1	1.2	8.81	EL47A-D5
1000	G	112.4	2.1		
978	G	61.7	1.2		
979	G	78.7	1.5		
792	G	82.1	1.6	6.34	EL47A-D4
791	G	85.6	1.6		
786	G	75.3	1.4	3.05	EL47A-D3
1360	G	19.0	0.4		
284	G	136.0	2.6		
233	G	160.9	3.0	5.98	EL47A1-D2
780	G	45.8	0.9		
285	G	141.7	2.7		
304	G	136.1	2.6		
283	G	9.8	0.2	6.31	EL47A1-D1
784	G	137.0	2.6		
783	G	133.8	2.5		
782	G	24.1	0.5		
781	G	137.9	2.6		
779	G	137.4	2.6	10.80	EL47A-D2

East Low - Pipe Lateral 47					
FIELD (number)	***Irrigation Category***	***Field (Acres)***	***Field Q (ft^3/s)***	***Delivery Q (ft^3/s)***	***Delivery Name***
775	G	129.3	2.4		
774	G	132.7	2.5		
773	G	37.4	0.7	5.67	EL47A-D1
1057	G	184.90	3.5		
1058	G	185.85	3.5		
1059	G	180.34	3.4		
1060	G	177.95	3.4		
888	G	175.25	3.3	17.13	EL47B-D6
1441	G	134.90	2.6		
1391	G	127.65	2.4		
1392	G	135.01	2.6	7.53	EL47B3-D1
762	G	123.24	2.3		
759	G	136.41	2.6		
1088	G	133.35	2.5		
757	G	137.45	2.6	10.05	EL47B-D5
755	G	135.27	2.6		
754	G	130.76	2.5	5.04	EL47B2-D1
758	G	137.47	2.6		
981	G	129.96	2.5		
756	G	132.25	2.5		
752	G	134.86	2.6	10.12	EL47B-D4
753	G	133.71	2.5		
749	G	135.28	2.6		
980	G	128.99	2.4		
750	G	131.36	2.5	10.02	EL47B-D3
751	G	18.13	0.3		
748	G	135.31	2.6		
747	G	138.46	2.6	5.53	EL47B-D2
772	G	144.94	2.7		
771	G	140.62	2.7		
998	G	4.90	0.1		
778	G	96.41	1.8		

East Low - Pipe Lateral 47					
FIELD (number)	***Irrigation Category***	***Field (Acres)***	***Field Q (ft^3/s)***	***Delivery Q (ft^3/s)***	***Delivery Name***
777	G	22.33	0.4	7.75	EL47B1-D1
995	G	137.02	2.6		
887	G	22.39	0.4		
776	G	137.95	2.6		
997	G	34.72	0.7		
996	G	20.82	0.4		
994	G	4.84	0.1		
785	G	4.59	0.1		
1008	G	25.26	0.5		
1401	G	32.69	0.6	7.96	EL47B-D1

East Low - Pipe Lateral 53					
FIELD (number)	***Irrigation Category***	***Field (Acres)***	***Field Q (ft^3/s)***	***Delivery Q (ft^3/s)***	***Delivery Name***
520	G	131.0	2.5		
532	G	130.6	2.5	5.0	EL53B-D11
430	G	126.8	2.4		
427	G	131.2	2.5	4.9	EL53B-D10
533	G	107.8	2.0		
534	G	130.4	2.5		
1302	G	50.8	1.0	5.5	EL53B5-D1
426	G	128.8	2.4		
424	G	125.8	2.4		
1329	G	122.9	2.3	7.1	EL53B-D9
425	G	128.1	2.4		
1055	G	130.4	2.5		
914	G	123.2	2.3	7.2	EL53B-D8
1099	G	132.7	2.5		
803	G	132.5	2.5		
1100	G	131.2	2.5		
800	G	130.9	2.5	10.0	EL53B4-D5
802	G	132.5	2.5		
801	G	131.5	2.5	5.0	EL53B4-D4
797	G	132.4	2.5		
796	G	132.4	2.5		
798	G	132.6	2.5	7.5	EL53B4-D3
1056	G	127.6	2.4		

East Low - Pipe Lateral 53					
FIELD (number)	***Irrigation Category***	***Field (Acres)***	***Field Q (ft^3/s)***	***Delivery Q (ft^3/s)***	***Delivery Name***
795	G	137.8	2.6		
1405	G	119.8	2.3	7.3	EL53B4-D2
521	G	134.5	2.5		
423	G	130.7	2.5		
869	G	125.0	2.4		
422	G	124.1	2.4	9.7	EL53B4-D1
1393	G	133.3	2.5		
793	G	132.5	2.5		
799	G	131.5	2.5		
794	G	130.1	2.5	10.0	EL53B3-D3
760	G	132.3	2.5		
928	G	132.2	2.5		
927	G	128.5	2.4		
761	G	129.1	2.4	9.9	EL53B3-D2
1053	G	130.9	2.5		
866	G	130.4	2.5		
868	G	121.5	2.3	7.2	EL53B3-D1
525	G	126.4	2.4		
867	G	24.3	0.5		
1054	G	128.1	2.4		
524	G	128.1	2.4	7.7	EL53B-D7
1314	G	122.5	2.3		
523	ACC	126.6	2.4		
526	G	127.0	2.4	7.1	EL53B-D6
529	G	128.4	2.4		
527	G	123.5	2.3		
528	G	125.7	2.4		
530	G	22.0	0.4		
531	G	122.4	2.3	9.9	EL53B2-D1
522	ACC	135.4	2.6		
654	G	131.3	2.5		
655	G	130.3	2.5	7.5	EL53B-D5
656	G	21.7	0.4		
657	G	130.4	2.5		
659	G	21.7	0.4		
669	G	131.5	2.5	5.8	EL53B-D4
658	G	130.7	2.5		
660	G	133.5	2.5	5.0	EL53B-D3

East Low - Pipe Lateral 53					
FIELD (number)	***Irrigation Category***	***Field (Acres)***	***Field Q (ft^3/s)***	***Delivery Q (ft^3/s)***	***Delivery Name***
1085	G	30.6	0.6		
1052	G	81.0	1.5		
1086	G	30.9	0.6		
535	G	129.2	2.4	5.1	EL53B1-D5
536	G	129.1	2.4		
917	G	134.2	2.5	5.0	EL53B1-D4
648	G	137.0	2.6		
647	G	111.4	2.1		
649	G	22.7	0.4		
646	G	155.0	2.9	8.1	EL53B1-D3
968	G	33.6	0.6		
653	G	133.7	2.5		
967	G	66.8	1.3	4.4	EL53B1-D2
652	G	130.9	2.5		
651	G	75.6	1.4		
650	G	132.0	2.5	6.4	EL53B1-D1
662	G	21.7	0.4		
668	G	132.6	2.5		
915	G	22.8	0.4		
661	G	132.1	2.5	5.9	EL53B-D2
667	G	133.7	2.5		
663	G	132.5	2.5		
665	G	22.6	0.4		
664	G	132.1	2.5		
666	G	131.2	2.5	10.5	EL53B-D1
746	G	133.7	2.5		
745	G	132.9	2.5		
744	G	22.4	0.4		
743	G	134.4	2.5		
742	G	132.9	2.5	10.5	EL53A-D2
737	G	134.4	2.5		
736	G	132.0	2.5		
735	G	17.6	0.3		
734	G	132.2	2.5	7.9	EL53A-D1
1390	G	148.8	2.8		
741	G	119.5	2.3		
740	G	58.9	1.1	6.2	EL53A1-D2
739	G	81.8	1.5		
738	WSC	91.6	1.7	3.3	EL53A1-D1

East Low - Pipe Lateral 65					
FIELD (number)	***Irrigation Category***	***Field (Acres)***	***Field Q (ft^3/s)***	***Delivery Q (ft^3/s)***	***Delivery Name***
265	WSCG	138.0	2.6		
266	WSCG	43.6	0.8	3.4	EL65-D3
392	G	93.6	1.8	1.8	EL65-D2
264	WSCG	6.2	0.1		
322	WSCG	141.1	2.7		
321	WSCG	140.7	2.7	5.5	EL65A-D1
276	G	58.4	1.1		
275	G	60.0	1.1	2.2	EL65-D1

East Low - Pipe Lateral 68					
FIELD (number)	***Irrigation Category***	***Field (Acres)***	***Field Q (ft^3/s)***	***Delivery Q (ft^3/s)***	***Delivery Name***
413	G	130.3	2.5		
415	G	129.6	2.5		
412	G	129.5	2.5		
414	G	18.6	0.4		
411	G	130.8	2.5	10.2	EL68B-D14
419	G	127.5	2.4		
901	G	128.7	2.4		
421	G	19.8	0.4	5.2	EL68B-D13
420	G	133.5	2.5		
417	G	86.3	1.6		
902	G	128.4	2.4		
416	G	128.5	2.4	9.0	EL68B-D12
418	G	75.5	1.4	1.4	EL68B-D11
478	G	128.7	2.4		
479	G	133.4	2.5		
476	G	133.0	2.5		
904	G	22.7	0.4	7.9	EL68B-D10
431	G	127.8	2.4		
460	G	129.5	2.5	4.9	EL68B15-D5
458	G	126.8	2.4		
457	G	130.9	2.5		
459	G	22.1	0.4		
456	G	131.3	2.5	7.8	EL68B15-D4
462	G	133.3	2.5		
461	G	131.5	2.5		

East Low - Pipe Lateral 68					
FIELD (number)	***Irrigation Category***	***Field (Acres)***	***Field Q (ft^3/s)***	***Delivery Q (ft^3/s)***	***Delivery Name***
463	G	132.2	2.5		
903	G	123.9	2.3	9.9	EL68B15-D3
472	G	128.4	2.4		
475	G	129.5	2.5	4.9	EL68B15-D2
471	G	131.8	2.5		
474	G	23.1	0.4		
473	G	132.3	2.5	5.4	EL68B15-D1
518	G	20.8	0.4		
519	G	104.9	2.0		
516	G	131.4	2.5	4.9	EL68B14-D4
517	G	109.9	2.1		
1383	G	126.5	2.4	4.5	EL68B14-D3
513	G	77.7	1.5		
515	G	130.7	2.5		
504	G	43.5	0.8	4.8	EL68B14-D2
505	G	34.0	0.6		
477	G	81.7	1.5	2.2	EL68B14-D1
507	G	130.0	2.5		
506	G	68.4	1.3		
508	G	124.5	2.4	6.1	EL68B-D9
455	G	130.5	2.5		
470	G	130.2	2.5		
454	G	132.3	2.5		
469	G	131.7	2.5	9.9	EL68B13-D3
468	G	22.4	0.4		
466	G	96.2	1.8		
465	G	42.8	0.8		
467	G	130.1	2.5	5.5	EL68B13-D2
497	G	72.6	1.4		
498	G	53.5	1.0		
464	G	129.7	2.5		
496	G	69.2	1.3	6.2	EL68B13-D1
500	G	130.9	2.5		
501	G	24.8	0.5		
1380	G	129.8	2.5		
499	G	128.5	2.4		
1298	G	129.2	2.4	10.3	EL68B12-D1
487	G	119.1	2.3		

East Low - Pipe Lateral 68

FIELD (number)	Irrigation Category	Field (Acres)	Field Q (ft^3/s)	Delivery Q (ft^3/s)	Delivery Name
486	G	23.9	0.5		
485	G	120.7	2.3		
488	G	120.6	2.3		
484	G	118.3	2.2	9.5	EL68B11-D2
503	G	124.2	2.4		
1047	G	11.6	0.2		
502	G	128.6	2.4	5.0	EL68B11-D1
510	G	128.6	2.4		
511	G	125.2	2.4	4.8	EL68B-D8
480	G	121.6	2.3		
483	G	119.8	2.3	4.6	EL68B10-D2
1446	G	56.2	1.1		
1447	G	30.8	0.6		
491	G	22.6	0.4		
1046	G	117.2	2.2	4.3	EL68B10-D1
906	G	132.9	2.5		
612	G	125.5	2.4	4.9	EL68B-D7
1377	G	122.7	2.3		
494	G	120.7	2.3	4.6	EL68B9-D2
489	G	140.6	2.7		
495	G	136.8	2.6		
490	G	136.5	2.6	7.8	EL68B9-D1
616	G	123.8	2.3		
615	G	132.2	2.5		
613	G	127.9	2.4		
614	G	130.7	2.5	9.7	EL68B-D6
1388	G	130.8	2.5		
1442	G	66.3	1.3		
1443	G	18.3	0.3	4.1	EL68B8.2-D2
1444	G	127.7	2.4		
1387	G	139.2	2.6		
632	G	139.7	2.6	7.7	EL68B8.2-D1
629	G	137.6	2.6		
628	G	137.7	2.6		
626	G	136.7	2.6		
627	G	22.2	0.4		

