5.0 Black Rock Alternative Facilities

5.1 Options Considered

Reclamation's engineering evaluation of appraisal-level designs and field cost estimates of the Black Rock alternative identifies the following preliminary options for facilities to pump, store, and deliver Columbia River water to potential water exchange participants in the lower Yakima River basin:

- a Columbia River intake facility with pumping only and pump/generation,
- an all tunnel inflow conveyance and a tunnel/pipeline inflow conveyance,
- a large and a small storage dam,
- three dam types,
- a large and a small reservoir size,
- highway and utility relocations,
- a tunnel/pipeline outflow conveyance with a single-level intake,
- pump and pressure outlet facilities for irrigation delivery, both with a new Black Rock powerplant,
- several irrigation delivery system plans to convey Columbia River water to potential water exchange participants,
- a new powerplant at Sunnyside Canal.

This chapter gives a summary of site characteristics followed by a brief overview of each of the above preliminary options. Figure 5-1 provides a general layout of these facilities, and table 5-1 summarizes the major features of three preliminary Black Rock alternative configurations. Three Storage Study technical series reports present further design and field cost estimate information:

- *Appraisal Assessment of the Black Rock Alternative Facilities and Field Cost Estimates, Technical Series No. TS-YSS-2* [6]
- *Appraisal Assessment of the Black Rock Alternative Delivery System for Roza, Terrace Heights, Selah-Moxee, and Union Gap Irrigation Districts, Technical Series No. TS-YSS-3* [7]
- *Appraisal Assessment of the Black Rock Alternative Delivery System for Sunnyside Division, Technical Series No. TS-YSS-4* [8]

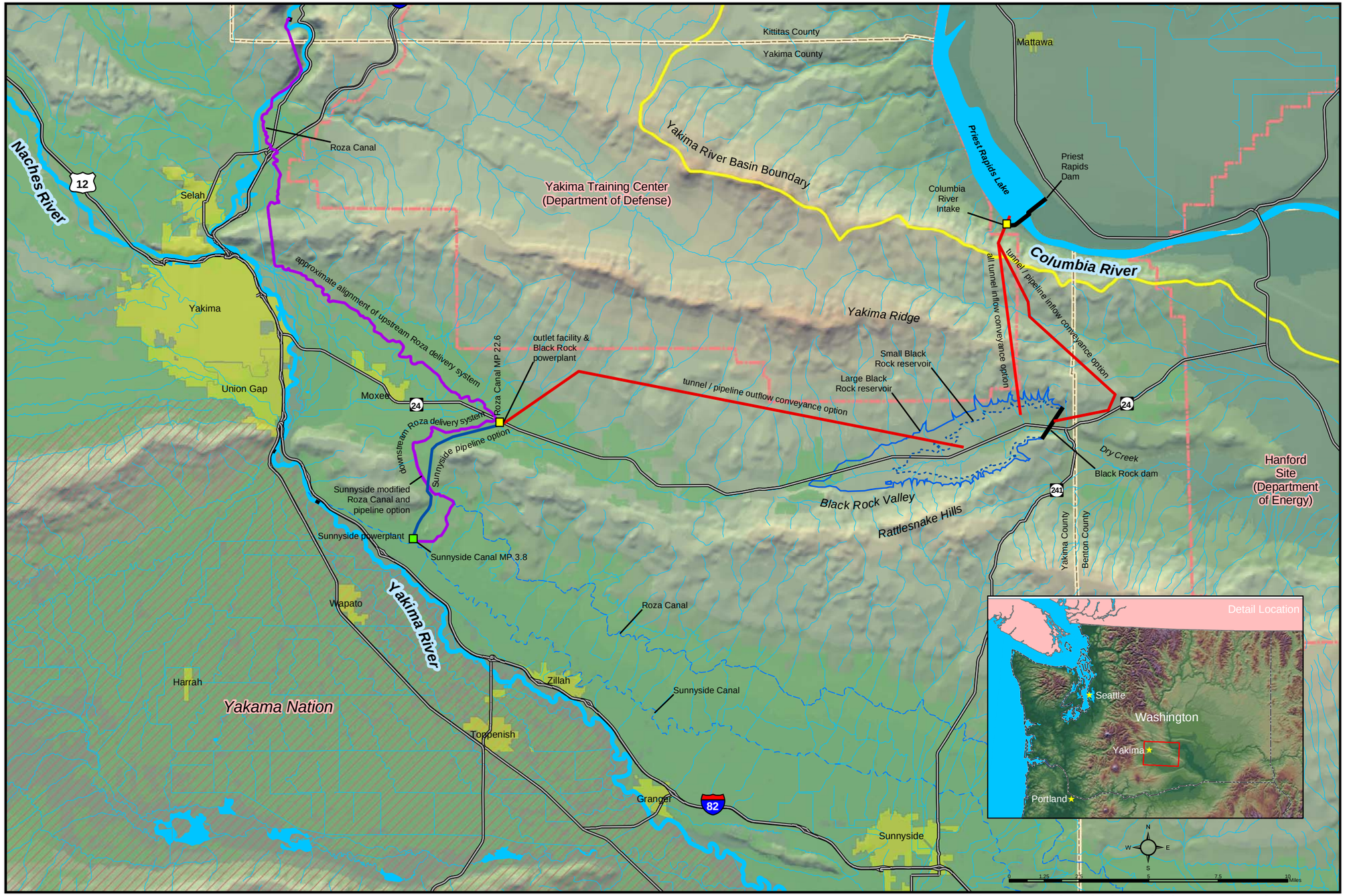


Figure 5-1. Schematic of the Black Rock alternative facility options

Table 5-1. Summary of major facilities for three preliminary Black Rock alternative configurations

OPTIONS →	LARGE RESERVOIR		SMALL RESERVOIR PUMP ONLY
	PUMP ONLY	PUMP/ GENERATION	
FACILITIES			
Priest Rapids Lake intake and fish screen			
design flow capacity	3,500 cfs		6,000 cfs
intake location	on right bank of Priest Rapids Lake		
normal Priest Rapids Lake operating water surface elevation range	481.5 to 488.0 feet		
separate tailrace channel to Priest Rapids Lake	n.a.	yes	n.a.
Priest Rapids plant	pumping	pump/generation	pumping
design flow capacity	3,500 cfs		6,000 cfs
500-cfs, two-stage spiral case pumps	three		n.a.
1,000-cfs, two-stage spiral case pumps	two		six
power design flow capacity	n.a.	3,500 cfs	n.a.
turbines	n.a.	two 1,750-cfs Francis turbines with 150-MW generators	n.a.
Inflow conveyance system			
design flow capacity	3,500 cfs		6,000 cfs
conveyance type	all tunnel		
	tunnel/pipeline	n.a.	
inlet/outlet structure	n.a.	multi-level fish screened	n.a.
Black Rock dam			
location	original Washington Infrastructure Services' damsite		
concrete face rockfill embankment dam			
crest elevation	1790.0-foot		1722.0-foot
structural height	760-foot		692-foot
crest width	40-foot		
slope	upstream and downstream 1.5:1		
central core rockfill embankment dam			
crest elevation	1785.0-foot		1717.0-foot
structural height	755-foot		687-foot
crest width	40-foot		
slope	upstream 1.75:1; downstream 1.5:1		
roller compacted concrete dam			
crest elevation	1781.0-foot		1715.0-foot
structural height	751-foot		685-foot
crest width	20-foot		
slope	upstream vertical; downstream 0.75:1		
spillway	none (See explanation in section 5.6.8.)		
low-level outlet works:			
rockfill embankment dam	upstream steel-lined concrete conduit, downstream buried steel pipe, and two jet flow gates in left abutment		
roller compacted concrete dam	fixed wheel gate, buried steel pipe, and two jet flow gates in upstream face of the dam near right abutment		

OPTIONS →	LARGE RESERVOIR		SMALL RESERVOIR PUMP ONLY
	PUMP ONLY	PUMP/ GENERATION	
FACILITIES			
Black Rock reservoir			
maximum water surface elevation	1778.0 feet		1712.0 feet
active storage capacity	1,300,000 acre-feet		800,000 acre-feet
elevation top of active storage	1775.0 feet		1707.0 feet
inactive storage capacity	157,610 acre-feet		
elevation top of inactive storage	1500.0 feet		
low-level outlet	depends on dam type		
Highway and utility relocations			
State Highway 24	relocated south of Black Rock reservoir in Rattlesnake Hills		
a buried fiber optic line and two overhead transmission lines	relocated along the realigned State Highway 24		
Outflow conveyance system			
design flow capacity	2,500 cfs		
intake structure	single-level fish screened		
conveyance type	tunnel/pipeline		
Black Rock outlet facility			
location	adjacent to Roza Canal MP 22.6		
pump delivery	all water through the powerplant and bypass into Roza Canal		
	1,500-cfs capacity powerplant		
pressure delivery	upstream bifurcation to pressurized pipeline delivery system		
	900-cfs capacity powerplant		
powerplant bypass design flow capacity	2,500 cfs		
Sunnyside powerplant and bypass			
powerplant capacity	900 cfs		
powerplant bypass design flow capacity	1,250 cfs		

5.2 Level of Detail

This chapter summarizes the engineering evaluation of appraisal-level designs of the primary components of the Black Rock alternative. The engineering evaluation is based on available design data from past WIS and Reclamation work and from more recent work as documented in the Assessment technical series reports listed in chapter 10.0.

Preliminary identification and sizing of the Black Rock alternative facilities are based on the water exchange, engineering judgment, limited analyses, available design data, and professional assumptions. Accordingly, the preliminary identification and sizing of facilities for the Black Rock alternative set forth in this Assessment will undoubtedly change if more detailed, feasibility-level analyses are done to refine these designs.

5.3 Site Characteristics

5.3.1 Topography

Reclamation and Aerometrics, Inc., developed aerial photogrammetry in August 2003 at approximately a 1:10,000 scale. The aerial photogrammetry covers the potential Columbia River intake area, Black Rock reservoir site, Black Rock outlet area, and most areas in between. These data were then used to generate grids, 2-foot contours, and orthophotos. Existing USGS 7.5-minute quad maps with 20-foot-contour intervals provided topographic information for locations outside the coverage area, including a small portion of the outflow conveyance system between the Black Rock reservoir site and Roza Canal and the Roza and Sunnyside delivery systems.

5.3.2 Geology

The *Appraisal Assessment of Geology at a Potential Black Rock Damsite* [9] report and Columbia Geotechnical Associates, Inc., investigations [10] document Reclamation's geologic investigations at an alternate Black Rock damsite and builds upon earlier work conducted by WIS for Benton County [2 and 11] at the original damsite. A brief geologic summary of the potential Black Rock site follows.

5.3.2.1 Regional Geology

The Black Rock site lies in the northwest-central portion of a large area covered by basalt flood lava referred to as the Columbia Plateau, which covers extensive portions of eastern Washington, northern Oregon, and western Idaho. The sequence of lava flows reaches thicknesses in excess of 10,000 feet and is known as the Columbia River Basalt Group. Beneath the alternative site, the lava flows form the foundation bedrock for the Black Rock damsite. Individual lava flows average about 100 feet thick and extend to a depth of at least 600 feet in test drilling completed to date at the damsite. The time periods between individual eruptions range from hundreds to tens of thousands of years, allowing for the deposition of sediments between lava flows. These sediments include sand and gravel bars laid down by the ancestral Columbia River and finer grained silt and clay layers that were deposited in shallow lakes formed by temporary damming of the Columbia River by the lava flows. These sediment layers are collectively known as the Ellensburg Formation.

Following the onset of the eruptive activity, the western portion of the Columbia Plateau was subjected to north-south compression that folded the flat-lying lava flows and sediment layers into a series of generally east-west trending ridges and valleys known as the Yakima Fold Belt. These ridges are typically asymmetrical in that the south slope is gently inclined while the north ridge slope is steeply folded. Low-angle thrust faults are present at the base of the steeply folded north slopes in each of the Yakima Fold Belt ridges. Landslides frequently are present in the Yakima Fold Belt and often occur on the steep north slopes where sliding happens in weaker layers of the Ellensburg Formation sediments that are sandwiched between the more rigid lava flows. This configuration exists at the Columbia River intake area, along the inflow conveyance

system (both of which are bound by or within Umtanum Ridge), and at the Black Rock damsite (which lies between the Yakima Ridge on the north and Horsethief Mountain/Rattlesnake Hills ridge on the south).

The volcanic bedrock is overlain by sedimentary layers derived from ancient river and lake systems and volcanic eruptions in the Cascade Range. These sediments are known as the Ringold Formation. More recent wind-blown silt and alluvial deposits overlie the Ringold Formation sediments. The Ringold Formation materials form the upper foundation at the Black Rock damsite and are expected to provide a firm foundation capable of supporting a large dam. The recent alluvial and wind-blown deposits would likely be removed prior to dam construction.