East Low - Pipe Lateral 68					
FIELD (number)	***Irrigation Category***	***Field (Acres)***	***Field Q (ft^3/s)***	***Delivery Q (ft^3/s)***	***Delivery Name***
625	G	137.1	2.6	10.8	EL68B8.1-D1
1315	G	33.3	0.6		
1316	G	29.8	0.6		
870	G	34.0	0.6	1.8	EL68B8-D3
1084	G	140.8	2.7		
630	G	147.5	2.8		
1083	G	136.0	2.6		
631	G	139.1	2.6	10.7	EL68B8-D2
617	G	141.5	2.7		
620	G	140.0	2.7		
618	G	141.2	2.7		
619	G	141.8	2.7	10.7	EL68B8-D1
622	G	136.9	2.6		
911	G	136.2	2.6		
623	G	23.0	0.4		
621	G	137.2	2.6		
624	G	137.5	2.6	10.8	EL68B-D5
909	G	136.4	2.6		
1039	G	135.3	2.6		
910	G	135.9	2.6	7.7	EL68B7-D1
1404	ACC	123.2	2.3		
1301	ACC	125.2	2.4		
1081	ACC	133.3	2.5		
1300	ACC	122.4	2.3	9.5	EL68B6-D3
1403	ACC	128.9	2.4		
929	G	133.4	2.5		
1040	ACC	132.6	2.5		
1336	G	134.8	2.6	10.0	EL68B6-D2
1038	G	24.5	0.5		
912	G	134.1	2.5		
913	G	127.6	2.4	5.4	EL68B6-D1
634	G	136.3	2.6		
633	G	136.4	2.6	5.2	EL68B-D4
989	G	131.0	2.5		
1072	G	135.7	2.6	5.1	EL68B4-D1

East Low - Pipe Lateral 68					
FIELD (number)	***Irrigation Category***	***Field (Acres)***	***Field Q (ft^3/s)***	***Delivery Q (ft^3/s)***	***Delivery Name***
592	G	140.2	2.7		
604	G	139.1	2.6		
593	G	140.5	2.7		
602	G	140.0	2.7		
603	G	21.1	0.4	11.0	EL68B5-D3
600	G	138.4	2.6		
610	G	137.7	2.6		
601	G	140.1	2.7		
605	G	137.7	2.6		
606	G	21.4	0.4	10.9	EL68B5-D2
611	G	144.2	2.7		
609	G	137.9	2.6		
608	G	138.7	2.6		
607	G	135.6	2.6	10.5	EL68B5-D1
636	G	138.1	2.6		
1294	G	119.2	2.3		
635	G	137.1	2.6		
1321	G	124.1	2.3	9.8	EL68B-D3
1069	ACC	130.3	2.5		
641	WSC	135.6	2.6		
991	G	122.7	2.3		
963	G	121.9	2.3	9.7	EL68B3-D3
1005	G	140.9	2.7		
1006	G	140.6	2.7		
1007	G	20.2	0.4		
936	WSC	134.8	2.6		
937	G	135.2	2.6	10.8	EL68B3-D2
931	G	51.1	1.0		
930	G	58.0	1.1		
933	G	167.8	3.2		
934	G	81.1	1.5	6.8	EL68B3-D1
1318	G	135.0	2.6		
1317	G	133.4	2.5	5.1	EL68B2-D2
1074	G	139.9	2.6		
1073	G	138.1	2.6		
1408	G	133.7	2.5		
1407	G	131.7	2.5	10.3	EL68B2-D1
644	G	137.2	2.6		

East Low - Pipe Lateral 68

FIELD (number)	Irrigation Category	Field (Acres)	Field Q (ft^3/s)	Delivery Q (ft^3/s)	Delivery Name
274	G	136.7	2.6		
272	G	137.5	2.6	7.8	EL68B1-D2
1063	G	137.2	2.6		
1062	G	134.8	2.6		
1064	G	137.4	2.6		
1061	G	137.6	2.6	10.4	EL68B1-D1
919	G	139.0	2.6		
956	G	89.9	1.7	4.3	EL68B-D2
960	WSC	132.0	2.5		
1003	WSCG	24.0	0.5		
696	WSCG	131.4	2.5		
959	G	139.9	2.7		
958	G	136.1	2.6	10.7	EL68B-D1
271	WSC	103.1	2.0		
267	WSCG	114.0	2.2		
270	WSCG	160.4	3.0		
268	WSCG	159.3	3.0	10.2	EL68A-D5
726	G	148.4	2.8	2.8	EL68A-D4
723	G	175.0	3.3		
1002	G	182.4	3.5		
1001	G	170.4	3.2	10.0	EL68A-D3
720	G	147.5	2.8		
724	G	108.5	2.1		
722	G	32.8	0.6		
990	G	35.7	0.7	6.1	EL68A-D2
710	G	59.3	1.1		
708	G	78.1	1.5		
709	G	118.5	2.2	4.8	EL68A-D1

East Low - Pipe Lateral 75

FIELD (number)	Irrigation Category	Field (Acres)	Field Q (ft^3/s)	Delivery Q (ft^3/s)	Delivery Name
932	G	27.3	0.5		
674	G	138.5	2.6		
639	G	135.5	2.6		
640	G	22.0	0.4		
637	WSCG	133.6	2.5	8.7	Delivery Point
638	G	136.5	2.6		
642	WSC	143.2	2.7		

East Low - Pipe Lateral 75					
FIELD (number)	***Irrigation Category***	***Field (Acres)***	***Field Q (ft^3/s)***	***Delivery Q (ft^3/s)***	***Delivery Name***
701	G	140.0	2.7		
688	WSCG	156.8	3.0	10.9	Delivery Point
697	WSCG	137.6	2.6		
699	G	21.3	0.4		
700	WSCG	139.9	2.7		
704	WSCG	136.3	2.6	8.2	Delivery Point
693	G	26.3	0.5		
694	WSCG	120.8	2.3		
692	G	51.9	1.0	3.8	Delivery Point
689	G	145.0	2.7		
691	G	23.0	0.4	3.2	Delivery Point
703	G	124.6	2.4		
687	G	69.8	1.3	3.7	Delivery Point
698	G	141.0	2.7		
702	G	30.9	0.6		
1017	G	35.1	0.7		
961	G	7.6	0.1		
1400	G	9.4	0.2		
1399	G	9.2	0.2	4.4	Delivery Point
1022	G	109.2	2.1		
707	G	72.4	1.4		
706	G	78.3	1.5	4.9	Delivery Point 3B
705	G	65.8	1.2		
690	G	92.2	1.7	3.0	Delivery Point 3A
685	G	2.2	0.0		
686	G	78.7	1.5		
1018	G	130.8	2.5	4.0	Delivery Point

East Low - Pipe Lateral 80					
FIELD (number)	***Irrigation Category***	***Field (Acres)***	***Field Q (ft^3/s)***	***Delivery Q (ft^3/s)***	***Delivery Name***
432	G	128.2	2.4		
433	G	93.7	1.8	4.2	EL80A-D10
434	G	131.5	2.5		
886	G	127.6	2.4		
436	G	129.4	2.5		
435	G	137.0	2.6	10.0	EL80A-D9
440	G	124.9	2.4	2.4	EL80A-D8
880	G	125.5	2.4		
879	G	121.6	2.3		
882	G	124.7	2.4		
883	G	47.5	0.9	7.9	EL80A8-D2
881	G	37.5	0.7		
442	G	117.9	2.2		
1045	G	37.7	0.7		
441	G	118.5	2.2	5.9	EL80A8-D1
443	G	126.4	2.4	2.4	EL80A-D7
550	G	128.1	2.4		
556	G	127.2	2.4		
549	G	124.3	2.4		
559	G	125.5	2.4	9.6	EL80A-D6
555	G	122.6	2.3		
885	G	123.1	2.3		
551	G	127.9	2.4	7.1	EL80A7-D1
554	G	128.9	2.4		
552	G	128.6	2.4		
553	G	127.7	2.4	7.3	EL80A6-D1
557	G	126.0	2.4		
567	G	125.4	2.4		
558	G	128.4	2.4		
565	G	118.3	2.2	9.4	EL80A-D5
548	G	122.6	2.3		
545	G	128.4	2.4		
544	G	117.5	2.2	7.0	EL80A5-D1
564	G	126.0	2.4		
561	G	121.8	2.3		
560	G	118.2	2.2		
1082	G	70.7	1.3	8.3	EL80A5-D2

East Low - Pipe Lateral 80

FIELD (number)	Irrigation Category	Field (Acres)	Field Q (ft^3/s)	Delivery Q (ft^3/s)	Delivery Name
566	ACC	127.4	2.4		
1027	G	137.1	2.6		
1326	G	91.2	1.7		
1325	G	78.8	1.5		
589	ACC	95.2	1.8	10.0	EL80A-D4
572	G	122.0	2.3		
547	G	125.9	2.4		
546	G	122.1	2.3	7.0	EL80A4-D2
576	G	132.7	2.5		
563	G	126.5	2.4		
562	G	126.4	2.4	7.3	EL80A4-D1
591	G	135.9	2.6		
599	G	137.0	2.6		
590	ACC	128.5	2.4		
596	G	66.5	1.3	8.9	EL80A-D3
918	G	113.7	2.2		
573	WSCG	131.6	2.5	4.6	EL80A3-D2
577	G	125.7	2.4		
938	G	123.7	2.3	4.7	EL80A3-D1
677	G	137.7	2.6		
1385	WSC	141.1	2.7		
595	G	132.6	2.5	7.8	EL80A-D2
574	WSC	143.0	2.7		
575	WSC	154.1	2.9	5.6	EL80A2-D3
585	WSCG	6.1	0.1		
586	WSCG	12.2	0.2		
1384	WSCG	5.0	0.1		
583	WSCG	84.2	1.6		
584	WSCG	134.6	2.5	4.6	EL80A2-D2
581	WSCG	136.5	2.6		
582	WSCG	3.4	0.1		
947	WSCG	10.9	0.2		
1032	WSCG	14.3	0.3	3.1	EL80A2-D1
678	G	135.9	2.6		
1286	G	6.8	0.1		
679	G	138.9	2.6	5.3	EL80A1-D2

East Low - Pipe Lateral 80					
FIELD (number)	***Irrigation Category***	***Field (Acres)***	***Field Q (ft^3/s)***	***Delivery Q (ft^3/s)***	***Delivery Name***
675	WSCG	120.9	2.3		
676	WSCG	119.6	2.3		
1024	WSCG	9.6	0.2	4.7	EL80A1-D1
681	G	102.5	1.9		
680	G	95.6	1.8	3.8	EL80A-D1

East Low - Pipe Lateral 85					
FIELD (number)	***Irrigation Category***	***Field (Acres)***	***Field Q (ft^3/s)***	***Delivery Q (ft^3/s)***	***Delivery Name***
920	G	132.5	2.5		
1,044	G	132.2	2.5	5.0	EL85-D4
537	G	131.0	2.5		
538	G	124.8	2.4		
874	G	119.7	2.3		
873	G	127.8	2.4	9.5	EL85B-D1
1,013	G	134.9	2.6		
1,014	G	126.8	2.4	5.0	EL85-D3
447	G	126.9	2.4		
448	G	121.7	2.3		
449	G	126.0	2.4		
450	G	123.5	2.3	9.4	EL85A-D3
453	G	128.4	2.4		
872	G	128.7	2.4	4.9	EL85A-D2
539	G	125.7	2.4		
540	G	127.3	2.4		
541	G	127.1	2.4		
542	G	127.1	2.4	9.6	DL85A-D1
877	G	127.1	2.4		
878	G	122.6	2.3	4.7	EL85-D2
876	WSCG	134.6	2.5		
569	WSCG	130.2	2.5		
875	G	128.3	2.4		
568	G	130.7	2.5		
571	G	26.2	0.5		
951	WSCG	86.1	1.6		
952	WSCG	15.0	0.3		
570	WSCG	3.1	0.1	12.4	EL85-D1

East Low - Pipe Lateral 89G

FIELD (number)	Irrigation Category	Field (Acres)	Field Q (ft^3/s)	Delivery Q (ft^3/s)	Delivery Name
1216	G	95.1	1.8		
1299	G	30.9	0.6		
1187	G	114.6	2.2	4.56	EL89-D5
1213	WSC	35.1	0.7		
1214	WSC	65.0	1.2		
1411	G	130.8	2.5	4.37	EL89K-D1
1414	G	15.3	0.3		
1212	G	145.2	2.7		
1169	G	131.0	2.5	5.52	EL89J-D1
1207	WSCG	39.2	0.7	0.74	EL89I-D5
1174	WSC	149.9	2.8		
1175	WSC	152.3	2.9		
1176	WSC	125.1	2.4		
1201	WSC	141.9	2.7		
1427	WSC	11.8	0.2	11.00	EL89I-D4
1168	G	7.3	0.1		
1160	G	7.5	0.1		
1161	G	10.8	0.2		
1164	G	129.8	2.5		
1165	G	6.3	0.1		
1166	G	125.1	2.4	5.43	EL89I-D3
1162	G	9.0	0.2		
1163	G	7.1	0.1		
1156	G	11.5	0.2		
1167	G	3.8	0.1	0.60	EL89I-D2
1157	G	132.9	2.5		
1158	G	141.2	2.7		
1200	G	29.3	0.6	5.74	EL89I-D1
1	G	3.9	0.1		
1107	G	1.9	0.04		
1108	G	135.7	2.6	2.68	EL89H-D3
1155	G	8.8	0.2		
1159	G	5.2	0.1		
1418	G	12.2	0.2		
1419	G	105.0	2.0		
1420	G	6.8	0.1		
1409	G	64.7	1.2	3.84	EL89H-D2
1119	G	9.5	0.2		

East Low - Pipe Lateral 85					
FIELD (number)	***Irrigation Category***	***Field (Acres)***	***Field Q (ft^3/s)***	***Delivery Q (ft^3/s)***	***Delivery Name***
1429	G	6.0	0.1	0.29	EL89H-D1
1417	G	4.6	0.1		
1111	G	35.2	0.7		
1110	G	2.4	0.0		
1109	G	12.8	0.2		
1205	G	35.8	0.7	1.72	EL89-D4
1113	G	3.8	0.1	0.07	EL89GR-D1
1170	WSC	98.1	1.9		
1172	WSCG	132.7	2.5		
1154	WSCG	45.5	0.9	5.23	EL89GL-D3
1171	WSCG	135.7	2.6		
1152	WSCG	69.2	1.3		
1153	WSCG	86.3	1.6	5.51	EL89GL-D2
1120	WSCG	26.7	0.5		
1118	WSCG	80.2	1.5		
1121	WSCG	130.8	2.5	4.50	EL89GL-D1
1116	WSC	33.9	0.6		
1117	WSC	182.2	3.4	4.09	EL89-D3
1196	G	10.7	0.2		
1122	WSC	33.5	0.6	0.84	EL89F-D1
1131	WSCG	128.5	2.4		
1410	WSCG	8.2	0.2		
1129	WSCG	94.1	1.8	4.37	EL89E-D1
1423	G	16.0	0.3		
1424	G	9.9	0.2		
1425	G	111.9	2.1		
1215	G	111.6	2.1	4.72	EL89D-D4
1185	WSC	89.0	1.7		
1197	G	14.0	0.3		
1198	WSC	8.0	0.2		
1199	G	6.3	0.1		
1151	WSCG	96.7	1.8		
1413	WSCG	124.0	2.3		
1415	WSC	102.7	1.9	8.35	EL89D-D3
1428	G	129.2	2.4		

East Low - Pipe Lateral 85

FIELD (number)	***Irrigation Category***	***Field (Acres)***	***Field Q (ft^3/s)***	***Delivery Q (ft^3/s)***	***Delivery Name***
1123	G	10.9	0.2		
1337	G	134.6	2.5	5.20	EL89D-D2
1182	G	23.0	0.4		
1195	G	133.2	2.5		
1183	G	130.0	2.5	5.42	EL89D-D1
1127	WSCG	16.0	0.3		
1193	G	10.6	0.2		
1128	G	92.1	1.7		
1130	G	48.4	0.9	3.16	EL89-D2
1338	G	131.6	2.5		
1339	G	4.8	0.1	2.58	EL89C-D2
1194	G	10.9	0.2		
1184	G	137.1	2.6		
1124	G	160.0	3.0	5.83	EL89C-D1
985	G	118.8	2.3	2.25	EL89B-D2
1030	G	120.3	2.3		
1125	G	137.0	2.6		
1126	G	134.5	2.5	7.42	EL89B-D1
1191	WSC	6.5	0.1		
1221	WSC	19.8	0.4	0.50	EL89A-D6
1222	WSC	7.2	0.1		
1138	WSC	49.0	0.9		
1139	WSC	25.9	0.5	1.55	EL89A-D5
1136	WSC	14.9	0.3		
1217	WSC	4.2	0.1		
1180	WSC	16.3	0.3	0.67	EL89A-D4
1177	WSC	6.5	0.1		
1137	WSC	174.3	3.3		
1141	WSC	19.7	0.4	3.80	EL89C-A-B-D1
1140	WSC	139.6	2.6		
1181	WSC	56.1	1.1		
1150	WSC	142.7	2.7	6.41	EL89A-D3
954	G	21.8	0.4	0.41	EL89A-A-D1
942	G	85.2	1.6		

East Low - Pipe Lateral 85					
FIELD (number)	***Irrigation Category***	***Field (Acres)***	***Field Q (ft^3/s)***	***Delivery Q (ft^3/s)***	***Delivery Name***
945	WSC	114.4	2.2		
946	WSC	6.5	0.1	3.90	EL89A-D2
944	WSC	51.5	1.0		
988	G	137.9	2.6	3.59	EL89A-D1
986	G	127.6	2.4		
987	G	138.7	2.6		
1029	G	140.4	2.7		
1043	G	130.2	2.5	10.17	EL89-D1

East High Canal Lateral Deliveries

East High - Pipe Lateral 04					
FIELD (number)	***Irrigation Category***	***Field (Acres)***	***Field Q (ft^3/s)***	***Delivery Q (ft^3/s)***	***Delivery Name***
375	G	156.95	2.97	2.97	EH4 Del. Sta. 98+00
323	G	46.47	0.88	0.88	EH4 Del. Sta. 204+13
376	G	161.51	3.06	3.06	EH4 Del. Sta. 222+75 End of EH4
1374	G	28.84	0.55		
410	G	138.97	2.63	3.18	EH4A Del. Sta. 126+01
380	G	64.00	1.21		
374	G	187.54	3.55	4.76	EH4A Del. Sta. 144+41 End of EH4A
1375	G	30.56	0.58	0.58	EH4A1 Del. Sta. 26+81 End of EH4A1

East High - Pipe Lateral 11					
FIELD (number)	***Irrigation Category***	***Field (Acres)***	***Field Q (ft^3/s)***	***Delivery Q (ft^3/s)***	***Delivery Name***
326	G	136.9	2.6		
381	G	16.4	0.3	2.9	EH11 Del. Sta. 393+69
336	G	292.6	5.5	5.5	EH11 Del. Sta. 446+45
337	G	71.6	1.4		
338	G	128.4	2.4	3.8	EH11 471+53=0+00 EH11C & Del.
343	G	260.9	4.9		
346	WSCG	94.0	1.8		

East High - Pipe Lateral 11

FIELD (number)	*Irrigation Category*	*Field (Acres)*	*Field Q (ft^3/s)*	*Delivery Q (ft^3/s)*	*Delivery Name*
344	WSC	57.8	1.1		
345	WSC	165.6	3.1	11.0	EH11 Del. Sta. 569+63 End of EH11
334	G	128.1	2.4		
387	G	24.1	0.5		
370	G	131.1	2.5	5.4	EH11A Del. Sta. 73+08
385	G	10.1	0.2		
335	G	121.4	2.3		
371	G	85.0	1.6	4.1	EH11A Del. Sta. 100+29 End of EH11A
386	G	11.0	0.2		
333	G	130.3	2.5		
331	G	130.9	2.5		
330	G	131.9	2.5	7.7	EH11A1 Del. Sta.26+98
329	G	130.0	2.5	2.5	EH11A1 Del Sta. 53+75 End of EH11A1
327	G	148.9	2.8		
328	G	124.9	2.4	5.2	EH11B Del. Sta. 27+50 End of EH11B
342	G	133.5	2.5		
339	G	32.2	0.6		
340	G	36.3	0.7		
341	G	29.6	0.6	4.4	EH11C Del. Sta. 40+22 End of EH11C

East High - Pipe Lateral 15

FIELD (number)	*Irrigation Category*	*Field (Acres)*	*Field Q (ft^3/s)*	*Delivery Q (ft^3/s)*	*Delivery Name*
1,293	G	66.7	1.3	1.3	EH15 Del. Sta. 36+44 End of EH15

East High - Pipe Lateral 19

FIELD (number)	*Irrigation Category*	*Field (Acres)*	*Field Q (ft^3/s)*	*Delivery Q (ft^3/s)*	*Delivery Name*
1,438	G	122.9	2.3	2.3	EH19 Del. Sta. 16+16

East High - Pipe Lateral 19					
FIELD (number)	***Irrigation Category***	***Field (Acres)***	***Field Q (ft^3/s)***	***Delivery Q (ft^3/s)***	***Delivery Name***
362	G	130.6	2.5		
361	G	129.8	2.5		
347	G	123.8	2.3		
348	G	125.3	2.4	9.7	EH19 Del. Sta. 67+57
383	G	123.4	2.3	2.3	EH 19 Del. Sta. 120+74
358	G	94.7	1.8	1.8	EH19 Del. Sta. 174+02
357	G	131.1	2.5		
356	G	22.4	0.4		
354	G	127.3	2.4	5.3	EH19 Del. Sta. 226+98
352	G	23.8	0.5		
353	G	123.9	2.3		
384	G	9.5	0.2	3.0	EH 19 Del. Sta. 276+05
389	G	124.9	2.4		
406	G	123.7	2.3		
1,439	G	76.2	1.4	6.2	EH 19 Del. Sta. 305+27
377	G	132.6	2.5		
407	G	119.9	2.3		
379	G	126.4	2.4		
369	G	136.7	2.6	9.8	EH 19 Del. Sta. 358+74
378	G	119.8	2.3	2.3	EH19 Sta. 385+60 Del. End of EH19
365	G	91.5	1.7		
366	G	186.3	3.5		
363	G	173.7	3.3	8.5	EH19A Del. Sta. 49+77
364	G	96.6	1.8		
359	G	126.0	2.4		
360	G	126.8	2.4	6.6	EH19A Del. Sta. 79+70 End of EH19A
350	G	125.3	2.4		
351	G	126.7	2.4		
382	G	11.3	0.2		
349	G	39.8	0.8		
1,373	G	22.2	0.4		

East High - Pipe Lateral 19

FIELD (number)	Irrigation Category	Field (Acres)	Field Q (ft^3/s)	Delivery Q (ft^3/s)	Delivery Name
368	G	125.5	2.4	8.5	EH19B Del. Sta. 28+20
388	G	128.5	2.4		
1,372	G	128.7	2.4	4.9	EH19B Del. Sta. 52+60 End of EH19B

East High - Pipe Lateral 29

FIELD (number)	Irrigation Category	Field (Acres)	Field Q (ft^3/s)	Delivery Q (ft^3/s)	Delivery Name
239	G	128.3	2.43		
246	G	125.3	2.37		
1,432	G	10.2	0.19		
160	G	100.4	1.90		
1,450	G	47.8	0.91		
1,306	G	126.8	2.40	10.2	EH29 Sta. 93+86 =0+00 EH29A & Del.
235	WSCG	68.4	1.3		
237	WSC	133.7	2.5		
238	WSCG	14.7	0.3	4.1	EH29 Del. Sta. 277+62
234	WSCG	66.8	1.3		
236	WSC	188.5	3.6	4.8	EH29 Del. Sta. 331+09
408	G	119.0	2.3		
41	G	213.8	4.0	6.3	EH29 Del. Sta. 390+10
42	G	7.4	0.1		
43	G	67.7	1.3	1.4	EH29 Del. Sta. 415+93 End of EH29
1,433	G	2.8	0.1		
1,434	G	85.9	1.6	1.7	EH29A Del. Sta. 27+99 End of EH29A
49	G	178.5	3.4		
50	G	88.7	1.7	5.1	EH29B Del. Sta. 110+92
1,345	WSCG	128.5	2.4		
47	G	124.5	2.4		
1,346	WSCG	22.7	0.4	5.2	EH29B Del. Sta. 137+77

East High - Pipe Lateral 29					
FIELD (number)	***Irrigation Category***	***Field (Acres)***	***Field Q (ft^3/s)***	***Delivery Q (ft^3/s)***	***Delivery Name***
1,344	WSCG	124.8	2.4		
48	G	121.9	2.3	4.7	EH29B Sta. 190+71=0+00 EH29B1 &Del.
53	WSC	128.2	2.4		
55	WSC	141.1	2.7		
54	WSC	94.8	1.8	6.9	EH29B Del. Sta. 246+75 End EH29B
46	WSC	135.3	2.6	2.6	EH29B1 Del. Sta. 27+28 End of EH29B1

East High - Pipe Lateral 33					
FIELD (number)	***Irrigation Category***	***Field (Acres)***	***Field Q (ft^3/s)***	***Delivery Q (ft^3/s)***	***Delivery Name***
26	G	132.2	2.5		
33	G	126.6	2.4	4.9	EH33 Del. Sta. 177+95
40	G	126.9	2.4	2.4	EH33 Sta. 204+97 = 0+00 EH33D & Del.
32	G	128.3	2.4	2.4	EH33 Sta. 232+87 = 0+00 EH33E & Del.
38	G	130.9	2.5	2.5	EH33 Sta. 284+22 =0+00 EH33F & Del.
36	G	122.5	2.3		
39	G	132.5	2.5		
22	G	122.3	2.3		EH33 Del. Sta. 337+57
23	G	129.7	2.5	9.6	EH33 Del. Sta. 337+57
175	G	124.5	2.4		
176	G	131.9	2.5	4.9	EH33 Sta. 391+43 =0+00 EH33G & Del.
117	G	78.3	1.5		
118	G	9.9	0.2	1.7	EH33 Del. Sta. 473+55
318	G	48.2	0.9		
113	ACC	78.6	1.5	2.4	EH33 Del. Sta. 526+27