5.3.2.2 Geology of Columbia River Intake and Inflow Conveyance Areas

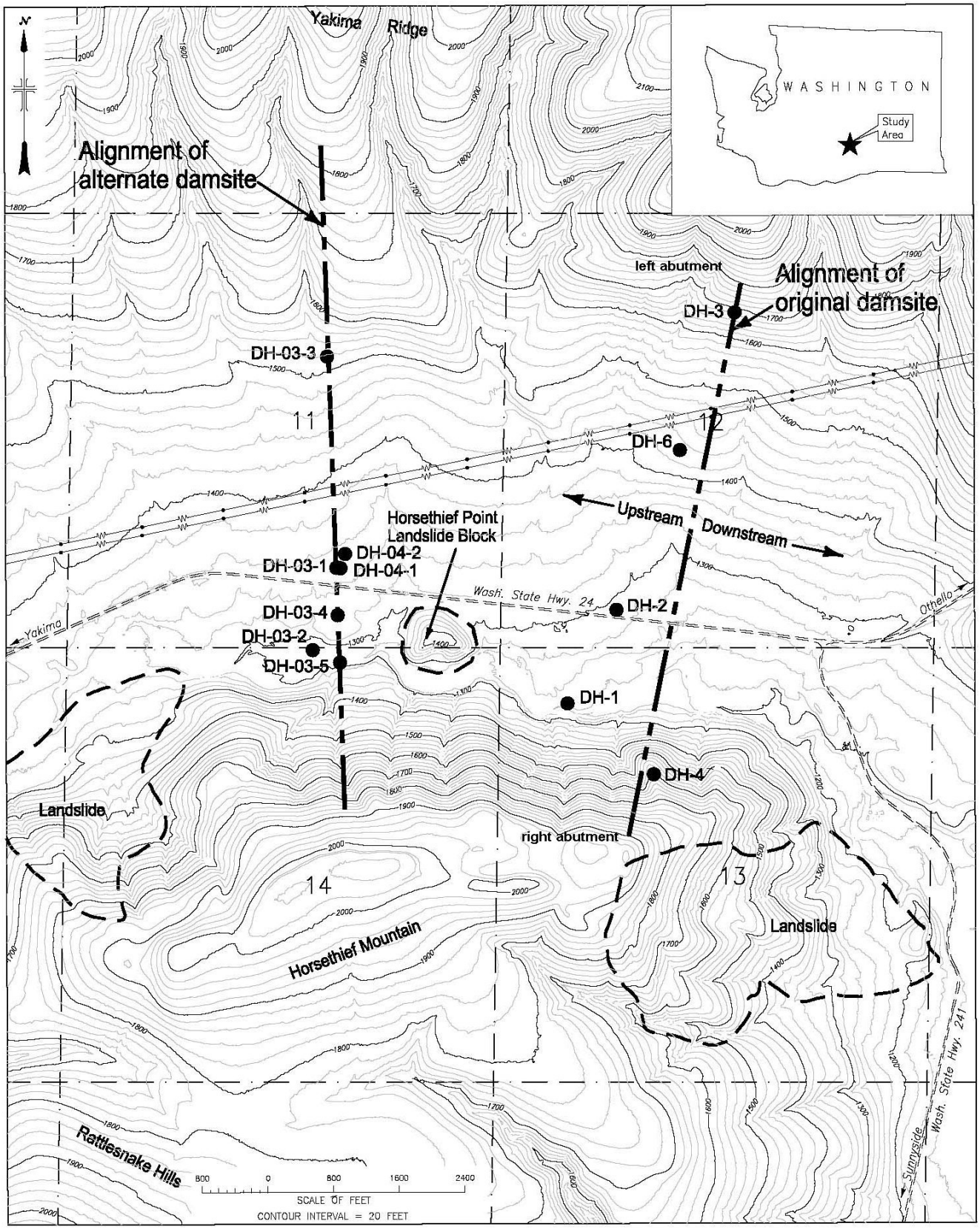
Information that the Grant County Public Utility District (Grant PUD) used to design and construct Priest Rapids Dam provides the primary basis for geologic conditions at the intake area (adjacent to Priest Rapids Lake) and along the inflow conveyance system. The intake and pumping plant structure likely would be founded on either Priest Rapids Basalt of the Columbia River Basalt Group or on terrace deposits laid down by the Columbia River. The pumping plant would likely be located in an area with less than 20 feet of unconsolidated terrace gravel lying above the Priest Rapids Basalt. If the basalt is dense and not highly fractured, excavation for the pumping plant likely would need relatively minor water control.

The inflow tunnel would penetrate Umtanum and Yakima Ridges, which are composed of folded and faulted Columbia River basalt flows layered with Ellensburg Formation sediments. The north slope of Umtanum Ridge at the nearby Priest Rapids Dam is overturned and dips to the north. An upper fault (Buck thrust) and a lower fault (Umtanum fault) define the overturned flows on the north side of the fold. Landslides exist along the steep, overturned north slope of Umtanum Ridge.

5.3.2.3 Geology of Black Rock Damsite

The geology of the Black Rock damsite was developed from 2003 WIS investigations [11] at the original damsite. Reclamation performed additional investigations [9] in 2003 and 2004 at a potential alternate damsite that the 2002 WIS report [2] identified about 1 mile west or upstream from the original site. Figure 5-2 shows the locations of the two damsites and the exploratory drill holes to date at each site. Both damsites and the reservoir site are underlain by the same geologic units: Recent wind-blown silt and alluvial deposits, Ringold Formation deposits, and Columbia River basalts with interbedded Ellensburg Formation sediments. The geologic and engineering properties of the shallow and deep foundation materials are similar at both damsites.

Figure 5-2. Locations of the two Black Rock damsites and exploratory drill holes



Tables 5-2 and 5-3 identify the depth of drilling in the exploratory holes and the depths at which bedrock was encountered.

Table 5-2. Summary of test drilling WIS performed at original Black Rock damsite

Hole Number	Location	Hole Depth (feet)	Depth to Bedrock (feet)
DH-1	maximum section near upstream toe	236.0	215.5
DH-2	left of maximum section near upstream toe	230.0	156.6
DH-3	left abutment	250.0	0.8
DH-4	right abutment	115.0	10.3
DH-5	hole was cancelled	-	-
DH-6	left abutment near upstream toe	150.1	11.6

Table 5-3. Summary of test drilling Reclamation performed at alternate Black Rock damsite

Hole Number	Location	Hole Depth (feet)	Depth to Bedrock (feet)
DH-03-1	left of maximum section	169.6	146.9
DH-03-2	maximum section	73.9	*
DH-03-3	left abutment	99.0	87.0
DH-03-4	maximum section	105.5	*
DH-03-5	maximum section	106.6	*
DH-04-1	left of maximum section	562.3	145.3
DH-04-2	left of maximum section	530.0	144.0
* Bedrock was not encountered in drill holes DH-03-2, DH-03-4, or DH-03-5.			

Figure 5-3 is a generalized geologic section that identifies the geologic units and where they occur in the alternate Black Rock damsite area. Figure 5-4 identifies the vertical depths and thicknesses of the geologic members present in the alternate Black Rock damsite area.

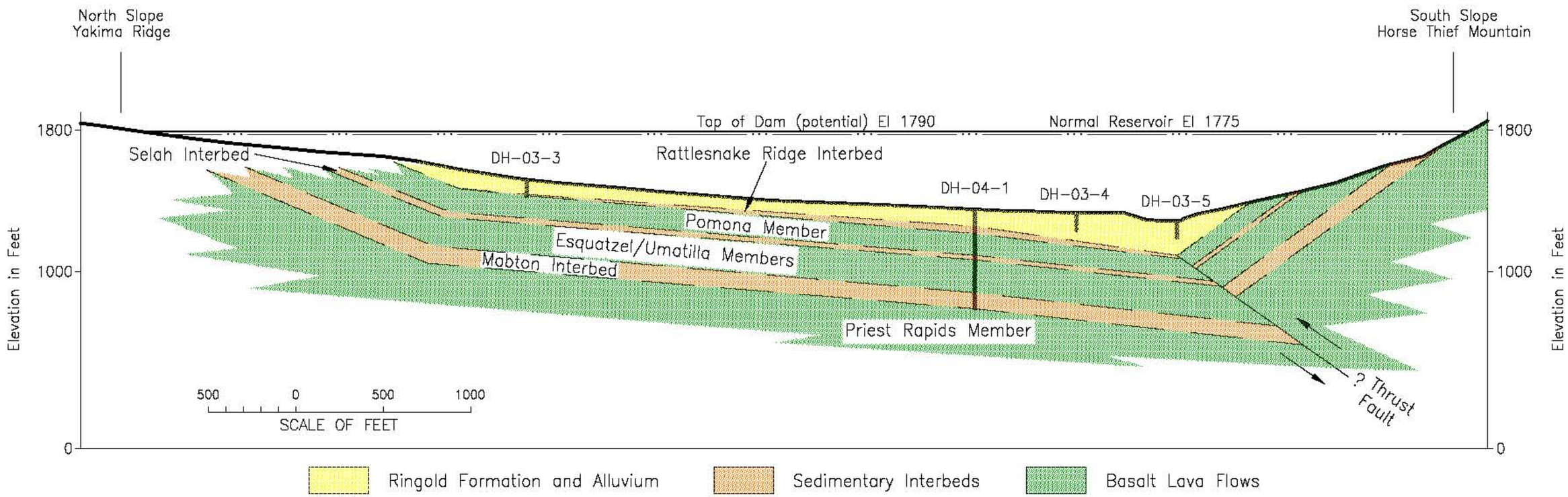


Figure 5-3. Generalized geologic section through the alternate Black Rock damsite

Period	Epoch	Group	Formation	Member (*)	Age (Ma)	Thickness (ft)	Depth (ft)
Quaternary	Recent / Pleistocene		Loess/Alluvium		0.05–0.1	31.7	31.7
Tertiary	Pliocene		Ringold		3.4–8.5	58.8	90.5
	Miocene	Columbia River Basalt Group	Saddle Mountains Basalt	Rattlesnake Ridge		28.0	118.5
				Pomona (peperite)	12	136.3	145.3
				Pomona (basalt)			254.8
				Selah		24.6	279.4
				Esquatzel	13	187.6	
				Umatilla			467.0
				Mabton		88.8	555.8
			Wanapum Basalt	Priest Rapids	14.5	>6.5	562.3

Bottom of Hole

Ringold Formation and Alluvium

Ellensburg Formation Sedimentary Interbeds

Columbia River Basalt Group Lava Flows

Figure 5-4. Stratigraphic section of geologic units in drill hole DH-04-1

Geologic explorations by both WIS and Reclamation confirm the presence of a thick deposit of sediments (overburden) overlying the basalt bedrock. Drilling indicates the overburden in Black Rock Valley is thicker than 200 feet near the base of the right dam abutment near Horsethief Mountain and may average more than 100 feet thick across the right center portion of the valley. Although the overburden includes some wind-blown silt and recent stream deposits, most appears to consist of the Ringold Formation. Based on the limited explorations, the Ringold Formation appears to be a highly variable deposit, consisting mainly of basalt gravels and cobbles in a sand matrix, but also including numerous layers of sand, silt, and clay. These varying materials range from poorly to well consolidated and from soft to hard. Portions of the Ringold Formation, particularly the deeper layers, are a dense, hard deposit of gravels and cobbles. This type of material is expected to be a firm, nonliquefiable foundation capable of supporting a large dam, although some type of cutoff to bedrock may be needed to minimize seepage.

The deep foundation at the damsite is composed of volcanic rocks of the Columbia River Basalt Group which are interbedded with sediments of the Ellensburg Formation. Individual basalt flows are up to 100 feet thick. The bedrock consists of a number of basalt flows with sedimentary interbeds sandwiched between them. Core samples obtained from test drilling indicate that some of the basalt flows plowed into the previously deposited sediments and rafted the older sediments onto the top of the new flow or mixed the sediments within the basalt (locally termed peperite). Zones of higher permeability generally exist at the top or bottom of flows due to shearing and intermixing during deposition, or from differences in how the flows cooled. The rock quality could vary significantly within flows or between flows.

The extent and causes of potential landslides in the Black Rock site need to be established. Landslides in the Yakima Fold Belt generally form on sloping limbs of the anticlines, due to failure of the lower strength sedimentary interbeds. The primary areas where these conditions exist include Horsethief Mountain (south abutment of the Black Rock damsite), and potentially along the south rim of the Black Rock reservoir area.

Several potential landslides have been identified on the Horsethief Mountain anticline (see figure 5-2 for locations). One landslide is located on the north slope of the ridge upstream from the alternate damsite. Horsethief Point is a prominent butte that projects from the valley floor upstream from the original damsite. This point appears to be a buried remnant of a landslide block that has moved off Horsethief Mountain. The third slide area is downstream from the damsite on the east slope of Horsethief Mountain near SH 241.

Additional investigations are needed to evaluate the potential impacts of a reservoir located on the landslide areas. These investigations would include identifying the sites, evaluating the impacts associated with the highway relocation along the south rim of the reservoir, evaluating the slope stability at the damsite during and after construction of the dam and appurtenant structures, and evaluating the reservoir rim stability during reservoir operation.

Excavating the dam's foundation down to solid bedrock may require removal of approximately 200 feet of overburden material. Further geologic investigations of the varying bedrock composition and quality would provide a better understanding of bedrock characteristics,

foundation conditions, and the amount of material to remove beneath the dam. These investigations may find that complete excavation of the Ringold Formation down to the top of bedrock across the entire footprint of the dam would be required, if unsuitable layers of soft material were found at depth.

5.3.2.4 Geology Along Outflow Conveyance System and at Black Rock Powerplant and Delivery System Areas

Reclamation's geologic investigations of the Black Rock alternative have concentrated initially on the alternate damsite location. The geology of the potential outflow conveyance system, the Black Rock powerplant area, and the delivery system areas to the west of the damsite is poorly known. Further geologic investigations are needed to provide a better understanding of bedrock characteristics, foundation conditions, and groundwater conditions at these sites.

Findings: The original damsite identified by WIS is preferable to the alternate site investigated by Reclamation.

The deeper layers of the Ringold Formation are a dense, hard deposit of gravels and cobbles. This type of material may be expected to be a firm, nonliquefiable foundation capable of supporting a large dam, although some type of cutoff to solid bedrock may be required on the upstream portions of the dam. Or, complete excavation to bedrock may be necessary beneath the entire footprint of the dam, requiring the removal of approximately 200 feet of overburden material.