East High - Pipe Lateral 33

FIELD (number)	Irrigation Category	Field (Acres)	Field Q (ft^3/s)	Delivery Q (ft^3/s)	Delivery Name
114	ACC	121.1	2.3		
115	G	69.4	1.3		
1,309	G	112.3	2.1		
108	G	127.9	2.4	8.2	EH33 Del. Sta. 579+57
104	G	132.1	2.5		
103	G	133.1	2.5		
1,353	G	129.7	2.5	7.5	EH33 Del. Sta. 631+26
95	G	127.7	2.4		
96	G	132.2	2.5	4.9	EH33 Del. Sta. 655+83 End of EH33
210	G	133.6	2.5		
1,297	G	133.8	2.5		
241	G	22.6	0.4		
211	G	132.3	2.5		
242	G	133.4	2.5	10.5	EH33A Del. Sta. 100+91 End of EH33A
30	G	128.8	2.4		
27	G	126.7	2.4	4.8	EH33B Del. Sta. 27+12 End of EH33B
31	G	128.6	2.4		
1,342	G	125.4	2.4		
29	G	122.0	2.3	7.1	EH33C Del. Sta. 25+85 End of EH33C
34	G	125.4	2.4		
1,287	G	3.4	0.1	2.4	EH33D Del. Sta. 26+86 End of EH33D
35	G	125.1	2.4		
37	G	126.9	2.4		
21	G	124.8	2.4		
18	G	128.6	2.4	9.6	EH33E Del. Sta. 53+35 End of EH33E
216	G	127.5	2.4		
217	G	129.1	2.4	4.9	EH33F Del. Sta. 26+09 End of EH33F

East High - Pipe Lateral 33

FIELD (number)	Irrigation Category	Field (Acres)	Field Q (ft^3/s)	Delivery Q (ft^3/s)	Delivery Name
116	G	126.6	2.4		
1,310	G	122.7	2.3		
1,358	G	10.7	0.2		
173	ACC	106.8	2.0		
174	ACC	73.3	1.4	8.3	EH33G Del. Sta. 51+90
170	ACC	117.9	2.2		
88	G	123.9	2.3		
89	ACC	77.1	1.5	6.0	EH33G Del. Sta. 105+45 End of EH33G
1,288	G	21.9	0.4		
167	G	121.3	2.3		
1,440	ACC	70.0	1.3	4.0	EH33H Del. Sta. 52+35
172	ACC	31.4	0.6		
171	ACC	74.7	1.4	2.0	EH33H Del. Sta. 105+27 End of EH33H

East High - Pipe Lateral 35

FIELD (number)	Irrigation Category	Field (Acres)	Field Q (ft^3/s)	Delivery Q (ft^3/s)	Delivery Name
6	ACC	131.8	2.5		
5	G	129.8	2.5	5.0	EH35 Sta. 20+54 = 0+00 EH35B & Del.
177	ACC	123.4	2.3		
1,347	ACC	122.9	2.3	4.7	EH35 Del. Sta. 73+73
52	ACC	124.4	2.4		
51	ACC	123.4	2.3		
58	WSCG	86.9	1.6		
59	WSCG	18.9	0.4		
1,289	G	5.2	0.1		
60	WSCG	163.1	3.1	9.9	EH35 Del. Sta. 127+30
61	WSCG	130.0	2.5		
62	WSCG	83.2	1.6		
79	G	109.7	2.1	6.1	EH35 Sta. 180+48 = 0+00 EH35C & Del.
69	G	171.6	3.3	3.3	EH35 Sta. 239+65 = 0+00

East High - Pipe Lateral 35					
FIELD (number)	***Irrigation Category***	***Field (Acres)***	***Field Q (ft^3/s)***	***Delivery Q (ft^3/s)***	***Delivery Name***
					EH35D & EH35E & Del.
314	G	137.9	2.6	2.6	EH35 Sta. 293+94 =0+00 EH35F & Del.
315	G	136.0	2.6		
75	WSCG	138.6	2.6	5.2	EH35 Sta. 346+98 = 0+00 EH35G & Del.
76	G	133.3	2.5		
78	WSCG	133.4	2.5	5.1	EH35 Del. Sta. 399+49
63	WSC	79.6	1.5		
191	G	134.3	2.5	4.1	EH35 Del. Sta. 429+91
190	G	135.3	2.6		
64	WSC	17.7	0.3	2.9	EH35 Del. Sta. 453+15 End of EH35
8	G	126.3	2.4	2.4	JCT EH35 & EH35A Del. On EH35A
9	ACC	124.7	2.4		
10	ACC	123.4	2.3		
19	G	125.7	2.4	7.1	EH35A Del. Sta. 86+09
14	G	124.2	2.4		
24	ACC	125.3	2.4		
17	G	115.6	2.2		
1,296	ACC	120.1	2.3	9.2	EH35A Del. Sta. 139+55 End of EH35A
25	ACC	128.7	2.4		
7	ACC	126.8	2.4		
4	WSCG	129.5	2.5		
11	WSCG	129.1	2.4	9.7	EH35B Del. Sta. 53+00
3	WSCG	78.2	1.5		
13	WSCG	126.0	2.4		
12	WSCG	21.7	0.4		
15	G	127.1	2.4		
16	G	127.7	2.4	9.1	EH35B Del. Sta. 105+74 End of EH35B

East High - Pipe Lateral 35					
FIELD (number)	**Irrigation Category**	**Field (Acres)**	**Field Q (ft^3/s)**	**Delivery Q (ft^3/s)**	**Delivery Name**
1,348	G	119.2	2.3	2.3	EH35C Del. Sta. 47+50 End of EH35C
1,332	G	136.8	2.6		
1,290	G	12.2	0.2		
202	WSCG	133.4	2.5		
205	WSCG	11.9	0.2	5.6	EH35D Del. Sta. 31+84
1,350	WSCG	140.5	2.7		
1,351	G	31.9	0.6		
204	WSC	137.2	2.6		
1,430	G	8.0	0.2		
180	G	6.0	0.1		
178	WSCG	23.8	0.5		
70	G	72.8	1.4	8.0	EH35D Del. Sta. 85+06 End of EH35D
80	G	167.6	3.2	3.2	EH35E Del. Sta. 21+91
91	G	54.1	1.0	1.0	EH35E Del. Sta. 74+33
90	G	60.6	1.1	1.1	EH35E Del. Sta. 122+04 End of EH35E
305	WSCG	138.5	2.6		
200	WSCG	150.2	2.8		
313	G	133.0	2.5		
302	G	133.4	2.5		
181	WSCG	20.2	0.4	10.9	EH35F Del. Sta. 30+85
201	WSC	153.1	2.9		
199	WSCG	133.4	2.5		
74	G	70.5	1.3		
203	WSCG	16.1	0.3		
179	G	5.2	0.1		
71	G	27.9	0.5		
73	G	8.4	0.2	7.8	EH35F Del. Sta. 74+26 End of EH35F
303	G	137.7	2.6		
194	G	135.0	2.6		
77	G	133.5	2.5		

East High - Pipe Lateral 35

FIELD (number)	Irrigation Category	Field (Acres)	Field Q (ft^3/s)	Delivery Q (ft^3/s)	Delivery Name
192	WSCG	137.5	2.6	10.3	EH35G Del. Sta. 31+24
195	G	20.1	0.4		
240	G	10.9	0.2		
197	G	136.3	2.6		
198	G	9.5	0.2		
196	G	10.7	0.2		
193	WSCG	137.8	2.6	6.2	EH35G Del. Sta. 84+84 End of EH35G

East High - Pipe Lateral 42

FIELD (number)	Irrigation Category	Field (Acres)	Field Q (ft^3/s)	Delivery Q (ft^3/s)	Delivery Name
102	G	131.4	2.5		
106	G	24.4	0.5		
101	G	128.1	2.4	5.4	EH42 Sta. 17+48 = 0+00 EH42A & Del.
154	WSC	137.4	2.6		
218	WSC	126.2	2.4		
245	WSC	18.2	0.3		
300	WSCG	182.4	3.5	8.8	EH42 Del. Sta. 150+22
1,365	WSCG	133.7	2.5		
1,366	WSC	146.8	2.8	5.3	EH42 Del. Sta. 177+36
141	WSCG	46.4	0.9		
1,367	WSCG	132.2	2.5		
1,449	WSCG	35.8	0.7		
1,362	WSC	132.9	2.5	6.6	EH42 Del. Sta. 230+15 End of EH42
107	G	126.6	2.4		
105	G	131.4	2.5		
112	G	130.7	2.5		
111	G	132.4	2.5		
120	G	22.9	0.4	10.3	EH42A Del. Sta. 25+64
121	G	128.9	2.4		
119	G	134.3	2.5		
122	G	129.9	2.5		
125	G	61.1	1.2	8.6	EH42A Del. Sta. 78+68

East High - Pipe Lateral 42					
FIELD (number)	***Irrigation Category***	***Field (Acres)***	***Field Q (ft^3/s)***	***Delivery Q (ft^3/s)***	***Delivery Name***
124	G	106.8	2.0		
123	G	122.0	2.3	4.3	EH42A Del. Sta. 130+17 End of EH42A
99	G	133.6	2.5		
97	WSC	130.3	2.5		
1,357	WSC	19.9	0.4	5.4	EH42B Del. Sta. 26+12
92	WSC	135.4	2.6		
83	WSC	149.8	2.8	5.4	EH42B Del. Sta. 78+91
81	G	132.3	2.5	2.5	EH42B Del. Sta. 105+33 End of EH42B
100	WSC	128.3	2.4		
109	WSCG	126.3	2.4	4.8	EH42C Del. Sta. 26+19
136	WSCG	27.3	0.5		
110	WSCG	124.9	2.4	2.9	EH42C Del. Sta. 78+73 End of EH42C
134	WSC	133.0	2.5		
142	WSC	132.2	2.5		
133	WSCG	128.1	2.4		
299	WSCG	132.9	2.5		
1,291	WSC	9.3	0.2	10.1	EH42D Del. Sta. 26+97
132	WSC	127.5	2.4		
312	WSC	132.8	2.5		
301	WSC	139.2	2.6		
311	WSC	137.8	2.6	10.2	EH42D Del. Sta. 79+02
1,356	WSC	149.3	2.8		
317	WSC	135.1	2.6		
306	WSC	138.2	2.6	8.0	EH42D Sta. 131+75 = 0+00 EH42D1 & Del.
309	WSCG	99.6	1.9		
250	WSCG	83.1	1.6	3.5	EH42D Del. Sta. 166+92 End of EH42D
249	WSCG	186.0	3.5		

East High - Pipe Lateral 42					
FIELD (number)	***Irrigation Category***	***Field (Acres)***	***Field Q (ft^3/s)***	***Delivery Q (ft^3/s)***	***Delivery Name***
256	WSCG	24.8	0.5		
1,363	WSC	111.5	2.1		
1,355	WSC	3.7	0.1		
258	WSCG	88.8	1.7	7.9	EH42D1 Del. Sta. 28+03
255	WSCG	6.8	0.1		
248	WSCG	65.5	1.2		
253	WSC	77.1	1.5		
254	WSC	233.7	4.4		
252	WSC	9.2	0.2		
251	WSC	2.9	0.1	7.5	EH42D1 Del. Sta. 82+09 End of EH42D1
135	WSCG	130.2	2.5		
143	WSCG	128.4	2.4		
137	WSCG	130.2	2.5		
144	WSCG	133.3	2.5	9.9	EH42E Del. Sta. 25+49
138	WSCG	127.4	2.4		
139	G	132.0	2.5	4.9	EH42E Del. Sta. 79+22 End of EH42E
153	WSCG	128.1	2.4		
152	WSCG	99.6	1.9	4.3	EH42F Del. Sta. 24+80 End of EH42F

East High - Pipe Lateral 47

FIELD (number)	*Irrigation Category*	*Field (Acres)*	*Field Q (ft^3/s)*	*Delivery Q (ft^3/s)*	*Delivery Name*
294	G	133.5	2.5		
295	G	127.7	2.4	4.9	Begin EH47 Sta. 0+00 = 0+00 EH47A & Del.
296	G	130.5	2.5		
166	G	132.4	2.5		
297	G	128.9	2.4	7.4	EH47 Del. Sta. 59+21 End of EH47
290	G	123.7	2.3		
291	G	123.6	2.3		
293	G	22.3	0.4		
289	G	124.8	2.4		
292	G	122.8	2.3	9.8	EH47A Del. Sta. 16+81 End of EH47A

East High - Pipe Lateral 50G

FIELD (number)	*Irrigation Category*	*Field (Acres)*	*Field Q (ft^3/s)*	*Delivery Q (ft^3/s)*	*Delivery Name*
288	G	126.1	2.4		
809	G	122.9	2.3	4.7	EH50 Sta. 34+68 = 0+00 EH50A & Del.
808	G	132.6	2.5		
287	G	88.8	1.7		
806	G	124.1	2.3	6.5	EH50 Sta. 88+01 = 0+00 EH50B & EH50C & Del.
1,303	G	51.3	1.0		
805	G	130.1	2.5	3.4	EH50 Sta. 140+34 End of EH50 = 0+00 EH50D & EH50E & Del.
148	G	131.6	2.5	2.5	EH50A Del. Sta. 54+18
147	G	132.6	2.5		
140	WSCG	133.6	2.5	5.0	EH50A Del. Sta. 107+19 End of EH50A
151	G	176.9	3.4	3.4	EH50B Del. Sta. 54+28
149	G	134.4	2.5		

East High - Pipe Lateral 50G					
FIELD (number)	***Irrigation Category***	***Field (Acres)***	***Field Q (ft^3/s)***	***Delivery Q (ft^3/s)***	***Delivery Name***
150	WSCG	98.2	1.9		
146	G	135.2	2.6		
223	WSCG	133.3	2.5	9.5	EH50B Del. Sta. 107+04
221	WSCG	40.6	0.8		
222	WSCG	18.5	0.3		
220	G	22.6	0.4	1.5	EH50B Del. Sta. 159+66 End of EH50B
1,395	WSCG	26.3	0.5		
975	WSC	114.8	2.2		
974	WSC	67.1	1.3	3.9	EH50C Del. Sta. 19+40 End of EH50C
286	G	153.1	2.9		
156	G	93.5	1.8	4.7	EH50D Del. Sta. 54+33
155	G	206.9	3.9	3.9	EH50D Del. Sta. 79+46 End of EH50D
1,394	G	122.2	2.3		
807	WSCG	24.0	0.5		
977	WSCG	164.9	3.1	5.9	EH50E Del. Sta. 51+06
976	G	127.8	2.4	2.4	EH50E Del. Sta. 104+54 End of EH50E

Black Rock Branch Canal Lateral Deliveries

Black Rock Branch - Pipe Lateral 02					
FIELD (number)	***Irrigation Category***	***Field (Acres)***	***Field Q (ft^3/s)***	***Delivery Q (ft^3/s)***	***Delivery Name***
185	G	109.7	2.1		
186	G	129.2	2.4		
187	G	129.1	2.4		
405	G	112.6	2.1	9.1	BRB2 Del. Sta. 14+11
183	G	121.1	2.3		
320	G	122.2	2.3	4.6	BRB2 Sta. 39+71 = 0+00 BRB2A & Del.

Black Rock Branch - Pipe Lateral 02					
FIELD (number)	***Irrigation Category***	***Field (Acres)***	***Field Q (ft^3/s)***	***Delivery Q (ft^3/s)***	***Delivery Name***
188	G	131.6	2.5		
1371	G	120.5	2.3		
189	G	130.0	2.5		
404	G	122.4	2.3	9.6	BRB2 Del. Sta. 93+15
402	G	120.7	2.3	2.3	BRB2 Del. Sta. 120+59 End of BRB2
184	G	128.6	2.4	2.4	BRB2A Del. Sta. 54+23 End of BRB2A

Black Rock Branch - Pipe Lateral 07					
FIELD (number)	***Irrigation Category***	***Field (Acres)***	***Field Q (ft^3/s)***	***Delivery Q (ft^3/s)***	***Delivery Name***
1224	G	131.2	2.5		
1225	G	125.8	2.4	4.9	BRB7 Del. Sta. 21+05
1238	G	122.7	2.3		
1237	G	117.9	2.2	4.6	BRB7 Del. Sta. 153+34
1245	G	113.8	2.2		
1246	G	119.3	2.3		
1248	G	128.7	2.4		
1247	G	127.7	2.4	9.3	BRB7 Sta. 207+13 = 0+00 BRB7B & Del.
1250	G	124.3	2.4		
1249	G	124.0	2.3		
1251	G	108.9	2.1		
1252	G	106.3	2.0	8.8	BRB7 Del. Sta. 260+02
1334	G	125.9	2.4		
1335	G	128.2	2.4		
1275	G	130.6	2.5	7.3	BRB7 Del. Sta. 313+39
1276	G	130.3	2.5		
1278	G	75.3	1.4		
1277	G	129.4	2.5	6.3	BRB7 Sta. 366+62 = 0+00 BRB7C & Del.
1279	G	123.3	2.3	2.3	BRB7 Del. Sta. 418+12

Black Rock Branch - Pipe Lateral 07

FIELD (number)	Irrigation Category	Field (Acres)	Field Q (ft^3/s)	Delivery Q (ft^3/s)	Delivery Name
1280	G	125.3	2.4		
1283	ACC	125.3	2.4	4.7	BRB7 Del. Sta. 473+03 End of BRB7
1241	G	123.3	2.3	2.3	BRB7A Del. Sta. 25+86
1271	G	125.4	2.4		
1243	G	124.9	2.4		
2	G	20.2	0.4	5.1	BRB7A Del. Sta. 105+00
1272	G	113.7	2.2		
1244	G	119.0	2.3		
1331	G	33.9	0.6		
1330	G	125.4	2.4	7.4	BRB7A Del. Sta. 157+80
1281	G	115.3	2.2		
1273	G	125.2	2.4		
1274	G	131.5	2.5	7.0	BRB7A Del. Sta. 210+70 End of BRB7A
1265	G	126.2	2.4		
1266	G	126.3	2.4	4.8	BRB7B Del. Sta. 52+13
1254	G	106.3	2.0		
1257	G	105.0	2.0		
1253	G	123.6	2.3		
1256	G	105.6	2.0	8.3	BRB7B Del. Sta. 105+57
1255	G	123.6	2.3		
1333	G	125.3	2.4		
1282	G	499.8	9.5	14.2	BRB7B Del. Sta. 158+67
1258	G	113.1	2.1		
1267	G	121.0	2.3	4.4	BRB7B Del. Sta. 210+95
1268	G	112.6	2.1		
1264	G	121.6	2.3		
1263	G	109.6	2.1	6.5	BRB7B Del. Sta. 265+00
1261	G	109.6	2.1		
1262	G	109.6	2.1	4.2	BRB7B Del. Sta. 318+32
1260	G	109.6	2.1	2.1	BRB7B Del. Sta. 343+90 End

Black Rock Branch - Pipe Lateral 07					
FIELD (number)	***Irrigation Category***	***Field (Acres)***	***Field Q (ft^3/s)***	***Delivery Q (ft^3/s)***	***Delivery Name***
					of BRB7B
1259	G	128.2	2.4	2.4	BRB7C Del. Sta. 26+73 End of BRB7C

Black Rock Branch - Pipe Lateral 11					
FIELD (number)	***Irrigation Category***	***Field (Acres)***	***Field Q (ft^3/s)***	***Delivery Q (ft^3/s)***	***Delivery Name***
1067	G	134.6	2.5	2.5	BRB11 Sta. 17+99 = 0+00 BRB11A & Del.
1068	G	129.6	2.5		
853	G	128.4	2.4	4.9	BRB11 Del. Sta. 52+54
856	G	122.2	2.3		
854	G	132.3	2.5		
1396	G	162.1	3.1	7.9	BRB Del. Sta. 104+39
1065	G	487.9	9.2	9.2	BRB11 Sta. 129+16 = 0+00 BRB11B & Del.
852	G	104.1	2.0		
924	G	158.1	3.0		
1034	G	131.8	2.5		
1402	G	129.1	2.4	9.9	BRB11 Del. Sta. 181+58
162	G	129.6	2.5	2.5	BRB11 Sta. 209+06 = 0+00 BRB11C & Del.
161	G	128.5	2.4	2.4	BRB11 Del. Sta. 263+07
212	G	130.6	2.5	2.5	BRB11 Del. Sta. 287+91
213	G	128.6	2.4		
126	G	135.3	2.6	5.0	BRB11 Del. Sta. 340+19
214	G	128.8	2.4		
129	G	117.6	2.2		
130	G	24.1	0.5		
215	G	129.9	2.5		
131	G	125.7	2.4	10.0	BRB11 Del. Sta. 365+13 End

Black Rock Branch - Pipe Lateral 11

FIELD (number)	***Irrigation Category***	***Field (Acres)***	***Field Q (ft^3/s)***	***Delivery Q (ft^3/s)***	***Delivery Name***
					of BRB11
1066	G	120.8	2.3		
1033	G	122.6	2.3	4.6	BRB11A Del. Sta. 25+85
1230	G	125.2	2.4		
1269	G	121.5	2.3		
1231	G	128.4	2.4		
1233	G	123.3	2.3	9.4	BRB11A Del. Sta. 79+80
1229	G	127.4	2.4		
1228	G	128.5	2.4	4.8	BRB11A Del. Sta. 104+33 End of BRB11A
851	G	103.2	2.0		
1292	G	316.5	6.0	7.9	BRB11B Del. Sta. 25+19 End of BRB11B
163	G	128.5	2.4		
1035	G	132.8	2.5		
1308	G	129.2	2.4	7.4	BRB11C Del. Sta. 24+84
164	G	132.6	2.5		
165	G	133.5	2.5	5.0	BRB11C Del. Sta. 48+40 End of BRB11C

Black Rock Branch - Pipe Lateral 17

FIELD (number)	***Irrigation Category***	***Field (Acres)***	***Field Q (ft^3/s)***	***Delivery Q (ft^3/s)***	***Delivery Name***
865	G	80.7	1.5	1.5	BRB17 Del. Sta. 52+12 End of BRB17

Black Rock Branch - Pipe Lateral 18

FIELD (number)	***Irrigation Category***	***Field (Acres)***	***Field Q (ft^3/s)***	***Delivery Q (ft^3/s)***	***Delivery Name***
922	G	73.7	1.4		
923	G	128.0	2.4	3.8	BRB18 Del. Sta. 18+33 End of BRB18

Black Rock Branch - Pipe Lateral 27

FIELD (number)	Irrigation Category	Field (Acres)	Field Q (ft^3/s)	Delivery Q (ft^3/s)	Delivery Name
1096	G	70.4	1.3		
1397	G	124.8	2.4	3.7	BRB27 Sta. 58+59 = 0+00 BRB27A & Del.
850	G	120.2	2.3	2.3	BRB27 Sta. 110+68 = 0+00 BRB27B & Del.
846	G	137.1	2.6		
843	G	135.4	2.6		
835	G	131.4	2.5	7.6	BRB27 Sta. 269+75 = 0+00 BRB27E & Del.
1094	G	123.1	2.3		
815	G	130.4	2.5		
844	G	134.3	2.5	7.3	BRB27 Sta., 323+62 = 0+00 BRB27F & Del.
814	G	131.7	2.5	2.5	BRB27 Del. Sta. 377+20
813	G	129.8	2.5	2.5	BRB27 Del. Sta. 429+52 End of BRB27
1398	G	120.7	2.3	2.3	BRB27A Del. Sta. 53+58
1445	G	133.7	2.5		
1092	G	116.0	2.2	4.7	BRB27A Del. Sta. 134+69 End of BRB27A
897	G	59.7	1.1	1.1	BRB27B Del. Sta. 53+74
847	G	122.8	2.3		
834	G	135.7	2.6		
833	G	113.7	2.2	7.0	BRB27B Del. Sta. 106+21
1090	G	122.0	2.3		
816	G	66.9	1.3	3.6	BRB27B Del. Sta. 193+79 End of BRB27B
1075	G	62.5	1.2		
1078	G	111.7	2.1	3.3	BRB27C Del. Sta. 25+78 End of BRB27C
1077	G	122.6	2.3		
1076	G	102.2	1.9		

Black Rock Branch - Pipe Lateral 27					
FIELD (number)	***Irrigation Category***	***Field (Acres)***	***Field Q (ft^3/s)***	***Delivery Q (ft^3/s)***	***Delivery Name***
900	G	127.9	2.4	6.7	BRB27D Del. Sta. 52+42 End of BRB27D
836	G	120.0	2.3		
899	G	125.1	2.4		
926	G	132.3	2.5	7.1	BRB27E Del. Sta. 53+70
925	G	131.2	2.5		
837	G	131.9	2.5	5.0	BRB27E Del. Sta. 106+42
838	G	117.9	2.2	2.2	BRB27E Del. Sta. 135+33 End of BRB27E
812	G	134.6	2.5		
842	G	131.2	2.5		
841	G	131.5	2.5		
811	G	522.7	9.9	17.4	BRB27F Del. Sta. 53+14
845	G	139.7	2.6		
840	G	140.3	2.7	5.3	BRB27F Del. Sta. 108+74
839	G	143.7	2.7	2.7	BRB27F Del. Sta. 135+76 End of BRB27F

Black Rock Branch - Pipe Lateral 28					
FIELD (number)	***Irrigation Category***	***Field (Acres)***	***Field Q (ft^3/s)***	***Delivery Q (ft^3/s)***	***Delivery Name***
859	G	119.9	2.3	2.3	BRB28 Del. Sta. 52+57
861	G	136.5	2.6		
862	G	127.9	2.4	5.0	BRB28 Del. Sta. 144+44 End of BRB28
864	G	133.7	2.5		
863	G	126.2	2.4	4.9	BRB28A Del. Sta. 26+50 End of BRB28A

Black Rock Branch - Pipe Lateral 29G					
FIELD (number)	***Irrigation Category***	***Field (Acres)***	***Field Q (ft^3/s)***	***Delivery Q (ft^3/s)***	***Delivery Name***
858	G	132.0	2.5	2.5	BRB29 Del. Sta. 6+07

Black Rock Branch - Pipe Lateral 29G					
FIELD (number)	***Irrigation Category***	***Field (Acres)***	***Field Q (ft^3/s)***	***Delivery Q (ft^3/s)***	***Delivery Name***
860	G	111.3	2.1	2.1	BRB29 Del. Sta. 61+46
832	G	137.7	2.6		
825	G	132.2	2.5		
821	G	131.4	2.5	7.6	BRB29 Sta. 169+10 = 0+00 BRB29A & Del.
826	G	122.4	2.3		
817	G	132.4	2.5		
828	G	132.1	2.5	7.3	BRB29 Sta. 220+78 = 0+00 BRB29B & BRB29C & Del.
823	G	98.6	1.9		
1098	G	134.0	2.5	4.4	BRB29 Del. Sta. 274+30
891	G	95.3	1.8	1.8	BRB29 Del. Sta. 318+60 End of BRB29
820	G	43.8	0.8	0.8	BRB29A Del. Sta. 54+82 End of BRB29A
1097	G	99.8	1.9		
827	G	132.5	2.5		
822	G	129.7	2.5	6.9	BRB29B Del. Sta. 51+86 End of BRB29B
819	G	184.0	3.5		
818	G	41.0	0.8		
894	G	142.2	2.7		
893	G	122.2	2.3	9.3	BRB29C Del. Sta. 53+77 End of BRB29C