Further investigations are needed to determine whether a Black Rock reservoir could cause currently stable beds of three ancient landslides identified on Horsethief Mountain ridge to become saturated and start moving.

Further investigations are needed to determine whether three ancient landslides identified on Horsethief Mountain ridge could impact the realigned highway.

5.3.3 Groundwater

The *Appraisal Assessment of Hydrogeology at a Potential Black Rock Damsite* [12] report documents Reclamation's hydrogeologic investigations. A brief summary of the Black Rock damsite follows.

The Columbia River basalts and interbedded sedimentary units that underlie the Black Rock dam and reservoir site comprise the framework for the groundwater flow system. The primary

water-bearing zones are generally limited to the tops of the basalt flows and within coarse-grained layers of the sedimentary interbeds. Groundwater can flow horizontally or vertically, in response to head differences and the hydraulic properties of the geologic materials. Hydraulic conductivity is a measure of the ability of water to flow through a unit and is generally reported in feet per day. The flow tops have relatively high horizontal hydraulic conductivity. The dense flow interiors have very low horizontal conductivity, but contain vertical joints and fractures that, if not filled with secondary minerals, could accommodate vertical groundwater movement. Structural folds and faults may impede groundwater flow or act as vertical flow pathways.

Hydraulic properties of the unsaturated zone above the water table are important at the Black Rock site since the differences within and between the distinctive unsaturated materials in the reservoir basin would result in variable downward migration and horizontal flow of infiltrated water. Alternating layers of fine-grained and coarse-grained sediments create conditions that tend to enhance horizontal spreading. At the alternate damsite, the unsaturated zone includes all of the overburden sediments and the upper-most basalt member, the Pomona Basalt. Though these units are currently dry, they would become saturated from reservoir leakage if the Black Rock alternative were built.

Hydrologic testing of a borehole (DH-04-02) located at the alternate damsite was conducted from April to June 2004. The purpose was to determine hydraulic properties of selected hydrogeologic units, identify hydraulic boundaries, and assess the capacity for vertical communication (leakage) between units. In addition, groundwater samples were collected for hydrochemical and isotopic analyses, and hydraulic head information was studied to determine hydraulic relationships between the units at this location. These studies were part of the initial assessment phase for characterizing the Black Rock Valley hydrogeology and assessing the potential impact that seepage from a reservoir could have on local and adjacent groundwater conditions.

Five unsaturated zones and two groundwater zones were successfully characterized in DH-04-02 using a suite of hydrologic test methods and analyses. Hydraulic conductivity values range from 0.04 to 2.8 feet per day in the unsaturated zones and 0.03 to 2.69 feet per day in the groundwater zones. These values fall within the lower range reported for the Hanford Site for comparable hydrogeologic units [13]. Groundwater samples collected at DH-04-02 have similar characteristics and chemistry as those within the Ellensburg Formation/Saddle Mountains Basalt at the Hanford Site and surrounding Pasco basin [13]. The groundwater samples from DH-04-02 appear to be relatively young and not altered by a long residence time within the groundwater flow system. Hydrologic testing at DH-04-02 did not identify any faults or discontinuities within about 300 feet of the borehole.

During the current investigations, no unconfined or perched water table conditions were found. In DH-04-02, the first groundwater was encountered at the bottom of the Pomona Basalt and top of the Selah sedimentary interbed, at about 254 feet below ground surface. The water level rose in the well to about 194 feet.

The Pomona Basalt appears to be a hydraulic barrier to vertical downward leakage, at least at the site of these investigations. The Pomona Basalt flow interior may inhibit large quantities of

vertical leakage. Additional Pomona Basalt characterization is needed to confirm that it is a vertical hydraulic barrier and to verify the hydrologic characteristics of the unit across the entire reservoir basin. Where the basalt flow tops and interbeds are exposed or thinly covered by sediments on the adjacent ridges, large amounts of reservoir leakage could still occur.

The Selah interbed is underlain by the Esquatzel and Umatilla Basalts, which in turn are underlain by the Mabton sedimentary interbed. Based on testing responses, leakage appears to be pervasive through these basalts, and the two sedimentary interbeds are hydraulically connected.

Total leakage from a Black Rock reservoir would vary depending on the hydraulic gradients and operations of the reservoir, the final design selected for the dam foundation (whether the overburden were completely or only partially removed beneath the dam), and many other conditions. Further investigations would assess the potential for reservoir seepage into the abutments and the variability of hydraulic characteristics within the hydrogeologic units that underlie the reservoir and damsite.

5.3.3.1 Capability of Reservoir Basin to Retain Stored Water

Findings: **No unconfined or perched water table conditions were found. Groundwater was first encountered at a depth of about 254 feet in the Selah interbed at the bottom of the Pomona Basalt.**

The Pomona Basalt appears to be a hydraulic barrier that may inhibit vertical leakage in the reservoir basin.

Significant amounts of reservoir leakage could occur where the basalt flow tops and interbeds (i.e., Selah and Esquatzel/Umatilla) are exposed or thinly covered by sediments on the adjacent ridges if the exposure area would be within the reservoir pool.

5.3.3.2 Movement of Groundwater

Findings: **Vertical leakage appears to be pervasive between the Selah interbed, the Esquatzel and Umatilla Basalts, and the Mabton interbed. These units are hydraulically connected and constitute a single groundwater flow system.**

Groundwater samples appear to have similar characteristics and chemistry as those within the Ellensburg Formation/Saddle Mountains Basalt at the Hanford Site and surrounding Pasco basin. The groundwater samples appear to be relatively young

and not altered by a long residence time within the groundwater flow system.

The studies conducted to date do not identify the resultant effects on down gradient areas such as the Hanford Site. However, if reservoir leakage were to bypass the dam through the underlying overburden sediments or reach the Selah interbed, there could be significant regional effects on the groundwater system.

5.3.4 Seismotectonics

The Technical Memorandum titled *Probabilistic Seismic Hazard Assessment for Appraisal Studies of the Proposed Black Rock Dam* [14] documents the preliminary characterization of the earthquake potential at Black Rock damsite. Probabilistic Seismic Hazard Assessment (PSHA) is a technique that provides an assessment of the annual levels of earthquake ground motions that the site might experience based on the rates of seismic activity and fault movements in the region surrounding the site. Peak Horizontal Acceleration (PHA), a measure of very high frequency earthquake ground motions, can be estimated through PSHA and was used in the preliminary assessments of the potential Black Rock damsite.

Seismic hazard information is used to guide engineering decisions on the design and placement of the dam and related structures. High levels of earthquake ground motion can potentially lead to liquefaction (a loss of material strength that can result in large areas of slope failure) of saturated, lower density soils. Other potential concerns include the stability of natural and engineering slopes and the effects of potential fault displacements on the dam and related structures.

The initial assessment indicates that the Black Rock damsite lies in an area of relatively high earthquake potential. For example, at a return period of 10,000 years, the estimated mean PHA is about 0.95 g, a level of ground shaking that might be associated with the occurrence of magnitude 6 to 7+ earthquakes relatively near to the site. Faults that are associated with the Yakima Fold Belt near the Black Rock damsite are the main sources of potential ground motion. These include the large fold on Horsethief Mountain, which is related to a low-angle thrust fault (a part of the Black Rock Valley fault) that surfaces in the lower portion of the right (south) dam abutment and dips to the south beneath Horsethief Mountain. Because of its proximity to the site, the Black Rock Valley fault is the largest contributor to the initial estimates of PHA for the site. The Cascadia Subduction Zone (a deep fault zone along the coast of Washington and Oregon that is capable of producing very large magnitude earthquakes) is not a major contributor to the PHA at the damsite.

While the Black Rock Valley fault has not been studied in sufficient detail to define its activity, it is assumed at this stage of study that the fault may be capable of large-magnitude earthquake and that associated fault offsets within the dam footprint could range from a few centimeters to several meters. Given the orientation of the east-west folds comprising the Yakima Fold Belt,

which includes Black Rock Valley, the orientation of the displacements would be in the north-south (cross valley) direction reflecting compression of the folds. Several secondary faults, scarps, and lineaments that appear to be related to the fold atop Horsethief Mountain are also potential sites of secondary faulting, fissuring, and landslides.

Findings: Based on presently available information, design ground motions for Black Rock damsite and associated facilities may be relatively large due to the potential activity of faults with close proximity to the sites.

Characterization of the Black Rock Valley fault has a significant impact on the evaluation of potential seismic hazards at the Black Rock damsite. No detailed studies are presently available for this fault. Detailed investigations of the Black Rock Valley fault and potentially related geologic structures within the Yakima Fold Belt are required to reduce the high levels of uncertainty associated with the current estimates.

5.3.5 Probable Maximum Flood

Reclamation's preliminary probable maximum flood study [15] offers insight into the hydrologic hazard associated with a potential Black Rock reservoir and assists in sizing the dam. The study examines how much storm runoff could be generated within the 61.2-square-mile Black Rock reservoir drainage area.

Typically, winter conditions (November through March) would produce the maximum precipitation amounts; however, the June through October summer months in this region would produce nearly 20 percent larger probable maximum precipitation. Therefore, this Assessment includes both the winter and summer general storm conditions. Large antecedent floods are most likely to occur in the winter months but are not likely at all in the summer months. Table 5-4 summarizes the probable maximum floods identified in this study.

Table 5-4. Probable maximum floods for a Black Rock reservoir

Flood Description	Peak (cfs)	Volume (acre-feet)	Duration (days)
winter general probable maximum precipitation storm (with 100-year antecedent rain flood between November and March)	20,200	29,100	10.5
summer general probable maximum precipitation storm (with no antecedent flood between June and October)	28,900	28,700	3.5
summer local thunderstorm (with no antecedent flood between June and October)	74,900	17,000	1

Findings: Concerns about a spillway channel and potential environmental impacts of discharging Columbia River water into Dry Creek,

which drains into the Yakima River, led to designs for a Black Rock dam high enough to completely contain a full probable maximum flood in the reservoir.

5.4 Columbia River Intake Facilities

Based on the Columbia River water availability assessment [3], this Assessment analyzes a large reservoir option with either a 3,500-cfs intake and pumping plant or a 3,500-cfs pump/generation facility that would permit Columbia River water stored in a Black Rock reservoir to return to the Columbia River to generate power. It also analyzes a small reservoir option with a 6,000-cfs intake and pumping plant on the Columbia River. This section first describes the intake facilities, then the pumping plant facilities, and last, the pump/generation plant facilities.

The appraisal-level design for the three intake options would include:

- an alternate access road along the alignment of an abandoned railroad track on the right side of the Columbia River from SH 24 to the intake facilities
- construction of a circular-type cofferdam in Priest Rapids Lake to permit construction of the intake facilities
- excavation of about 20 feet of overburden down to competent basalt.

5.4.1 Intake, Trashracks, and Fish Screens

The identified site for the intake structure would be on the right bank of Priest Rapids Lake about 3,600 feet upstream from Priest Rapids Dam. The intake channels and fish screen design would meet the maximum and minimum Priest Rapids Lake operating water surface elevations and provide sufficient freeboard to prevent overtopping during flood events. The fish screens and bypass pipes would meet the NOAA Fisheries salmonid criteria which include channel velocities, screen approach velocities, screen sweeping velocities, exposure time along screen, maximum bypass pipe flow velocity, and minimum radius of bypass pipe bends. A description of the appraisal-level designs for the three intake options follow.

5.4.1.1 3,500-cfs Pump Only Option

In this option, the 2,366-foot-long intake channel would run from the intake at Priest Rapids Lake to the face of the pumping plant. Maintenance roads would parallel each side of the channel. Guardrails and fencing would provide for safety protection. The initial 1,412 feet of the intake channel would have three channel bays with vertical structural concrete walls. Two of the channel bays would be each sized for 1,500-cfs flows, with a third channel sized for 500 cfs, totaling the 3,500-cfs flow capacity. Each channel would have bulkheads and guides to isolate that channel while maintaining the water diverting operation. Downstream from the fish screen, the three intake channels would open to a 608-foot-long single channel section. It would then widen into a 346-foot-long transition to the pumping plant.

Trashracks with an automated rake and a conveyor system would collect trash at the inlet. Three top-sealed radial gates at the reservoir intake would isolate the channels for emergency or short-term maintenance of the fish screens and could also regulate downstream water surfaces. An access bridge deck over the inlet would allow access across the intake channel.

The intake fish screens would be vertical, stainless-steel, wedge wire panels installed within metal guide/support structures. Adjustable baffles in guides directly downstream from the screens would provide for uniform flow distribution over the screen surface. Horizontal brush-type fish screen cleaners would clean and remove debris from the screens. Metal barrier panels above the screens would extend higher than the maximum design operating water surface.

5.4.1.2 6,000-cfs Pump Only Option

This option consists of four 1,500-cfs channel bays similar to the 1,500-cfs channel bays of the 3,500-cfs option (see section 5.4.1.1). The 2,340-foot-long intake channel would include the structural concrete channel bay section, an excavated unreinforced concrete-lined channel, and a transition to the pumping plant. The access bridge, maintenance roads, guardrails, and safety fencing would be similar to the 3,500-cfs option. Since this option would have four bays, the total channel width would be greater than the 3,500-cfs option. Four sets of bulkheads and guides would be available for both upstream and downstream use. Four top-sealed radial gates would isolate the channels for fish screen maintenance and to regulate downstream water surfaces. Larger trashracks and rakes would accommodate the extra bay for this option. The four bypass pipes and outfalls for the four fish screen bays would be the same as for the 3,500-cfs option.