Appendix C - 804 Contracts

Municipal and Industrial Water Service Contracts

UNIT NUMBER	AC/FT O	WNER	TURN-OUT STATION	TURN-OUT LOCATE	Original Contract Date
804 Contracts M&I					
040-801-801	2	Maiers Enterprises, LLC		EL6.9H WW	17-Apr-00
041-096-804-02	70	Lakes Mobile Home Park LLC		EL20X	18-Sep-07
041-801-804	450	Central Terminals LLC		RCD	24-May-99
042-390-804	1,000	JR Simplot, Moses Lake	STA 1510+80	ELC	25-Jan-07
043-801-804	1,000	OB-3	STA 2775+00	ELC	26-Mar-08
044-801-804	40	Bar E Dairy		EL63.8	26-May-00
045-099-804-01	10	Bethel Spanish Assembly of God		EL68	01-Jun-07
045-801-804	200	City of Othello		EL68	08-May-95
045-802-804	300	Othello School District		EL68 & EL68U	02-Feb-92
045-803-804	38	PJT House Associates		EL68M	04-Sep-02
045-804-804	28	Curtis Roberts		SL68T5	21-Jun-99
045-806-804	33	Mendonca Dairy		EL68T5	21-Jun-99
046-801-804	1,000	Jr Simplot, Othello	STA 4202+00	ELC	02-Apr-91
046-802-804	1,000	McCain Foods Inc	STA 4099+00	ELC	24-May-00
049-801-804	65	Adams Co. Parks & Rec		PE14.7	06-Jul-99
049-802-804	45	Assembly of Faith		PE14.7	04-May-99

(Information provided by Lisa Lusk, ECBID, on 08/07/08)

Appendix D - East High Canal Bridge Crossings and Road Relocations

Bridge	Relocated Road	Station	Lanes each	Span length	Bridge height	Bedrock depth	Abutment height	Approach Road Surfacing Property Type	Bridge Deck Barrier length (ft)	Reloc. Road Road Miles	Crossing Name
1		250+35.66	1	54	17.5	15	16	gravel	108.0	0	27 NE Road (Wildlife Crossing)
2		638+10.46	2	54	17.5	0	9	gravel	108.0	0	22 NE Road
3		1062+49.05	1	54	17.5	15	16	gravel	108.0	0	17 NE Road - dirt
4		1116+32.61	1	54	17.5	15	16	gravel	108.0	0	16 NE Road - dirt
5		1205+14.35	1	54	17.5	0	9	gravel	108.0	0	Road R NE
7		1417+13.37	1	43	15.5	0	9	gravel	86.0	0	Basalt Mine- dirt - realign?
8		1507+69.24	1	43	15.5	3	9	gravel	86.0	0	12 NE Road - dirt - Needed?
9		1570+78.87	2	43	15.5	6	9	gravel	86.0	0	11 NE Road
10		1643+67.14	2	43	15.5	10	11	gravel	86.0	0	Road S NE - surface unknown
	1a	1781+81.40	1	43	15.5	14	15	gravel	0		Road R NE
	1b	1798+98.27	1	43	15.5	14	15	gravel	0	1.0	Road R NE #2
	1c	1830+09.49	1	43	15.5	14	15	gravel	0		Road R NE #3
11		1836+22.03	2	43	15.5	14	15	gravel	86.0	0	Road 9 NE
12		1910+09.51	1	37	13.5	6	9	gravel	74.0	0	Road 8 NE
13		1980+31.97	2	37	13.5	14	15	gravel	74.0	0	Road 7 NE
14		2003+20.65	1	37	13.5	14	15	gravel	74.0	0	Road R NE
15		2103+04.08	2	37	13.5	7	9	gravel	74.0	0	Road 6 NE
		2220+02.69	2	37	13.5	12	13	gravel	74.0	0	Road 4 NE
	2a	2260+51.86	2	37	13.5	12	13	gravel	0		Road 4 NE #2
	2b	2273+27.85	2	37	13.5	12	13	gravel	0	0.6	Road 4 NE #3
	2c	2275+40.65	2	37	13.5	12	13	gravel	0		Road T NE
	3	2319+72.35	2	37	13.5	12	13	gravel	0	0.5	Road 4 NE #4
16		2340+96.60	2	37	13.5	12	13	gravel	74.0	0	Road U NE
	4	2354+03.63	2	37	13.5	18	19	gravel	0	0.5	Road 4 NE #5
17		2396+94.68	1	37	13.5	12	13	gravel	74.0	0	Road V NE
	5	2440+74.49	2	37	13.5	12	13	gravel	0	0.3	Road 4 NE #6
18		2454+00.34	2	37	13.5	12	13	paved	74.0	0	Road W NE
	Relocated		Lanes Span		Bridge	Bedrock	Abutment	Approach Road Surfacing	Bridge Deck Barrier		Road Crossing

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Bridge	Road	Station	each	length	height	depth	height	Property Type	length (ft)	Miles	Name
	6	2457+74.35	2	37	13.5	12	13	gravel	0	0.15	Road 4 NE #7
19		2542+47.14	2	37	13.5	7	9	gravel	74.0	0	8 NE Road
20		2602+59.07	2	37	13.5	7	9	paved	74.0	0	W Rosenoff Road
Black Rock Branch Canal											
21		64+56.01	1	59	14.0	2	9	gravel	118.0	0	Road V Crossing
22		136+49.68	2	59	14.0	2	9	paved	118.0	0	Road W Crossing
23		183+50.17	2	59	14.0	17	18	gravel	118.0	0	13 NE Road
	7a	360+53.76	2	55	13.5	17	18	gravel	0	0.6	Kissler Road E
	7b	389+18.81	2	55	13.5	2	9	gravel	0		Kissler Road E #2
	8a	429+47.27	2	55	13.5	2	9	gravel	0	0.5	Lessor Road N
	8b	448+74.76	2	55	13.5	2	9	gravel	0		Lessor Road N #2
24		472+74.56	2	55	13.5	2	9	paved	110.0	0	Davis Road E
	9	480+43.32	2	55	13.5	2	9	gravel	0	0.2	N Irby Road
	10	622+35.64	2	48	13.0	2	9	gravel	0	0.5	W Arlt Road
25		645+49.28	2	48	13.0	2	9	paved	96.0	0	N Moody Road
	11a	679+38.59	2	48	13.0	5	9	gravel	0	0.4	W Arlt Road #2
	11b	693+74.00	2	48	13.0	13	14	gravel	0		W Arlt Road #3
26		864+70.69	2	29	11.5	5	9	gravel	58.0	0	North Deal Road
27		973+42.90	2	29	11.5	13	14	gravel	58.0	0	Tokio Road W
28		1244+49.41	2	39	11.5	20	21	gravel	78.0	0	N Klum Road
	12	1273+26.44	2	39	11.5	20	21	gravel	0	0.5	W Rehn Road
29		1418+72.24	2	33	10.0	2	9	gravel	66.0	0	Schoessler Road W
	13a	1432+69.82	2	33	10.0	2	9	gravel	0	0.8	Schoessler Road W 2
	13b	1438+84.14	2	33	10.0	2	9	gravel	0		Schoessler Road W 3
	13c	1462+88.16	2	33	10.0	14	15	gravel	0		Schoessler Road W 4
	13d	1470+19.71	2	33	10.0	14	15	gravel	0		Schoessler Road W 5
	14a	1484+86.09	2	33	10.0	14	15	paved	0	0.6	N Batum Road
	14b	1489+34.96	2	33	10.0	14	15	paved	0		N Batum Road 2
	14c	1502+37.35	2	33	10.0	14	15	paved	0		N Batum Road 3
	14d	1506+45.30	2	33	10.0	6	9	paved	0		N Batum Road 4
30		1524+60.17	2	33	10.0	6	9	paved	66.0	0	N Batum Road 5

Appendix E - Drainage Inlets/Culverts

Station	Drainage Capacity	CMP (Pipe Drain Inlet)					Concrete Chute Inlet with Deck	RCP Cross Drainage Culvert (Single Barrel pipe unless noted)								
		# of Barrels	18"	21"	24"	30"		30"	33"	39"	42"	45"	48"	54"	66"	84"
	ft^3/s		LF	LF	LF	LF	EA	LF	LF	LF	LF	LF	LF	LF	LF	LF
East High Canal Reach 1																
13+00	85	0					1									
24+00	23	1				47										
63+00	11	1		47												
70+00	11	1		47												
79+00	26	1				47										
84+00	45	0							289							
95+00	27	2			95											
102+00	86	0										289				
133+00	39	2				95										
146+50	36	2				95										
156+00	32	2				95										
176+00	35	2				95										
192+00	85	0										289				
203+50	46	2				95										
216+00	52	2				95										
242+50	25	1				47										
259+00	126	0												289		
270+50	900	0						289 LF of Double Barrel Box Culvert (est. 8'x8')								
276+50	27	2			95											
281+50	47	2				95										
301+00	23	1				47										
323+00	56	0					1									
352+50	36	2				94										
361+00	109	0					1									
369+00	32	2				94										
375+00	23	1				47										
381+50	25	1				47										
405+00	61	0					1									
491+00	34	2				94										
502+00	38	2				94										
515+00	41	2				94										
523+00	31	2				94										
534+00	27	2			94											
551+00	34	2				94										
558+00	24	0					1									
567+00	22	1				47										
572+00	16	1				47										
601+50	33	2				94										
605+00	25	1				47										
633+00	11	1			47											
661+00	18	1				47										
673+50	33	2				94										
752+00	20	1				47										
764+00	33	2				94										
881+00	24	1				47										

Station	Drainage Capacity	CMP (Pipe Drain Inlet)					Concrete Chute Inlet with Deck	RCP Cross Drainage Culvert (Single Barrel pipe unless noted)								
		# of Barrels	18"	21"	24"	30"		30"	33"	39"	42"	45"	48"	54"	66"	84"
	ft^3/s		LF	LF	LF	LF	EA	LF	LF	LF	LF	LF	LF	LF	LF	LF
890+00	13	1			47											
906+00	33	2				94										
967+00	65	0					1									
987+00	187	0													283	
1006+00	37	2				94										
1019+00	42	2				94										
1115+00	278	0														283
1127+00	17	1				47										
1135+00	94	0											283			
1181+00	38	2				94										
1200+50	18	1				47										
1216+00	69	0					1									
1232+00	107	0												283		
1255+00	45	2				94										
1262+50	96	0											283			
1286+50	31	2				94										
1298+50	22	1				47										
1304+00	40	2				94										

Station	Drainage Capacity	CMP (Pipe Drain Inlet)					Concrete Chute Inlet with Deck	RCP Cross Drainage Culvert (Single Barrel pipe unless noted)								
		# of Barrels	18"	21"	24"	30"		30"	33"	39"	42"	45"	48"	54"	66"	84"
	ft^3/s		LF	LF	LF	LF	EA	LF	LF	LF	LF	LF	LF	LF	LF	LF
East High Canal 2 Reach 1																
1342+50	36	2				84										
1357+00	42	2				84										
1384+00	55	0					1									
1401+00	10	1		42												
1419+00	32	2				84										
1435+00	53	0					1									
1460+50	35	2				84										
1486+00	46	2				84										
1500+50	65	0								228						
1510+00	24	1				42										
1521+00	54	0					1									
1539+00	19	1				42										
1550+50	32	2				84										
1560+00	18	1				42										
1568+50	33	2				84										
1574+50	33	2				84										
1590+00	11	1		42												
1618+00	17	1				42										
1625+50	96	0											226			
1645+30	27	2			84											
1650+80	17	1				42										
1661+00	29	2			84											
1678+00	8	1		42												
1695+00	17	1				42										
1710+60	17	1				42										
1725+50	51	2				84										
1730+00	17	1				42										
1746+00	87	0											226			
1765+00	17	1				42										
1773+00	15	1			42											
1800+00	15	1			42											
1815+00	27	2			84											
1819+30	51	2				84										
1833+00	4	1	42													
1836+00	11	1		42												
1853+00	19	1				42										
1868+00	21	1				42										
1877+50	57	0					1									
1904+00	23	1				42										
1912+50	98	0											226			
1921+70	18	1				42										
1942+50	21	1				42										
1953+00	49	2				84										
1966+00	48	2				84										
1981+00	16	1				42										
1991+00	11	1		42												

Station	Drainage Capacity	CMP (Pipe Drain Inlet)					Concrete Chute Inlet with Deck	RCP Cross Drainage Culvert (Single Barrel pipe unless noted)								
		# of Barrels	18"	21"	24"	30"		30"	33"	39"	42"	45"	48"	54"	66"	84"
	ft^3/s		LF	LF	LF	LF	EA	LF	LF	LF	LF	LF	LF	LF	LF	LF
2004+50	17	1				42										
2008+50	23	1				42										
2037+70	19	1				42										
2039+20	15	1			42											