5.4.1.3 3,500-cfs Pump/Generation Option

This option would have the same intake arrangement and criteria as described in section 5.4.1.1 for the 3,500-cfs intake channel, except that the intake area configuration would also accommodate a tailrace for power generation units. The channel layout would be the shortest possible path back to the reservoir.

The 483-foot-long tailrace channel, which would lie between the intake channel and Priest Rapids Lake, would consist of a transition from the powerplant face to a structural channel with vertical walls. The design velocity of the tailrace channel would be 10 feet per second during the minimum water surface elevation with a 3,500-cfs maximum power generation discharge. The velocity would decrease as the reservoir rises to the maximum water surface. Maintenance roads would parallel either side of the channel. An access bridge would align with the centerline of the existing Priest Rapids Dam embankment.

5.4.2 Pumping Plant and Switchyard

The pumping plant and service yard would be set at an elevation compatible with the surrounding area and to reduce their visibility from the Wanapum Indian Village. The plant would be a reinforced-concrete, indoor-type structure with a precast-concrete double-tee roof

structure. A low, unit bay structure and a raised service bay superstructure would allow access and handling of equipment into and out of the building structure from the service yard.

5.4.2.1 3,500-cfs Pump Only Option

In this option, the 3,500-cfs pumping plant would house three 500-cfs pump units and two 1,000-cfs pump units that would require 62 feet of submergence below the minimum intake water surface elevation (see table 5-5). The 1,400-foot steady state lift from Priest Rapids Lake to a Black Rock reservoir is very high. Therefore, the spiral-case-type, two-stage pumping units would accommodate water in the months when downstream Columbia River flow targets would restrict the volume of water that could be pumped from the river. The smaller units would provide flexibility of operations, reduce the unit submergence requirements, and permit unit maintenance without sacrificing a large percentage of the plant capacity.

The plant would be equipped with two 200-ton overhead cranes acting in tandem in the unit bays and a 100-ton overhead crane in the service bay. The plant would have space for unit disassembly and auxiliary mechanical and electrical equipment, although specific equipment was not identified. A new 500-kilovolt powerline would run from the pumping plant to the Midway substation, which is 6 miles east of the Priest Rapids switchyard. The new switchyard would include transformers, circuit breakers, disconnect switches, and a circuit breaker and disconnect switches for the tie into the Midway substation.

Table 5-5. Preliminary Priest Rapids 3,500-cfs pumping plant unit data

Unit Data	500-cfs Units	1,000-cfs Units
number of pump units	three	two
type of units	two-stage spiral case	two-stage spiral case
design discharge	500 cfs	1,000 cfs
design head	1,400 feet	1,400 feet
motor	98,000 hp	200,000 hp
minimum impeller submergence	62 feet	62 feet
maximum spiral case dimension	18.2 feet	26.0 feet
top elevation of suction tube invert	468.0 feet	468.0 feet
guard valve	60-inch spherical	78-inch spherical
guard valve weight	110,000 lbs. each	175,000 lbs.

5.4.2.2 6,000-cfs Pump Only Option

This option would have a similar pumping plant and switchyard arrangement as the 3,500-cfs pump only option (see section 5.4.2.1), except the enlarged structure would accommodate six 1,000-cfs pumping units.

5.4.3 3,500-cfs Pump/Generation Plant and Switchyard

This option would have a similar arrangement as the 3,500-cfs pump only option (see section 5.4.2.1), except the enlarged structure and yard would accommodate two 1,750-cfs Francis

turbines and a tailrace channel adjacent to the pumping plant intake channel. Table 5-5 shows the preliminary pump unit data; table 5-6 shows the preliminary powerplant turbine data.

Table 5-6. Preliminary Priest Rapids 3,500-cfs turbine unit data

Unit Data	1,750-cfs Unit
number of turbine units	two
type of units	Francis
design discharge	1,750 cfs
design head	1,130 feet
speed	400 rpm
assumed unit efficiency	90 percent
power output at design head	150 MW
minimum turbine submergence	29.8 feet
maximum scroll case dimension	26.5 feet
bottom elevation of draft tube	431.5 feet
guard valve	102-inch spherical

5.5 Inflow Conveyance System

The inflow conveyance system options evaluated to transport water from the Columbia River intake to a Black Rock reservoir include 3,500-cfs and 6,000-cfs pump only options and a 3,500-cfs pump/generation option.

The inflow conveyance facilities would have capacity to carry 3,500- to 6,000-cfs entering the reservoir via an outlet structure at approximately 100 feet above the valley floor during initial filling of the reservoir's inactive pool. To minimize erosion of the hillside until the reservoir storage reached the outlet elevation, a short open channel would direct flow from the outlet structure into a 20- to 24-foot-diameter steel pipe. The pipe would carry the flow downhill toward the valley bottom where the pipe would terminate with a 90-degree upward bend. A large concrete thrust block would decrease the water pressure as it exits the pipe.

5.5.1 3,500-cfs Pump Only Option

Design factors and assumptions for the 3,500-cfs pump only option led to two separate inflow conveyance systems (an all tunnel option and a tunnel/pipeline option). Both alignments would encroach on the Yakima Training Center to some extent. Discussions with Training Center representatives would be necessary to address alignment concerns.

5.5.1.1 All Tunnel Inflow Conveyance

The all tunnel option would include a manifold connected to a 17-foot-diameter tunnel sloping steadily up towards a Black Rock reservoir. The tunnel shaft would be similar to the WIS design, but would be simplified to have a constant slope. The tunnel portal would be located just outside the pumping plant switchyard with the centerline at elevation 495 feet. The end of the tunnel would be located at elevation 1440 feet, an elevation to prevent a negative down surge in

the tunnel near Black Rock reservoir. The maximum down surge, the tunnel diameter, and the surge shaft diameter are based on a Black Rock reservoir elevation of 1500 feet (top of inactive pool). The maximum pressure at the pumping plant and the elevation at the top of the surge shaft are based on the top of the active reservoir pool (elevation 1775 feet). The tunnel would have a surge shaft. The top of the surge shaft would be on a level spot for construction activities and would be set to prevent overtopping.

Tunnel excavation would likely be accomplished using tunnel boring machines and could be excavated uphill. All of the tunnels would be above the current water table, so groundwater should not be a major problem. The tunnel design would include initial support during construction to stabilize the tunnel opening until the final support or lining was in place.

Design standards indicate that an unlined tunnel would be possible based on the mineralogy of the rock. However, this evaluation uses a tunnel lining to ensure reasonable hydraulic friction and to account for rock that may be highly fractured. The design uses some reinforcement to curtail leakage in highly fractured reaches in the pressure tunnels and incorporates steel lining at the portals to insure water tightness as the tunnel nears the surface. This steel liner would be backfilled with concrete and grouting. The tunnel portal beneath Black Rock reservoir would not require steel lining.

5.5.1.2 Tunnel/Pipeline Inflow Conveyance

The tunnel/pipeline option initially was to be a pressurized system from the intake pumping plant to a Black Rock reservoir. However, after reviewing the hydraulics and the transient analysis and increasing the size of the tunnel beyond the surge shaft, a gravity tunnel would work better. The selected design would thus include a 16-foot-diameter discharge pipe, tunnel and vertical shaft; a 21-foot-diameter gravity tunnel; and an 18-foot-diameter pipeline. The increased tunnel size (downstream from the surge shaft) and the hydraulic grade line of the gravity tunnel would allow the 3,500 cfs to flow around Yakima Ridge to Black Rock reservoir in the pipeline. Once the pipeline reached the south side of Yakima Ridge, the pipe diameter would reduce to 17 feet. The manifold and initial tunnel diameter would maintain a flow velocity below 20 feet per second. The discharge pipe and tunnel would have an 18-foot-per-second flow velocity at 3,500 cfs. The tunnel design would be similar to that of the all tunnel option. The pipeline would connect to the low-level Black Rock reservoir outlet works pipe so both the outlet works and the pipeline could use the same tunnel through the dam's abutment.

5.5.2 6,000-cfs Pump Only Option

The 6,000-cfs pump only option would be similar to the 3,500-cfs all tunnel design (see section 5.5.1.1) and would use a 22-foot-diameter tunnel sloping steadily up towards a Black Rock reservoir. The tunnel would have a surge shaft. The tunnel diameter would be sized for 6,000 cfs plus a 5 percent allowance to account for the pumps' wear factor and manufacturer's allowance. The end of the tunnel would be located to prevent a negative down surge in the tunnel near Black Rock reservoir. The top of the surge shaft would be set to prevent overtopping and would be located to provide a level spot for construction activities.

5.5.3 3,500-cfs Pump/Generation Option

The pump/generation inflow conveyance option would be the same as the 3,500-cfs pump only, all tunnel inflow conveyance (see section 5.5.1.1) option, except for a multi-level intake/outlet structure at Black Rock reservoir. This intake would control the withdrawal elevation for water returning to the Columbia River to meet water quality objectives. The valve-controlled intakes would be fixed at four different reservoir elevations and allow any combination of withdrawal levels. The intakes would discharge into a wet well before entering a tunnel to the Priest Rapids intake pump/generation plant. During normal pumping operations, pumped water would discharge through two gates at the bottom of the wet well.

Half-cylinder-shaped fish screens installed at each intake level (with flat side panels attached to the intake tower) would be used only during power generation. Passing pumped (back flush) water through the screens for a short period during pumping operation would clean the screens.

5.6 Black Rock Storage Facilities

The storage facilities would impound water received via the inflow conveyance system. Storage facility options suitable for the Black Rock damsite would include both a large dam/large reservoir and a small dam/small reservoir option, as well as two rockfill embankment dam types and a roller compacted concrete dam. This section first describes the design considerations related to the large dams, then the small dams. It explains why a spillway would not be needed and identifies the design options for an emergency evacuation outlet works, the reservoir, and highway/utility relocations.

5.6.1 Storage Dam Alignment

This Assessment analyzes two potential dam alignments. The original, farthest east (downstream) alignment proposed and explored by WIS, would have the shortest crest length; however, exploratory drilling identified more than 200 feet of overburden material toward the south (right) abutment that would likely have to be excavated. Drill hole testing at an alternate alignment located further west (upstream), where possibly less excavation would be needed, revealed the overburden was at least as deep as that of the downstream alignment. The alternate alignment resulted in about 10 percent fewer cubic yards of above-ground embankment materials, even though the crest length would be longer and the dam higher to get an equivalent reservoir storage. This could be because the ground surface rises heading upstream. The longer axis of the alternate site would result in significantly more below-ground excavation than the original site. The alternate axis would have about 10 million cubic yards more of combined above and below ground embankment fill materials, which would significantly increase costs. The alternate alignment lacks technical advantage (such as improved rock quality and better outlet works location) over the original alignment.

Findings: The alternate dam alignment, with no technical advantages and, needing about 10 million more cubic yards of embankment material, is dropped from further analyses.

5.6.2 High Seismicity

The Black Rock damsite lies in an area of high earthquake potential. Because of its proximity to the site, the Black Rock Valley fault is the largest seismic hazard contributor to the initial estimates of high frequency ground motions. This dictates that, at this stage of the work, a dam at this location be designed to resist potential ground shaking that might be associated with the occurrence of magnitude 6 to 7+ earthquakes. Earthquakes of this magnitude could cause lower density, saturated embankment or foundation soils to liquefy. This loss of material strength could result in large slope failures. Therefore, all potentially liquefiable foundation soils should be removed and all embankment materials compacted to high densities.

Designing a dam for severe and lengthy earthquake shaking that could induce embankment slope failures involves careful deformation analysis of the dam; designing crest dimensions, zoning, and embankment slopes to ensure stability; selecting strong materials; and including a drainage system to keep the phreatic surface (water level) in the embankment as low as possible.

5.6.3 Potential Fault Displacements

At least one low-angle thrust fault lies within the dam's footprint, at the base of the right (south) abutment. Preliminary investigation of available information leads to the assumption that fault offsets within the dam footprint are possible, and that such displacements could range from a few centimeters to several meters. The east-west fold orientation of the Yakima Fold Belt indicates displacement would reflect compression of the folds in a north-south (cross valley) orientation. Severe transverse cracks through the dam would likely result from low-angle fault offset.

An embankment dam generally would be less rigid than a concrete dam and may best accommodate potential fault displacements. An embankment dam would have filters and drains to ensure that cracking, offsets, or differential movements does not provide pathways for the reservoir contents to escape and erode the embankment. The clean, cohesionless, and permeable sands and gravels that make up these filters and drains would collapse or rearrange if the dam cracks, causing these materials to fill in the crack. While the upstream water barrier (such as an earth core or concrete face) would likely crack and possibly stay open from a fault offset, the filter would ensure that no fine-grained core materials eroded downstream through the filter. The gravel drain downstream from the filter would provide safe collection of seepage that passed through an earth core or concrete face crack.

Large rockfill shells, constructed of 3-foot rocks, would form a stable downstream buttress for the earth core or concrete face of a dam where fault displacement would be possible. Extensive reservoir leakage would safely flow through the rockfill zone without causing dam failure.

5.6.4 Large Rockfill Embankment Dam

For all dam types considered in this Assessment for the large 1,300,000-acre-foot reservoir, the top of normal water surface would be set at elevation 1775 feet; the maximum reservoir water surface would be set at elevation 1778 feet.

Basalt for a rockfill dam is present throughout the damsite and reservoir area, with relatively little soil cover on the abutment and reservoir rims. Quarrying would provide an unlimited basalt source of rockfill, making a rockfill dam suited for the Black Rock site. Rockfill dams are one of the best performing dams under the seismic conditions believed to be present at the Black Rock damsite. These dams keep a large downstream portion of the embankment material unsaturated and strong, while providing permeability to let seepage pass through should the impervious zone of the dam become cracked or damaged. The two types of rockfill embankments best suited to the Black Rock site are a concrete face rockfill dam and a central core rockfill dam. Due to the high earthquake potential, all embankment zones would be compacted to maximum practicable densities to prevent liquefaction.

5.6.4.1 Foundation Treatment

The amount of foundation treatment required would depend on the quality of rock encountered. Two overburden excavation variations (the complete excavation to bedrock option and the excavate to competent Ringold Formation option) address the uncertainty of the amount of Ringold Formation to be removed. The complete excavation to bedrock option would expose bedrock along the entire footprint of both rockfill dams. This would reduce all uncertainties of foundation liquefaction potential, and allow for use of steeper rockfill slopes. The excavate to competent Ringold Formation option would remove all of the overburden beneath the upstream portion of the dam to ensure a foundation of competent bedrock and that foundation treatment and grouting were effective. For most of the downstream portion of the dam, removing the fine-grained lake deposits in the upper portions of the Ringold Formation would minimize foundation settlement and the potential for liquefaction.

Foundation excavation likely would encounter several different basalt flows, areas of thin veneer basalt and sedimentary interbeds, and poor rock quality at the contacts of these various flows; all a result of the geologic folding in Black Rock Valley. Localized areas of overexcavation would remove poor quality rock, thin basalt veneers, and the interbed zone.

Available information is insufficient to positively confirm the presence and nature of an apparent fault beneath the right abutment. The types of foundation treatment for an uncovered fault zone depend on the nature of the material exposed and range from additional excavation, to an enlarged blanket and curtain grouting, to placing impermeable soils upstream from the water barrier, to covering a pervious zone with a filter/drain, to a cutoff zone to the depth of the fault.

Special upstream and downstream foundation treatment and filters would be required in areas of particularly poor rock quality, which may include highly fractured rock or highly weathered or altered rock, or in areas of faulting. This would prevent any potential seepage movement of poor foundation materials into the coarse rockfill embankment materials.

5.6.4.2 Large Concrete Face Rockfill Dam

An advantage of a concrete face rockfill dam is that it would not contain a soil core vulnerable to erosion under a concentrated leak. The upstream reinforced concrete face would act as a water barrier and would not be susceptible to erosion. The concrete face would tie into the rock foundation with a concrete footing (plinth). Downstream from the concrete face, a zone (zone 2) of well-graded sand and gravel with fines would serve as a firm foundation for the concrete face slab. Should any seepage pass through the concrete face, this zone 2 would form a semi-pervious barrier that would significantly increase head loss and the amount of seepage. A pervious transition zone 3 (of clean gravel and cobble) immediately downstream from zone 2 would ensure that zone 2 could not erode from concentrated leaks and would provide sufficient drainage to allow seepage flows to pass into and through the large downstream rockfill zone (zone 4) of the dam. The rockfill would be constructed in 3-foot-thick lifts, and compacted with large vibratory rollers to create a layer with larger rock at the bottom and an accumulation of fines at the top. These stratified rockfill layers would have high horizontal permeability and safely drain off large seepage flows.

A disadvantage of a concrete face rockfill dam is that any type of fault offset likely would cause extensive cracking in the concrete face. Although the rockfill dam likely would not fail, the reservoir may have to be drained after such an event until the concrete face were repaired.

The smooth, concrete face rockfill dam would have 15 feet of freeboard to prevent wind/wave overtopping. A rockfill dam at this site would unlikely need extra freeboard to protect against major embankment deformations resulting from seismic events.

The amount of foundation treatment required for the concrete face rockfill dam would depend on the quality of rock encountered. Excavating to competent Ringold Formation would minimize the potential for settlement of overburden that may cause cracking of the concrete face slab. In all areas, a minimum amount of treatment would be a combination of blanket consolidation and curtain grouting. Extensive grouting would be likely in areas of fractured basalt and poor rock quality.

5.6.4.3 Large Central Core Rockfill Dam

A central core rockfill dam would be more plastic or deformable, and less cracking damage may result from a fault offset within the dam footprint. Furthermore, a central core containing appreciable clay likely would self-heal the cracks. Repairs, if needed after a fault offset, might not entail draining the reservoir. The massive downstream rockfill zone would need far less foundation treatment than would be required beneath an impervious zone.

The water barrier of a central core dam would consist of impermeable soils. An upstream-sloping, thin earth core of zone 1 materials (clayey gravel and lean clay or silty gravel) would ensure that the rockfill zone (zone 4) of the dam remained strong and unsaturated, thereby affording much stability. Reduced costs would result from keeping the core thin as the site lacks impermeable soils. The sloping core should reduce the potential for core cracking due to differing settlement properties of rockfill and impermeable material. Immediately downstream

from the earth core would be a zone 2 filter zone (consisting of clean sand and gravel) designed to prevent erosion of core materials in the event of cracking. Downstream from the zone 2 filter would be a clean gravel and cobble drainage zone (zone 3) to safely control and convey any seepage through core cracks. The majority of the central core dam would be rockfill as described for the concrete face rockfill dam.

The large central core rockfill dam, with a rougher rock upstream slope that would be more effective in dissipating waves, would have 10 feet of freeboard to prevent wind/wave overtopping.

Foundation treatment measures for the central core rockfill dam would be concentrated beneath the water barrier core of the dam and depend on the quality of rock encountered. Rock excavation and dental concrete would shape the bedrock surface to minimize abrupt surface changes and overhangs. Some slush grouting may be needed. A combination of blanket consolidation and curtain grouting would improve rock strength and create a low permeability zone beneath the core. Extensive grouting would be likely in some areas with a multiple row grout curtain likely over the entire footprint of the zone 1 core. Foundation treatment beneath the remainder of a rockfill dam would be less critical, except in areas of highly weathered rock or fault zones where seepage or displacement could occur.

5.6.5 Large Roller Compacted Concrete Dam

Roller compacted concrete (RCC) was selected for the concrete dam option since it would be more economical than mass concrete for wide canyons. RCC is a no-slump concrete placed by earth-moving equipment and compacted by vibrating rollers in horizontal lifts up to 12 inches thick. Bonding mortar would be placed at each lift line. RCC materials would be the same as those used for conventional mass concrete and include water, cementitious materials (cement and pozzolan), admixtures, and fine and coarse aggregate. Aggregate would be hauled from either the Columbia or Yakima River area due to the anticipated high cost of onsite processing of the Ringold Formation. The upstream and downstream faces of the dam would be slip-formed conventional concrete that serves as the forms for the RCC placement. Crack inducers and waterstops would be placed to form contraction joints. The dam would have drainage galleries and formed drains that would be drilled from the top of dam to the gallery near the upstream face to intercept any seepage through lift lines.

The RCC dam would have 6 feet of freeboard to the top of the parapet wall (elevation 1784) after storing the probable maximum flood.

The RCC dam requires a competent rock foundation beneath the entire footprint of the dam. Available geologic information indicates foundation excavation to the top of competent rock is 200 feet deep towards the right abutment. Ten feet of rock likely would be removed from the overall footprint to reach competent material.

5.6.6 Small Rockfill Embankment Dam

The large rockfill embankment dam design discussion (see section 5.6.4) also applies to the small rockfill embankment dam, which would still be a very large embankment. The only differences would be the crest elevation and dam height; all other design features would be the same. Table 5-7 compares the large and small embankment dams.

Table 5-7. Comparison of large and small rockfill embankment dams

Dam Type	Crest Elevation (feet)	Dam Height* (feet)
concrete face rockfill – large reservoir	1790	760
concrete face rockfill – small reservoir	1722	692
central core rockfill – large reservoir	1785	755
central core rockfill – small reservoir	1717	687
*The dam height is the maximum structural height from crest of the dam to the bottom of foundation excavation.		

5.6.7 Small Roller Compacted Concrete Dam

The large RCC dam design discussion (see section 5.6.5) also applies to the small RCC dam, which would also be a very large dam. The only differences would be the crest elevation, top elevation of the parapet wall, and dam height; all other design features would be the same. Table 5-8 compares the large and small RCC dams.

Table 5-8. Comparison of large and small RCC dams

Dam Type	Crest Elevation (feet)	Dam Height* (feet)
RCC dam – large reservoir	1781	751
RCC dam – small reservoir	1715	685
*The dam height is the maximum structural height from crest of the dam to the bottom of foundation excavation.		

5.6.8 Spillway

Both the large and small Black Rock reservoirs would be offstream and have large surface areas. Numerous concerns about a potential spillway located on the south side of Black Rock reservoir and concerns about discharging Columbia River water into the Yakima River led to investigating the possibility of storing the probable maximum flood in the reservoir rather than spilling the flood flows. Raising the large dam height by 3 feet and the small dam by 5 feet (to the heights shown on tables 5-7 and 5-8) eliminates the need for a spillway.

5.6.9 Low-Level Outlet Works

The low-level outlet works would be used to evacuate the reservoir contents into the normally dry Dry Creek in the event of a dam safety emergency. Trashracks would be provided at the outlet works intake; however, fish screens would not be provided since this outlet would be used infrequently, only for emergency evacuation. No separate detailed designs were prepared for the

outlet works for the small reservoir, but rather, quantities are estimated to be about 95 percent of those for the large reservoir outlet works.

The outlet works for the rockfill embankment dams would be on the north (left) abutment rather than the right abutment due to more favorable geology. Concrete thicknesses for the conduits are based on other Reclamation projects with similar sized facilities. The upstream conduit would be steel lined to handle the extreme pressures and potentially fractured rock. Steel pipe thicknesses would be sized to handle full reservoir pressures. The downstream steel pipe would be buried for support. A fixed-wheel emergency gate housed in a gate chamber at the bottom of a vertical shaft would reduce the length of pressure pipe through the dam. Two jet flow gates would control the outlet works discharge. A plunge pool stilling basin, which likely would see little use, would be lined with impervious material and riprap. Large concrete thrust blocks would be sized to handle anticipated thrust loads at the pipe bends.

The RCC dam outlet works intake would be constructed within the dam structure on the upstream face near the right abutment to shorten the outlet works. The intake structure would have a fixed-wheel emergency gate. The downstream steel pipe would be buried for support. Two jet flow gates would control the outlet works discharge. A plunge pool stilling basin, which likely would see little use, would be lined with impervious material and riprap. Large concrete thrust blocks would be sized to handle anticipated thrust loads at the pipe bends.

5.6.10 Reservoir

Reclamation's water availability assessment [4] identifies two reservoir sizes that meet water delivery criteria for a water exchange with Roza and Sunnyside Divisions. Aerial topographic data provides a basis for reservoir elevation versus reservoir volume and area curves. Holding the inactive storage to a minimum would reduce the dam height required for total storage. Also, designing the dam to store the probable maximum flood in the reservoir would eliminate the need for a spillway. Table 5-9 identifies the large and small reservoir parameters.

Table 5-9. Preliminary Black Rock reservoir parameters

Design Parameter	Large Reservoir	Small Reservoir
maximum water surface elevation	1778 feet	1712 feet
normal water surface elevation:	1775 feet	1707 feet
active capacity	1,300,000 acre-feet	800,000 acre-feet
surface area	13.5 square miles	10 square miles
top elevation of inactive water surface:	1500 feet	
inactive capacity	157,610 acre-feet	
surface area	3.25 square miles	

5.6.11 Highway and Utility Relocations

A Black Rock reservoir would inundate up to 13.5 square miles of Black Rock Valley including SH 24, a 2-lane asphalt road. The WIS report [2] relocates SH 24 to the south of the reservoir in the Rattlesnake Hills and indicates that Black Rock Valley residents prefer a northern relocation.

However, a northern realignment would run SH 24 through the Yakima Training Center. Also, the topography of a northern realignment is such that the road would include many bridges.

Reclamation identified an approximate 11.8-mile-long realignment similar to the WIS alignment; however, more detailed aerial topography provided a more refined horizontal alignment and vertical profile for SH 24. The new, straighter alignment would avoid some of the difficult terrain and improve the Rattlesnake Hills crossing. It would include larger horizontal curves to accommodate a 50-mph speed limit for mountainous terrain and a 70-mph speed limit for level terrain. Although the vertical profile would follow the existing terrain more closely, large cut and fill areas would still lie along the straighter alignment.

Local residents and city and county representatives have voiced concern with the southern alignment. If the Storage Study proceeds, Reclamation would pursue the SH 24 relocation with them, as well as with representatives of the Washington Department of Transportation and the Yakima Training Center.

Utilities impacted by a Black Rock reservoir would include a buried fiber optic line along the existing SH 24 and two overhead 115-kilovolt power lines on H-frame-type wood-pole supports. The buried fiber optic line would be abandoned in place. The transmission lines would be removed from the valley floor. Both utilities would run along the realigned SH 24.

5.7 Black Rock Reservoir Outflow

5.7.1 Conveyance System

A single-level intake at elevation 1500 feet was considered for release of reservoir water to the outflow conveyance system. A multi-level intake was not considered for this Assessment because no specific water quality objectives have been identified for the irrigation water, and there are no downstream fish water quality considerations. Fish screens would be included on the outlet structure to prevent fish that may be stocked in the reservoir from migrating into the Yakima River basin. Fish screen sizing criteria is assumed to be the same criteria used to size the intake structure at Priest Rapids Lake. The fish screened intake assumed for this Assessment would be a half-cylinder-shaped screen supported on a reinforced concrete slab. An air burst backwash system would clean the screens, and bulkhead gates and guides would be used dewatering the outflow conveyance system during emergencies.