Station	Drainage Capacity	CMP (Pipe Drain Inlet)					Concrete Chute Inlet with Deck	RCP Cross Drainage Culvert (Single Barrel pipe unless noted)								
		# of Barrels	18"	21"	24"	30"		30"	33"	39"	42"	45"	48"	54"	66"	84"
	ft^3/s		LF	LF	LF	LF	EA	LF	LF	LF	LF	LF	LF	LF	LF	LF
East High Canal 2 Reach 2																
2099+00	40	2				84										
2115+00	8	1		42												
2203+00	12	1			42											
2226+50	10	1		42												
2237+00	16	1				42										
2251+00	26	1				42										
2283+00	30	2			84											
2290+50	18	1				42										
2304+00	24	1				42										
2316+00	22	1				42										
2327+00	16	1				42										
2343+00	16	1				42										
2360+00	26	1				42										
2375+00	14	1		42												
2386+00	14	1		42												
2402+00	14	1		42												
2421+00	22	1				42										
2459+00	26	1				42										
2488+00	14	1		42												
2497+00	62	0					1									
2505+00	16	1				42										
2512+50	38	2				84										
2551+00	32	2				84										
2561+00	33	2				84										
2567+00	20	1				42										
2577+00	100	0											223			
2581+50	22	1				42										
2588+00	20	1				42										
2615+00	30	2			84											

Station	Drainage Capacity	CMP (Pipe Drain Inlet)					Concrete Chute Inlet with Deck	RCP Cross Drainage Culvert (Single Barrel pipe unless noted)								
		# of Barrels	18"	21"	24"	30"		30"	33"	39"	42"	45"	48"	54"	66"	84"
	ft^3/s		LF	LF	LF	LF	EA	LF	LF	LF	LF	LF	LF	LF	LF	LF
Black Rock Branch Canal Reach 1																
75+70	20	1				33										
84+70	34	1				65										
117+70	26	1				33										
127+20	37	1				65										
140+00	54	0					1									
156+30	56	0					1									
169+30	41	1				65										
176+40	29	1			65											
202+80	27	1			65											
236+00	26	1				33										
243+20	29	1			65											
252+50	13	1			33											
257+50	31	1				65										
273+30	15	1			33											
308+50	35	1				65										
328+70	97	0					1									
344+70	20	1				33										
370+60	55	0					1									
377+30	39	0						189								
382+00	63	0					1									
399+50	22	1				33										
430+60	82	0					1									
459+00	23	1				33										
478+00	13	1			33											
488+60	24	1				33										
508+30	52	1				65										
538+40	34	1				65										
548+80	169	0													180	
562+10	41	1				65										
577+50	17	1				32										
588+70	68	0									180					
603+50	70	0									180					
620+50	9	1		32												
638+60	40	1				60										
653+00	20	1				30										
663+00	25	1				30										
686+80	71	0					1									
698+20	62	0					1									
735+00	31	1				60										
756+30	21	1				30										
770+00	34	1				60										
788+00	24	1				30										
880+00	23	1				30										
900+00	29	1			60											
904+50	17	1				30										
940+00	50	1				60										

Station	Drainage Capacity	CMP (Pipe Drain Inlet)					Concrete Chute Inlet with Deck	RCP Cross Drainage Culvert (Single Barrel pipe unless noted)								
		# of Barrels	18"	21"	24"	30"		30"	33"	39"	42"	45"	48"	54"	66"	84"
	ft^3/s		LF	LF	LF	LF	EA	LF	LF	LF	LF	LF	LF	LF	LF	LF
947+00	21	1				30										
950+00	11	1		30												
955+00	53	0					1									
957+00	23	1				30										
961+00	15	1			29											
964+00	17	1				29										
980+00	19	1				29										

Station	Drainage Capacity	CMP (Pipe Drain Inlet)					Concrete Chute Inlet with Deck	RCP Cross Drainage Culvert (Single Barrel pipe unless noted)								
		# of Barrels	18"	21"	24"	30"		30"	33"	39"	42"	45"	48"	54"	66"	84"
	ft^3/s		LF	LF	LF	LF	EA	LF	LF	LF	LF	LF	LF	LF	LF	LF
Black Rock Branch Canal Reach 2																
1003+00	19	1				28										
1021+00	60	0					1									
1032+00	19	1				28										
1050+00	21	1				28										
1063+50	25	1				28										
1071+00	13	1			28											
1077+60	15	1			28											
1090+50	25	1				28										
1099+80	25	1				28										
1106+40	91	0											139			
1112+75	44	1				57										
1123+50	38	1				57										
1131+40	32	1				57										
1157+00	23	1				28										
1183+00	13	1			28											
1192+50	13	1			28											
1202+00	17	1				28										
1211+50	34	1				57										
1238+50	29	1			57											
1258+00	34	1				57										
1261+50	42	1				57										
1304+50	78	0					1									
1324+00	27	1			57											
1345+80	84	0										139				
1351+80	90	0											139			
1361+00	19	1				28										
1370+00	19	1				28										
1386+00	21	1				28										
1403+50	15	1			25											
1416+00	34	1				49										
1435+00	59	0					1									
1447+50	8	1		25												
1466+60	44	1				49										
1483+50	42	1				49										
1503+50	21	1				25										
1520+00	8	1		25												

Appendix F – Water Demand Design Criteria

BUREAU OF RECLAMATION
Technical Service Center – Denver, Colorado
Civil Engineering Services Division

Decision Memorandum No. DEC-OSSS-8120-1

Region: Pacific Northwest **Date:** November 15, 2010

Project: Columbia Basin Project, Washington

Feature: East Low Canal Enlargement/Extension, Proposed East High Canal, and Associated Water Conveyance Facilities

Subject: Water Demand Design Criteria – Feasibility Study – Odessa Subarea Special Study

Participants: Paul Ruchti, 86-68120 and Steve Robertson, 86-68140

Issue:

The proper sizing of irrigation systems requires an understanding of the water demand of the crops for which the system is being designed. This understanding is usually based on the completion of a detailed water demand study [Reclamation, 2006] that takes into account such things as crop mix, water requirements for a particular crop type, soil type, soil permeability, soil moisture content, local weather including wind and temperature, terrain (steep versus gentle slope), annual precipitation, and crop irrigation methods.

Development of the irrigation component of the Columbia Basin Project has been an on-going activity since the 1940s and over the years numerous studies have been undertaken to develop water demand requirements for use in the sizing of the project (Refer to the Appendix for a detail summary of these studies). The results of these studies have changed over time reflecting changes in crop mix and irrigation methods employed.

Table 1 – Water Application Rates by Year of Study

Year of Study	**Water Application Rate (Peak Month)**	
	On-Farm Acres / CFS	**On-Farm GPM / Acre**
1958	73.85	- - - -
1960	73.85	6.22
1968	68.10 to 72.18	6.59 to 6.22
1969	47 to 80.13	9.21 to 5.60
1970	63.32 to 72.18	6.22
1989	57.83 to 62.65	7.76 to 7.16
2007 Appraisal Study	47 to 69	9.55 to 6.50
2008 Feasibility Study (Proposed)	52.80 to 66.49	8.50 to 6.75

During the appraisal study and continuing into the feasibility study the Study Team has reviewed the information developed in the previous studies and completed computer simulations that modeled the existing East Low Canal system (including proposed modifications), the proposed East High Canal, current trends in crop mix, and current irrigation methods (sprinkler versus gravity). The goal of this effort was to develop appropriate water demand design criteria that will be used in the sizing of the irrigation systems that are part of this project. The information developed from this review and computer simulations is summarized in the table attached to this Decision Memorandum.

Most of the studies performed, and listed as references, discuss methods of establishing water demand in the canal serving many thousands of acres and over a variety of crops and lands including canal conveyance efficiency. This led to the use of average on-farm water demand values such as 3.0 acre-feet per year with 27.7 % of the annual allotment being delivered in the peak month of July.

A few of the documents [Reclamation, 1968a], [Reclamation, 1969], [SCS, 1987], [CH2M Hill, 1989], and [Reclamation, 2007] include particular crop mixes for the area which have varied over the years as the economy has changed. Several of the crops that remain within the mix have fairly high water use (evapotransportation, ET) for example: 34% alfalfa (9.74 inches in July), 25% potatoes (11.03 inches in July), and 6% corn (10.06 inches in July). The weighted average ET rate in July is 8.65 inches for the assumed diverse crop mix. The ET rates were estimated using the Washington State Guide for the Hatton, Washington area. Crop water use does vary throughout the typical month and may be higher than the monthly average used in July.

Assuming an average irrigation efficiency of 80 % (low pressure sprinkler), the water delivery over the month for July to meet the diverse crop water demand (8.65 inches) would be 10.81 inches. With the proportion of water being delivered in July of 27.7%, the annual water delivery predicted would be 39 inches per year or near the allotment of 3.0 feet being used for this project. The 39 inches of annual water delivery predicted need does not include needs for leaching nor frost protection. Some of these needs may be offset through the year by soil moisture storage and precipitation [Reclamation, 1969].

The diverse crop July rate of 10.81 inches would make the theoretical optimum water delivered be at the equivalent rate of near 6.75 gallons per minute (gpm) per acre or 66.5 acres per cubic foot per second (ac/cfs). If 10% of the fields are not under irrigation in July, the water could be delivered at the rate of 7.5 gpm/acre to the remaining area.

The typical sprinkler irrigation system application rate presently being used in the East High Area is at the rate of 7.5 gpm. This rate can be modified by changing the sprinkler package, but are typically only changed once a year. The three major crops listed above all are predicted individually to use in excess of the typical delivery rate in July. Under this condition, the crop yield would be somewhat less than optimal.

The proposed single crop ET demand for potatoes is 11.03 inches in July which is near the proposed 8.5 gpm/acre (51.7 ac/cfs) rate for single crops. There are some crops with slightly higher ET rates, such as beans (11.13 inches), mint (11.69), and orchards (13.3 inches) which could be present. The current crop mix ratio may change when higher quality and possibly less expensive surface water is provided.

Design for delivery distribution laterals for this project is planned to be via pipelines. It is necessary to size the pipes delivering to smaller tracts to have the capability to deliver at single crop demand rates. As the pipeline laterals pick up larger areas the assumption is that diverse crops are acquired. This allows the development of an "acre-Q" formula [Reclamation, 1969] [Reclamation, 2007] as outlined below.

Discussions (September 24, 2008 conference call) were held between Reclamation (PN Regional Office, Ephrata Field Office, and Technical Service Center) and the East Columbia Basin Irrigation District to arrive at a consensus as to the appropriate water demand design criteria to use in the feasibility study. The outcome of those discussions is an agreement between all parties to utilize the following water demand design criteria:

Acres Served	**Water Application Rate (Gallons per Minute per Acre Served)**
1,000 Acres and Less	8.5
1,001 to 4,999 Acres	Use straight line interpolation between 8.5 and 6.75
5,000 Acres and Greater	6.75

Risks:

The use of inappropriate water demand design criteria has the potential of resulting in a layout and sizing of the water conveyance features that are either too small or too large for the intended water delivery. In the case of an undersized system the project would not be able to supply the quantity of water needed resulting in economic impacts to the farmers because they would not be able to raise the same amount of crops they have been raising prior to construction of the project. In the case of an oversized system the cost of the features would be greater than what the system would have cost had the appropriate design criteria had been used.

Decision:

The layout and sizing of the water conveyance features of the feasibility study will be based on the crop water demand design criteria listed in the table shown above. At the Client's (PN Region) request, an in depth site specific agriculture water use study was not performed and this evaluation served its purpose for the feasibility study.

Decision Memorandum No. DEC-OSSS-8120-1
East Low Canal Enlargement/Extension and Proposed East High Canal
Crop Water Demand Design Criteria – Feasibility Study – Odessa Subarea Special Study

Recommended:

Paul M. Ruchti

Paul M. Ruchti, Team Leader
TSC – Plant Structures Group

Steve Robertson

Steve Robertson
TSC – Water Conveyance Group

Approved:

11-15-2010

Alfred I. Bernstein, Manager Date
TSC – Plant Structures Group

11/15/2010

David Edwards, Manager Date
TSC – Water Conveyance Group

REFERENCES

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Reclamation, 1958. Columbia Basin Project, Alternate Plans for Providing Irrigation Water for East High Canal Areas, Study No. 1, U.S. Department of the Interior, Bureau of Reclamation, Ephrata, Washington, July 1958.

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Reclamation, 1968a. Columbia Basin Project, East High Investigations, U.S. Department of the Interior, Bureau of Reclamation, Washington, February 1968.

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Reclamation, 2006. Feasibility Design Guidelines, General Guidelines for Preparation of Feasibility Design Reports, U.S. Department of the Interior, Bureau of Reclamation, Technical Service Center, Denver, Colorado, June 2006.

Reclamation, 2007. Odessa Subarea Special Study, Appraisal Study, Report of Findings, Columbia Basin Project – Washington, Pacific Northwest Region, U.S. Department of the Interior, Bureau of Reclamation, Technical Service Center, Denver, Colorado, 2007.

SCS, 1987. Technical Documentation – East High on Farm Water Allotment, U.S. Department of Agriculture, Soil Conservation Service, Ephrata, Washington, July 1987.

APPENDIX

TABLE SUMMARIZING PREVIOUSLY COMPLETED WATER DEMAND STUDIES

Columbia Basin Project - Odessa Subarea Special Study
Feasibility Grade Design

Steve Robertson
Date: Oct 24, 2008

This is a compilation of various water use and conveyance system sizing estimate methodologies used over the planning phases of the project from 1955 through 2008. This document concentrates on information specifically relating to the East High Area when identified within the original document. The documents present information in different formats, therefore, we needed to tabulate some values with discretion.