Several outflow conveyance options were considered for delivering water from a Black Rock reservoir to Roza Canal. A 2,500-cfs design flow was selected for the outflow conveyance based on the assumption of providing the Roza and Sunnyside Divisions' entire water supply. This amounts to an instantaneous flow of about 2,362 cfs plus an allowance for other entities whose main canal facilities are in the vicinity of Roza Canal.

The primary features of the selected option are shown in figure 5-5. The 17-foot-diameter tunnel would begin southeast of Taylor Ranch on the northern side of Black Rock reservoir and parallel the southern edge of Yakima Ridge for approximately 14 miles to the 40-foot-diameter surge

shaft. Beyond the surge shaft, the tunnel would angle out of the mountains and end on the northern side of SH 24. Then a 17-foot-diameter buried steel pipeline would cross under SH 24 and run to a Black Rock powerplant. The distance from the surge shaft to the powerplant would be approximately 3.5 miles.

The top elevation of inactive storage in Black Rock reservoir is the basis for the maximum down surge and sizing of the tunnel and pipeline diameters and the surge shaft. The top elevation of active storage in Black Rock reservoir is the basis for the maximum pressure at the powerplant and the top elevation of the surge shaft. The required minimum water surface, or hydraulic grade line, at a Black Rock powerplant would enable water to be delivered to Sunnyside powerplant without another pumping plant.

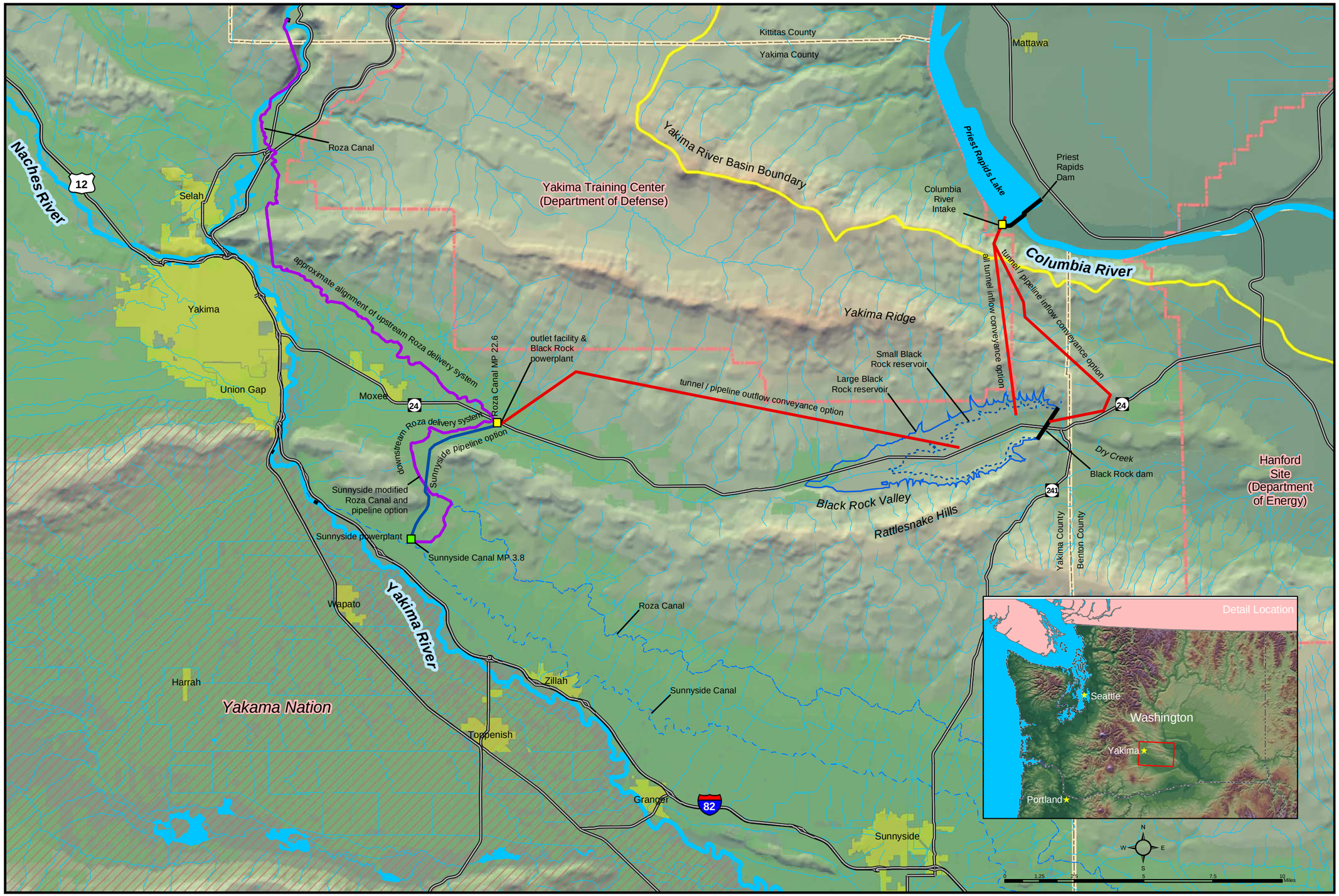


Figure 5-5. Preliminary Black Rock reservoir outflow conveyance system

5.7.2 Outlet Facility

The potential Black Rock outlet facility would be on the southeast corner of Roza Canal (at MP 22.6) and SH 24 intersection. The facility would include a Black Rock powerplant, a bypass structure to permit water deliveries when the unit was off line or to pass flows in excess of powerplant design flows, a flowmeter, and manifold piping and valving for pressure pipe diversions to Roza and Sunnyside Divisions. This location was selected for its position within the Roza and Sunnyside delivery systems, its proximity to a Black Rock reservoir, and its ease of access from SH 24.

The type of system selected to deliver water from the outflow conveyance system to the potential water exchange participants would determine the design capacity of the Black Rock powerplant. (Section 5.8 discusses water delivery systems.) The following preliminary Black Rock powerplant design flow options represent possible turbine capacities.

- With the canal delivery option (figure 5-6), all exchange water would be discharged from the outflow conveyance system through either a Black Rock powerplant and/or a bypass structure into a modified Roza Canal. The Black Rock powerplant turbine design flow would be 1,500 cfs, which represents the Roza and Sunnyside Divisions' combined April water rights.

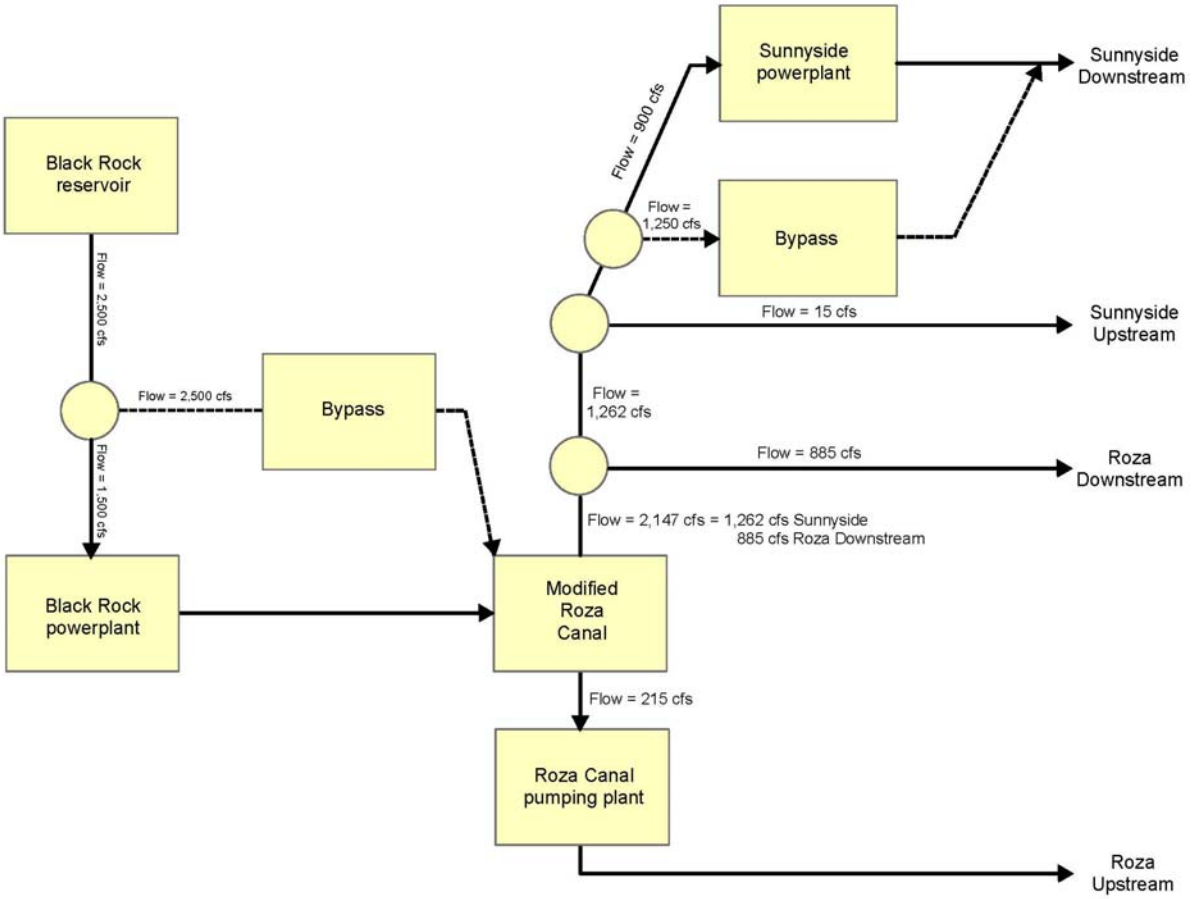


Figure 5-6. Preliminary Black Rock outlet facility flow diagram – canal delivery option

- With the pipeline delivery option (figure 5-7), only the exchange water required by Roza Division downstream from MP 22.6 (885 cfs) would be discharged from the outflow conveyance system through a Black Rock powerplant and bypass structure into Roza Canal. A 900-cfs turbine design, which represents anticipated deliveries to Roza Division downstream from MP 22.6, was selected for Black Rock powerplant under this type of delivery system. The exchange water required upstream from MP 22.6, and that for Sunnyside Division, would be diverted into pressurized pipeline delivery systems at bifurcation works upstream from the discharge to Black Rock powerplant.

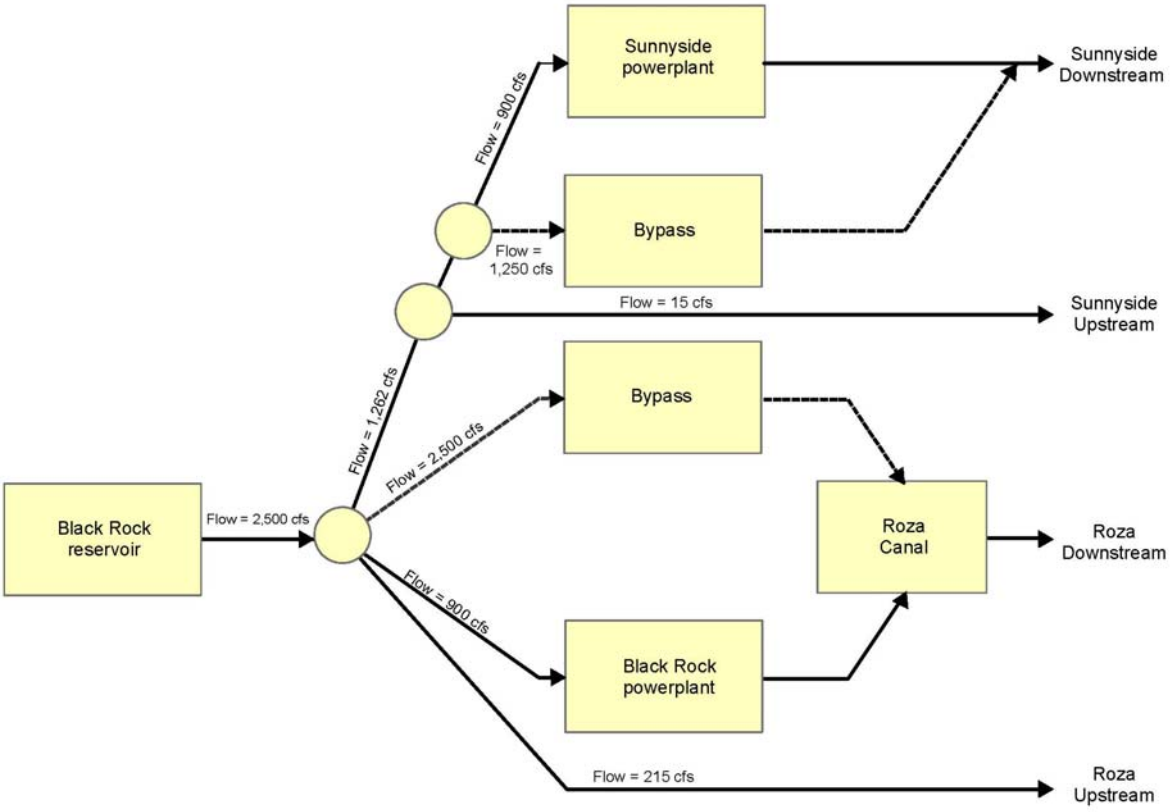


Figure 5-7. Preliminary Black Rock outlet facility flow diagram – pipeline delivery option

Although the outflow conveyance system's peak design flow would be 2,500 cfs, lesser turbine design flows were used so the powerplants could operate at full capacity for most of the April through October irrigation season and to reduce equipment costs. The bypass structure for both Black Rock powerplant options was sized to pass the total 2,500-cfs outflow conveyance system design flow.

The need for a temporary canal bypass was assumed during construction. Upstream and downstream earthen cofferdams with geomembrane linings would connect the transition structures to the canals. Three 9-foot-diameter corrugated metal pipes between the cofferdams would permit canal operation during construction.

The powerplant and bypass facilities would be contained in the same superstructure. The bypass would include four 84-inch sleeve valves to dissipate head. Each sleeve valve would discharge into a 33-foot-diameter by 20-foot-high stainless-steel-lined stilling chamber. A concrete-lined, open-channel outlet transition structure was sized to convey the outflow into Roza Canal.

Details and quantities for the 1,500-cfs powerplant and 2,500-cfs bypass structure were developed for this Assessment. No detailed layout or transient study of the 900-cfs powerplant were completed. For the 900-cfs powerplant option, the bypass was conservatively sized for the full outflow capacity to provide a means of bypassing the powerplant and pressurized water deliveries. This sizing should be reviewed and revised as necessary if the Black Rock alternative proceeds to feasibility design.

5.7.3 Outlet Powerplants

5.7.3.1 Black Rock Powerplant and Switchyard

A Black Rock powerplant would be an indoor-type plant with a structural steel superstructure enclosed with concrete masonry walls. The intermediate and substructure would be reinforced concrete. The powerplant would consist of a service bay and a single unit bay. The powerplant would use a single Francis turbine designed for longer operation at runaway speed and a reduced wicket gate closing rate. This turbine would be normal in design and available from most turbine manufacturers. One 90-ton overhead traveling crane would handle the powerplant and bypass electrical and mechanical items.

Black Rock reservoir operating water surface elevations would range from a low of 1500 feet to a high of 1775 feet. The water surface elevation in Roza Canal at MP 22.6 was assumed to be approximate 1170 feet. The steady state head at the Black Rock outlet facility (measured from canal water surface) would range from a low of 198 feet to a high of 477 feet. The powerplant design head for turbine sizing was assumed to be the average of the steady state head. Based on the assumed design criteria, table 5-10 shows a comparison of the Black Rock powerplant unit data for the two options.

Table 5-10. Preliminary Black Rock powerplant unit data

Unit Data	Canal Delivery 1,500-cfs powerplant	Pipeline Delivery 900-cfs powerplant
number/type of units	1 Francis turbine	1 Francis turbine
design discharge	1,500 cfs	900 cfs
design head	338 feet	338 feet
speed	327 rpm	400 rpm
assumed unit efficiency	90 percent	90 percent
power output at design head	38 MW	23 MW
minimum turbine submergence	30.5 feet	25 feet
maximum scroll case dimension	23.5 feet	19.0 feet
bottom elevation of draft tube	1118.8 feet	1128.3 feet
turbine guard valve	108-inch spherical	84-inch spherical

A switchyard would be located within a service yard sized to permit mobile crane access around the plant. A chain-link security fence would surround the service yard. The switchyard would include a transformer, circuit breakers, and disconnect switches, but no remote-control equipment.

5.7.3.2 Sunnyside Powerplant and Related Facilities

The potential Sunnyside powerplant, bypass, and switchyard would be located adjacent to Sunnyside Canal near its intersection with Konnowock Pass Road. Section 5.8.2 describes the two options (pipeline delivery and canal delivery) for conveyance of Black Rock reservoir water to Sunnyside powerplant and the bypass facilities.

The Sunnyside Canal outlet facilities would be similar in arrangement to the Roza Canal outlet facilities described in section 5.7.2. However, the Sunnyside bypass structure would be a separate indoor structure with a reinforced concrete substructure and a structural steel superstructure enclosed with concrete masonry walls. The bypass structure would house two 84-inch sleeve valves to dissipate head. Each sleeve valve would discharge into a 33-foot-diameter by 20-foot-high stainless-steel-lined stilling chamber. The bypass structure would discharge into a 12-foot-diameter steel pipe that would discharge into the riprap-lined outlet transition channel that would carry powerplant and bypass flows to Sunnyside Canal.

The Sunnyside powerplant would be an indoor-type plant with a structural steel superstructure enclosed with concrete masonry walls. The intermediate and substructure would be reinforced concrete. The powerplant would consist of a service bay and a single unit bay. A 125-ton overhead traveling crane would be provided to handle the powerplant electrical and mechanical items. Table 5-11 shows powerplant design considerations for the two delivery system options.

Table 5-11. Preliminary Sunnyside powerplant unit data

Unit Data	Pipeline Delivery Option	Canal Delivery Option
number/type of units	1 Francis turbine	1 Francis turbine
design discharge	900 cfs	900 cfs
design head	435 feet	221 feet
speed	400 rpm	300 rpm
assumed unit efficiency	90 percent	90 percent
power output at design head	29.5 MW	15 MW
minimum distributor submergence (negative if distributor is above tailwater)	+ 10.6 feet	- 1.1 feet
maximum scroll case dimension	19.4 feet	20.5 feet
bottom elevation of draft tube	859.2feet	854.5 feet
turbine guard valve	78-inch spherical	84-inch spherical

The service yard was sized to permit mobile crane access around the structures. A 7-foot chain-link fence was provided around the yard for security. Incoming power was assumed to be from a tap on an existing BPA line about 1 mile southwest of the switchyard. The line tap would include circuit breakers and disconnect switches. A new 69-kilovolt wood-pole line would be constructed from the tap to the switchyard. A 75-foot by 100-foot switchyard would be located within the service yard. The switchyard would include transformers, circuit breakers, and disconnect switches.

Findings: Based upon currently available information and the appraisal-level designs prepared for this Assessment, it is reasonably certain the construction of facilities to pump, store, and deliver Columbia

River water to willing exchange participants in the Yakima River basin would be technically viable.

5.8 Appraisal-Level Water Delivery System Plans

The appraisal-level water delivery plans developed to date could result in a Columbia River water exchange of up to 2,472 cfs. The extent of the exchange would depend on the capacity and configuration of delivery systems constructed to convey Columbia River water from the Black Rock outlet facility to exchange participants and upon the completion of exchange arrangements acceptable to the parties. At this time, potential exchange participants include Roza and Sunnyside Divisions, and Terrace Heights, Selah-Moxee, and Union Gap Irrigation Districts. Table 5-12 summarizes the irrigation requirements of potential exchange participants upstream and downstream from Roza Canal MP 22.6 and identifies possible water supply sources for each plan.

Table 5-12. Preliminary irrigation requirements based on six appraisal-level water delivery plans

Current Yakima River supply = 2,532 cfs						
Upstream From Roza Canal MP 22.6						
	(cfs)					
	Plan 1	Plan 2	Plan 3	Plan 4	Plan 5	Plan 6
Irrigation requirements	385	385	385	385	385	385
Potential Columbia River supply						
Roza and Terrace Heights	215	175	175	175	175	35
Selah-Moxee	—	—	—	80	80	—
Union Gap	—	—	—	70	70	—
Total Columbia River supply	215	175	175	325	325	35
Continued Yakima River supply						
Roza and Terrace Heights	—	40	40	40	40	180
Selah-Moxee	100	100	100	20	20	100
Union Gap	70	70	70	—	—	70
Total Yakima River supply	170	210	210	60	60	350
Total upstream from MP 22.6	385	385	385	385	385	385
Downstream From Roza Canal MP 22.6						
	Plan 1	Plan 2	Plan 3	Plan 4	Plan 5	Plan 6
Irrigation requirements	2,147	2,147	2,147	2,147	2,147	2,147
Potential Columbia River supply						
Roza	885	885	885	885	885	855
Sunnyside Division	1,262	1,262	1,262	1,262	1,262	1,262
Total Columbia River supply	2,147	2,147	2,147	2,147	2,147	2,117
Continued Yakima River supply						
Roza	—	—	—	—	—	30
Sunnyside Division	—	—	—	—	—	—
Total Yakima River supply	—	—	—	—	—	30
Total downstream from MP. 22.6	2,147	2,147	2,147	2,147	2,147	2,147
Potential Water Supply Sources						
Columbia River	2,362	2,322	2,322	2,472	2,472	2,152
Yakima River	170	210	210	60	60	380
Total	2,532	2,532	2,532	2,532	2,532	2,532

5.8.1 Plans For Delivery Upstream From MP 22.6

As described in the Roza delivery system assessment [7], six upstream plans provide various combinations of delivery of Columbia River water to Roza Division and Terrace Heights, Selah-Moxee, and Union Gap Irrigation Districts. Plans 1, 2, 3, and 4 would involve construction of a mainline delivery system extending from the Black Rock outlet facility. Plan 5 would involve installation of checks and pumps in Roza Canal to reverse the flow. Plan 6 would require no new construction.

The four appraisal-level plans (1, 2, 3, and 4) for mainline delivery systems would contain the following common features:

- A buried steel pipeline would originate at the Black Rock outlet facility bifurcation works. The static pressure resulting from the difference in head between stored water elevations (in a Black Rock reservoir) and mainline delivery system elevations would provide for upstream conveyance. A pressure-reducing valve system installed between the bifurcation works and the pipeline would dissipate any excess pressure to ensure the reliability and safety of the mainline delivery system.
- A buried steel pipeline would originate at a new pumping plant in Roza Canal at MP 22.6. Water discharged from the outlet facility through a new Black Rock powerplant would be pumped into the steel pipeline for upstream conveyance. Pressure would dissipate by discharging water through the generator.

Existing pumping plants of Roza and Terrace Heights Irrigation Districts currently lifting water from Roza Canal to high-elevation laterals upslope of Roza Canal would continue to be used in conjunction with the new mainline delivery systems in the following manner:

- With a high-pressure pipeline system originating at the bifurcation works, a dual operation would be required contingent on the Black Rock reservoir water surface elevation. When the reservoir was at elevation 1650.0 feet and higher, water released from the mainline delivery system would be routed through a new pressure-reducing valve system to the existing pumping plant discharge manifold extending to the high-elevation lateral. When the reservoir water surface was lower than elevation 1650.0 feet, water would be routed through the new pressure-reducing valve system to the existing pump sump for subsequent pumping to the high-elevation lateral. With a low-pressure pipeline, releases would go into the existing pump sump and then would be pumped to the high-elevation lateral.
- For a high-pressure system beginning at a new Roza Canal pumping plant, water released from the mainline delivery system for service to upslope lands would always be routed through the existing pumping plant discharge manifold to the high-elevation lateral. A low-pressure system starting at the same point would make releases to the existing pump sump for pumping at the existing plant to the high-elevation lateral.

Table 5-13 summarizes the above with respect to mainline water deliveries to existing Roza Canal pumping plants serving lands upslope of the canal. For purposes of discussing the individual plans, the two locations of the mainline inlet (at the bifurcation and at a new pumping

plant) are identified in each plan, i.e., for plan 1, a mainline inlet at the bifurcation is option 1 and a mainline inlet at a new pumping plant is option 1A.

Table 5-13. Water delivery by mainline delivery system to lands upslope of Roza Canal

Pipeline Inlet	Type of System	Releases Routed To:	Is Pumping Required At Existing Plants?
new bifurcation	high pressure	existing pump manifold discharge to high-elevation lateral - - - - - - - - - - - - - - - - - - - existing pump sump to high-elevation lateral	No – if reservoir water surface were at elevation 1650.0 feet or higher Yes – if reservoir water surface were lower than elevation 1650.0 feet
	low pressure	existing pump sump to high-elevation lateral	Yes – regardless of reservoir water surface elevation
new pumping plant	high pressure	existing pump manifold discharge to high-elevation lateral	No
	low pressure	existing pump sump to high-elevation lateral	Yes – regardless of reservoir water surface elevation

Lands downslope from the mainline delivery system would be served by turnouts to the existing lateral systems.

A brief description of the six appraisal-level water delivery plans follows.

5.8.1.1 Upstream Plan 1 – 215-cfs Exchange Using High-Pressure Pipeline

Upstream plan 1 (figure 5-8) would be a total exchange for Roza and Terrace Heights, whose combined April through October irrigation requirements (215 cfs) between Roza Diversion Dam and MP 22.6 would be met with Columbia River water. Up to 1,020 cfs currently diverted for hydroelectric generation at Roza Powerplant would be terminated.[3] Roza Canal would be dewatered from Roza Diversion Dam to MP 22.6. Plan 1 would not deliver Columbia River water to Selah-Moxee or Union Gap Irrigation Districts.

Plan 1 would involve construction of a new high-pressure mainline delivery system extending to about MP 5.0. This would provide up to 40 cfs to the Roza-Selah lands (those lands upstream from the inlet of Roza Canal tunnel No. 3 served by Roza Irrigation District). Up to 175 cfs would be provided to Roza and Terrace Heights lands downstream from the tunnel outlet (MP 11.0 to 22.6). Option 1A would provide the same delivery from a new pumping plant on Roza Canal.