NOTE:
The possible need for a Water Demand study is listed on the bottom of pg 12 in *General Guidelines for Preparation of Feasibility Design Reports* at http://intra.usbr.gov/~tsc/guidance/design/FDG-General.pdf

The Ac/cfs and gpm/acr values are for diverse cropping unless otherwise identified.

Document / East High Acres / Diverted / Canal Capacity

Date	Title	Comments	Page	Class 1	Class 1 Acres	Class 2	Class 2 Acres	Class 3	Class 3 Acres	Total Acres	Diverted ac-ft annually to Canal	% Conveyance losses (seasonal)	ac-ft/acre annual Diversion	Acres/cfs Diversion (peak month)	% Conveyance losses (peak month)	Flow (cfs) (peak month)
1943_7	Board of Engineers report															
1951	Revised Irrigation Water Requirements, Problem 4, CBP											25%			16%	
1956	Actual CBP records of irrigation yr (Letter from RD, 1958 Mar 6)											42%				
1958_5	Letter - Basis for capacity determination - East High Canal	Included in Alternative Plans for Providing Irrigation Water to East High Canal Area, Study 2 (Plan 1) Supporting Materials										25%	4.83	58.60	16%	
1958_7	Alternative Plans for Providing Irrigation Water to East High Canal Area, Study 1	Determined route between Main Canal and Black Rock Coulee Reservoir														
1960_3	Alternative Plans for Providing Irrigation Water to East High Canal Area, Study 2 (Plan 1)	Unit cost developed estimates	3 5, 7	47%	172,349	35%	128,345	18%	66,006	366,700	1,769,939	25%	4.83	58.60	16%	6,258
1960_3	Alternative Plans for Providing Irrigation Water to East High Canal Area, Study 2 (Plan 1) Supporting Materials	Unit cost developed estimates	3,									25%	4.83	58.60	16%	
	A report prepared in April 1964 indicated development of a pumped-storage power operation at this plant would be feasible and desireable. Other parts of the Primary Irrigation System (2nd Bacocon siphon and tunnel) are to be enlarged to serve the revised total acreage.															
1968_2	CBP, East High Investigations		13-15	43%	164,800	39%	149,950	18%	70,750	385,500	1,860,680	25%	4.83	58.60	16%	6,579
1968_2	CBP Comletion of Irrigation Facilities, Vol VI B (Incomplete copy)	Very good source for feature details	104-							385,500						
1968_4	CBP East High Investigations	Detailed cost information	13							385,500	1,860,680	25%	4.83	62.11	16%	6,207
1969_10	East High Canal - East Low Canal Extension Plan Selection, Water Requirements (incomplete copy) Utilized effect precip and effect soil moist. (This report proposed the 47 ac/cfs <1,000 acre and 63 ac/cfs >5,000 acre formula, Open Lat, Pg 55) (This report proposed the 43 ac/cfs <2,000 acre and 65 ac/cfs >6,000 acre formula, Pipe lat, Pg 27)	Daily evapotransoration for individual crops was calculated using the Jensen-Haise Solar Radiation Method. The weigted average for three water allotment classes were estimated. After deducting precipitation and soil moisture that would be effective in meeting the consumptive use of the crops and accounting for on-farm losses, the resulting farm delivery requirement for sprinkler application for each class is as follows.	2, 3, 4, 47, 54, 55, 58,							385,500		0%	5.62		8%	
										385,500		0%	4.56		8%	
										385,500		10.5%	3.82	77.23	8%	4,991
										385,500		10.5%	3.35	77.23	8%	4,991
										385,500		10.5%	3.35	77.23	8%	4,991
1970_4	East High Plan Selection, Summary Report, Plans and Estimates Supporting Data	Many of the costs for 2007 Apprasial Study were escalated from this report	6,							385,500		34.5%	5.53	61.00	8.0%	6,320
										385,500		29.5%	5.85	63.01	8.0%	6,118
1972_11	East High Canal - East Low Canal Extension, Supporting Data for Development of Costs for Initial Plan Formulation Studies	States that project pressure pipe system offered an economic advantage to the landowner.	3,							385,500						
1987_8	Tech Doc - East High on Farm Water Allotment (SCS)															
1989_3	Continued Development of the CBP, Irrigation System Conceptual Designs and Cost Estimates (CH2M Hill)		vii, 1, 2, 8-10,													
1989_9	Draft EIS, Continued Development of the CBP	Consumptive Use calculations of 3.14 do not include volumes needed for leaching or frost protection. This was adjusted to 3.25 for system design to account for leaching and frost protection. Alt 2 was preferred.	II-3, 4							87,000	316,821	13.5%	3.64	60.51	10.5%	1,438
2007	Odessa Subarea Special Study - Apprasial Level Design	1989 crop mix was revised.								140,000	492,737	10.5%	3.52	74.19	6.0%	1,887
2008	Odessa Subarea Special Study - Feasibility Level Design	Flow rate revised Sep 2008 following meetings								115,000	385,475	10.5%	3.35	58.77	31.0%	1,957

Deliver System Design to On-Farm / Peak Crop Requirement

Date	Peaking factor (peak month)	% Allotment delivered peak month	Lateral Distribution Losses & waste		ac-ft/acre annual on-farm allotment	ac-ft/acre design annual on-farm delivered	Unit Return Flow ac-ft/ac	Lateral delivery system	% Application Eff	Acres/cfs on-farm (peak month)	gpm/acre on-farm (peak month)	Inch Crop Use Single Crop (peak month)	Inch Crop Use Diverse Crop (peak month)	Inch Crop Use (peak month)
1943_7		21%								#DIV/0!				
1951		25%			3.85				50%					0.00
1956														
1958_5		23%			3.62	3.62				73.85				
1958_7														
1960_3		23%			3.62	3.62			80%	73.85	6.22			7.99
1960_3		23%			3.62	3.62								
1968_2	0%	23%			3.62	3.62			80%	72.18	6.22			7.99
1968_2														
1968_4	0%	23%			3.62	3.62			80%	68.10	6.59			
1969_10	0%	25%	14.0%		3.62	4.93		Sprinkler SingleCrop	75%	47	9.21	11.10	8.40	11.10
	0%	25%	14.0%		3.62	4.00		Sprinkler Diverse crop	70%	61	7.47			8.40
		25%	14%		3.00	3.00	2.00	Open Lateral	71%	80.13	5.60			
		25%	0%		3.00	3.00	1.30	Gravity Pipe Lat	87%	80.13	5.60			
		25%	0%		3.00	3.00	1.30	Pressure Pipe Lt	87%	80.13	5.60			
1970_4	0%	23%	0%		3.62	3.62		Gravity Pipe	87%	73.18	6.22			8.69
	0%	23%	14.0%		3.62	3.62		Open Lat	71%	63.32	6.22			7.09
1972_11														
1987_8		27.0%			2.87	3.14			75%					
1989_3	5%	30.5%		On-farm	3.00	3.00			79%	62.65	7.16	10.9		9.09
	5%	30.5%		System Design		3.25			79%	57.83	7.76	11.87		9.85
1989_9	5%	30.5%			3.00	3.25			80%	57.83	7.76	9.98		
2007	5%	27.7%			3.00	2.99	Diverse		80%	69.00	6.50		8.36	8.36
	5%	27.7%				3.99			80%	51.79	8.67		What we should have done	11.14
	5%	27.7%				4.40	Single		80%	47.00	9.55	12.28		12.28
2008	0%	27.7%			3.00	3.26	Diverse		80%	66.49	6.75		8.68	8.68
	0%	27.7%				4.19		87%	80%	51.84	8.66		What we should have done	11.13
	0%	27.7%				4.11	Single		80%	52.80	8.50	10.93		10.93

(30 d/month) (24 hr/d) (3600 s/h) / (43560 ft2/ac) / (X ac-ft/ac) / (X% annual/month) = X acres/cfs
59.504 / (X ac-ft/ac) (X% annual/month) = X acres/cfs

(X ac-ft/ac) (12 inch/ft) (X % annual/month) (X irr eff) = X inch/ month CU

(X ac-ft/ac) (X % annual/month) (43560 ft2/ac) (7.48 gal/ft3) (month / 30 d) (d/24 h) (h/60 min) = X gpm/ac
(X ac-ft/ac) (X % annual/month) 7.54 = X gpm/ac

(X inch/ac-month) (ft/12 inch) (month/ X %) = X ft/ac in peak month

(X inch/month CU) (ft/12 inch) (43560 ft2/ac) (7.48 gal/ft3) (m/30 d) (d/24 h) (h/60 min) / (X irr eff) = X gpm/ac
(X inch/month CU) 0.629 / (X irr eff) = X gpm/ac

Columbia Basin Project - Odessa Subarea
Feasibility Grade Design

Steve Robertson

Date: Oct 24, 2008

Steve Robertson

Date: Oct 24, 2008

This is a computation of various water use and conveyance syst
of the project from 1955 through 2008 This document conce
within the original document The documents present informa

The Ac/cfs and gpm/acr values are for diverse cropping unless

Dc		Irrigation water requirement or Allot (ac-ft/ac)					Crops (on Class 1 land)													Rainfall /	Crop Yields							
Date	Title	Class A	Class B	Class C	Class D	Weighted Average (ac-ft/ac)	Forage alfalfa	Dry Beans	Cereals grain	Field crops corn	Mint	Vegetables potatoes early	potatoes late	Oil Seed Onion	Seeds misc	Fruit Vineyard	orchards	Farmstead	Total % July Diverse CU inch	Soil Moisture inches annual	Corn (bu) Land Cl 1	Corn (bu) Land Cl 2	Corn (bu) Land Cl 3	Corn (bu) Land Cl 4	Potatoes (cwt) Land Cl 1	Potatoes (cwt) Land Cl 2	Potatoes (cwt) Land Cl 3	Potatoes (cwt) Land Cl 4
1943_7	Board of Engineers report	3.25	3.50	4.00	5.00	3.66																						
1951	Revised Irrigation Water Requirements, Problem 4, CBP	3.50	4.00	4.50	5.00	3.85														7.2								
1956	Actual CBP records of irrigation yr (Letter from RD, 1958 Mar 6)																											
1958_5	Letter - Basis for capacity determination - East High Canal																											
1958_7	Alternative Plans for Providing Irrigation Water to East High Canal Area, Study 1																											
1960_3	Alternative Plans for Providing Irrigation Water to East High Canal Area, Study 2 (Plan 1)																											
1960_3	Alternative Plans for Providing Irrigation Water to East High Canal Area, Study 2 (Plan 1) Supporting Materials																											
	A report prepared in April 1964 indicated developme																											
1968_2	CBP, East High Investigations						37%		20%	18%			13%		6%		0%	6%	100%		80	73	62	57	320	304	208	
1968_2	CBP Comletion of Irrigation Facilities, Vol VI B (Incomplete copy)																											
1968_4	CBP East High Investigations						37%		20%	18%			13%		6%		0%	6%	100%		80	73	62	57	320	304	208	
1969_10	East High Canal - East Low Canal Extension Plan Selection, Water Requirements (Incomplete copy)	2.90	3.10	3.30			33%	2%	25%	5%			12%		6%		15%		100%									
	Utilized effect precip and effect soil moist.																			6.69 Annual rain 6.69/30 = 22.3% Annual soil moist 2.8/30 = 9.3% July rain 0.11/8 = 1.4% July moist 0%								
	(This report proposed the 47 ac/cfs <1,000 acre and 63 ac/cfs >5,000 acre formula, Open Lat, Pg 55) (This report proposed the 43 ac/cfs <2,000 acre and 65 ac/cfs >6,000 acre formula, Pipe lat, Pg 27)																											
1970_4	East High Plan Selection, Summary Report, Plans and Estimates Supporting Data																											
1972_11	East High Canal - East Low Canal Extension, Supporting Data for Development of Costs for Initial Plan Formulation Studies																											
1987_8	Tech Doc - East High on Farm Water Allotment (SCS)						13.7%	9.1%	13.5%	10.7%	1.9%	7.0%	11.4%	13.7%	9.6%	7.0%	2.5%		100%									
								25.1%	51.8%			23.1%							7.63									
							l	h	l	l	h	m	h	m	m	l	h											
1989_3	Continued Development of the CBP, Irrigation System Conceptual Designs and Cost Estimates (CH2M Hill)					2.41	11.6%	7.7%	19.3%	11.6%	2.0%	7.7%	11.6%	11.6%	3.8%	3.3%	3.8%	6.0%	100%									
							10.06	11.50	3.40	10.38	11.69	11.12	11.39	10.74	11.02	8.96	13.32	0.00	8.79									
1989_9	Draft EIS, Continued Development of the CBP																											
2007	Odessa Subarea Special Study - Appraisal Level Design					2.61	34%	2%	24%	6%		5%	20%	9%	0%		0%		100%									
		What we did					9.74	11.13	3.28	10.06		11.03	11.03	10.76					8.65									
2008	Odessa Subarea Special Study - Feasibility Level Design					2.61	34%	2%	24%	6%		5%	20%	9%	0%		0%		100%									
		What we did					9.74	11.13	3.28	10.06		11.03	11.03	10.76					8.65									