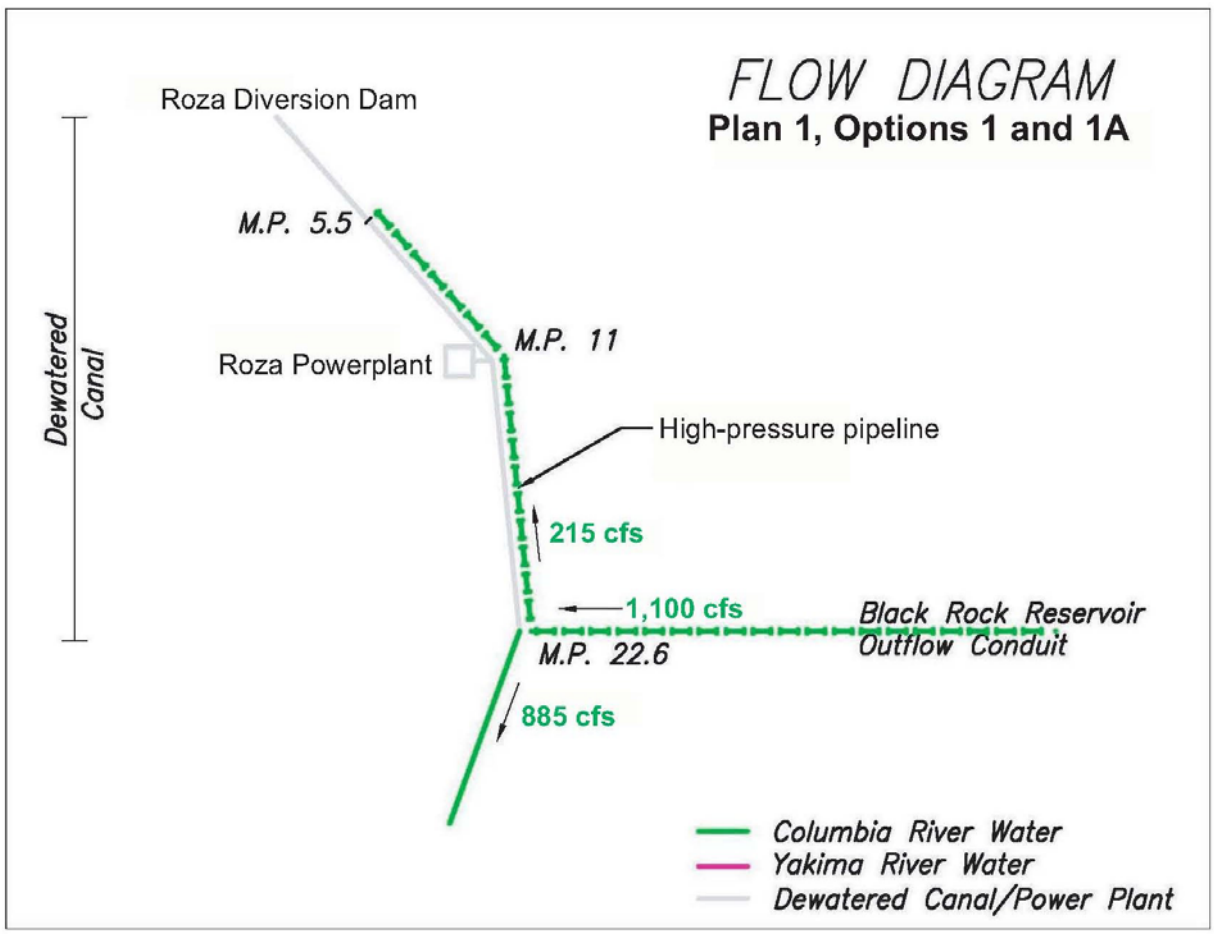


Figure 5-8. Flow diagram of upstream delivery plan 1

[3] The existing Roza Canal bifurcation works to the Roza Powerplant is at MP 11.0.

5.8.1.2 Upstream Plan 2 – 175-cfs Exchange Using High-Pressure Pipeline

Upstream plan 2 (figure 5-9) also would involve only Roza and Terrace Heights Irrigation Districts; however, the extent of the water exchange would decrease to 175 cfs by eliminating delivery of Columbia River water to the Roza-Selah lands. These lands would be served by continuing to divert 40 cfs from the Yakima River at Roza Diversion Dam (RM 127.9). This plan assumes Roza Powerplant near MP 11.0 would continue to operate (requiring the diversion of up to 1,020 cfs from the Yakima River), but that Roza Canal would be dewatered from MP 11.0 to MP 22.6. The new mainline high-pressure pipe system of plan 2 would extend upstream to about MP 11.7.

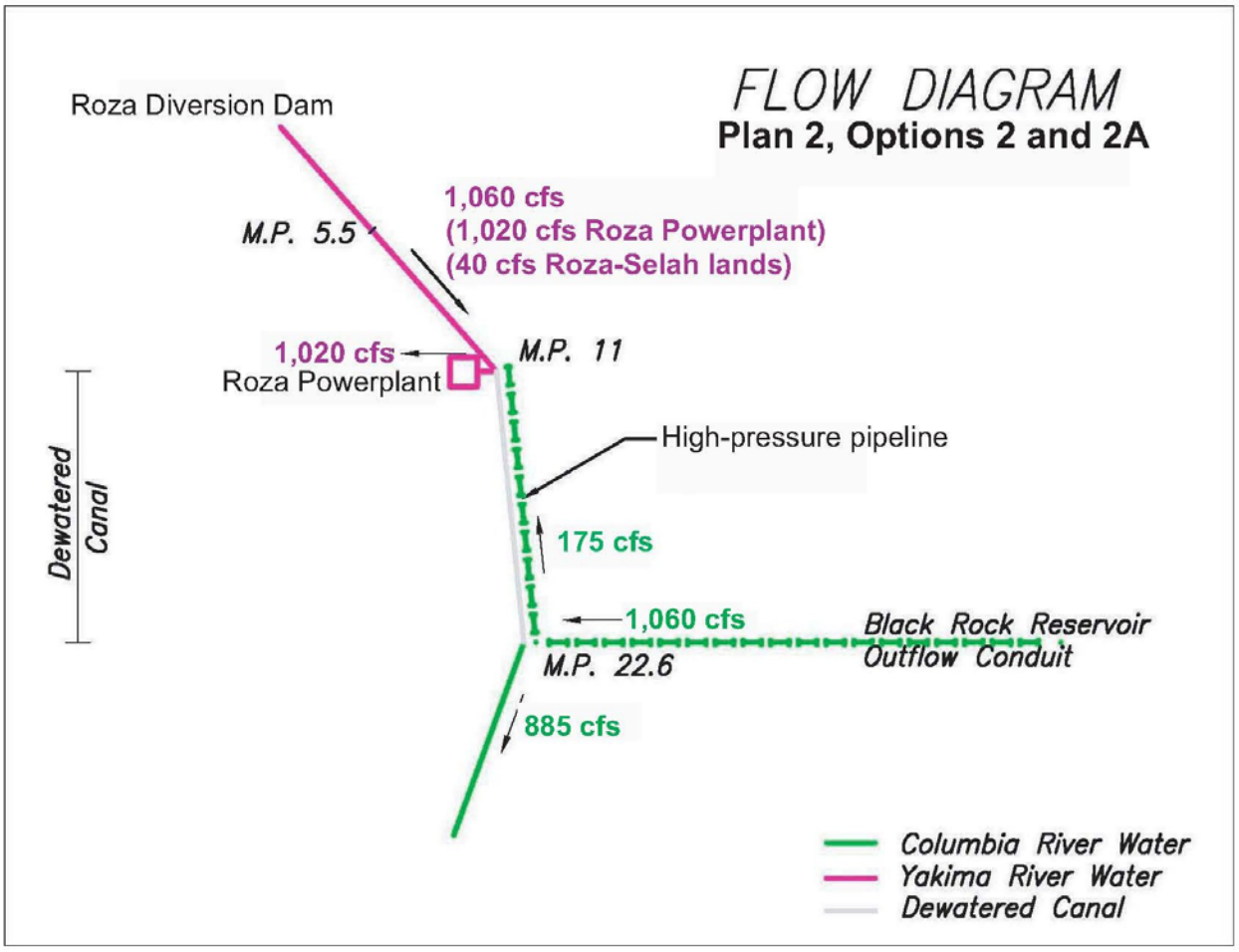


Figure 5-9. Flow diagram of upstream delivery plan 2

5.8.1.3 Upstream Plan 3 – 175-cfs Exchange Using Low-Pressure Pipeline

Upstream plan 3 (figure 5-10) would deliver 175 cfs of Columbia River water in the same manner as plan 2 – 175 cfs for Roza and Terrace Heights. The new mainline delivery system would be low pressure and require continued use of the existing pumping plants to lift water to the high-elevation laterals upslope of Roza Canal. This plan assumes Roza Powerplant would continue to operate (requiring the diversion of up to 1,020 cfs from the Yakima River), but that Roza Canal would be dewatered from MP 11.0 to MP 22.6.

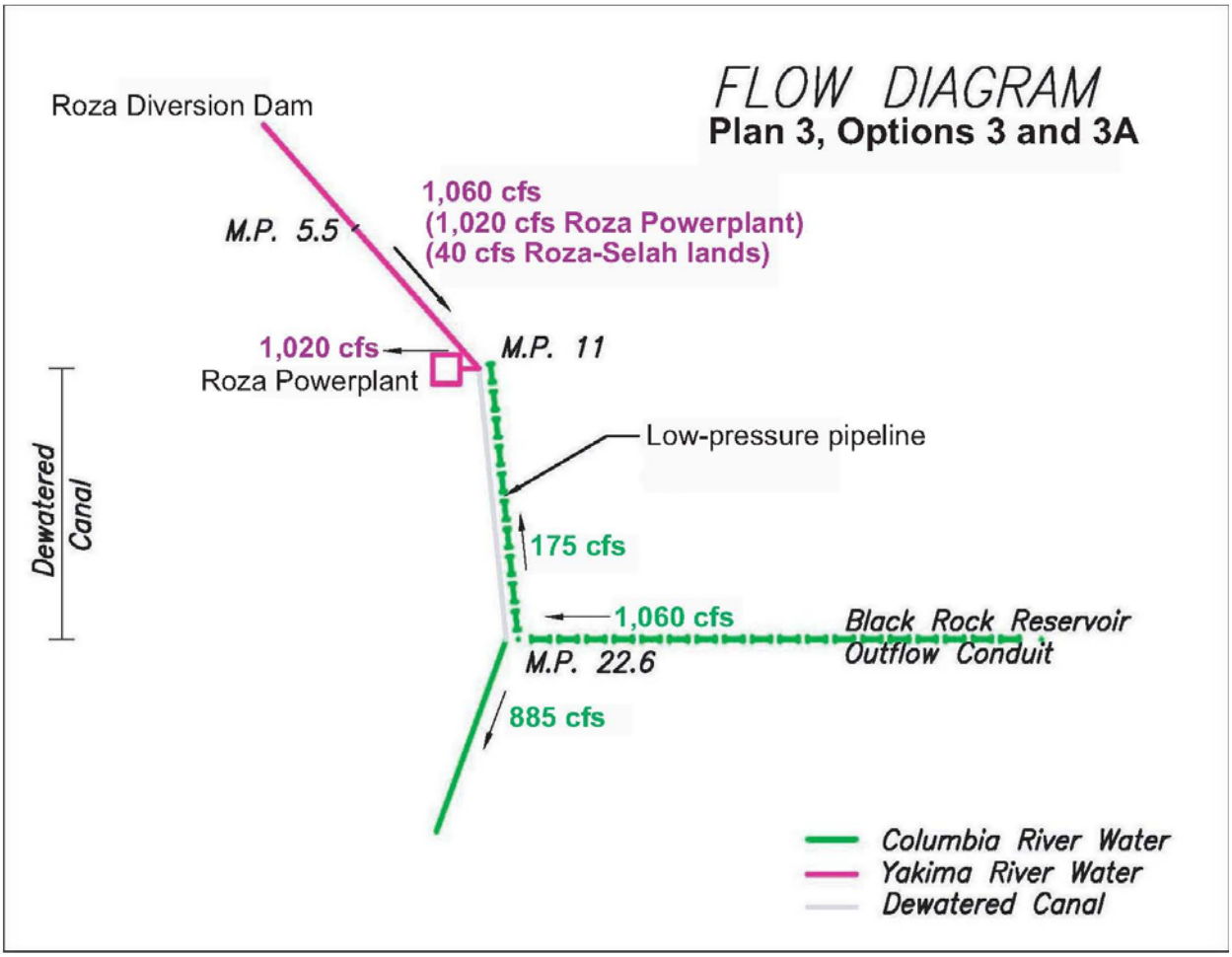


Figure 5-10. Flow diagram of upstream delivery plan 3

5.8.1.4 Upstream Plan 4 – 325-cfs Exchange Considering Three Pipeline Options

Upstream plan 4 (figure 5-11) would provide a total of 325 cfs of exchange water and include all potential water exchange participants upstream from MP 22.6 (Roza Division and Terrace Heights, Selah-Moxee, and Union Gap Irrigation Districts). This plan would meet all of the Roza and Terrace Heights irrigation requirements (175 cfs), except for the Roza-Selah lands, and all of the 70-cfs Union Gap irrigation requirements. Selah-Moxee would get 80 cfs of its 100-cfs requirement. Yakima River diversions of 60 cfs (40 cfs for the Roza-Selah lands and 20 cfs for Selah-Moxee lands) would continue. Plan 4 assumes Roza Powerplant would continue to operate (requiring the diversion of up to 1,020 cfs from the Yakima River) and that Roza Canal would be dewatered from MP 11.0 to MP 22.6.

This plan considers three mainline delivery systems:

- Option 4 would be a low-pressure pipeline extending from the bifurcation works.
- Option 4A would be a low-pressure pipeline extending from a new Roza Canal pumping plant.
- Option 4B would be a high-pressure, full-head-class pipe system beginning at the bifurcation works. This option would rely on the wall thickness of the pipe instead of a pressure-reducing valve system to handle the system pressure; thereby removing concerns that a pressure-reducing valve system (in option 4) may not consistently operate to ensure system pressure attributed to the head differential would not bypass the pressure-reducing valve system.

The new mainline delivery system would extend upstream to about M.P. 11.7. Water deliveries to the Selah-Moxee and Union Gap Canals would occur near this point.

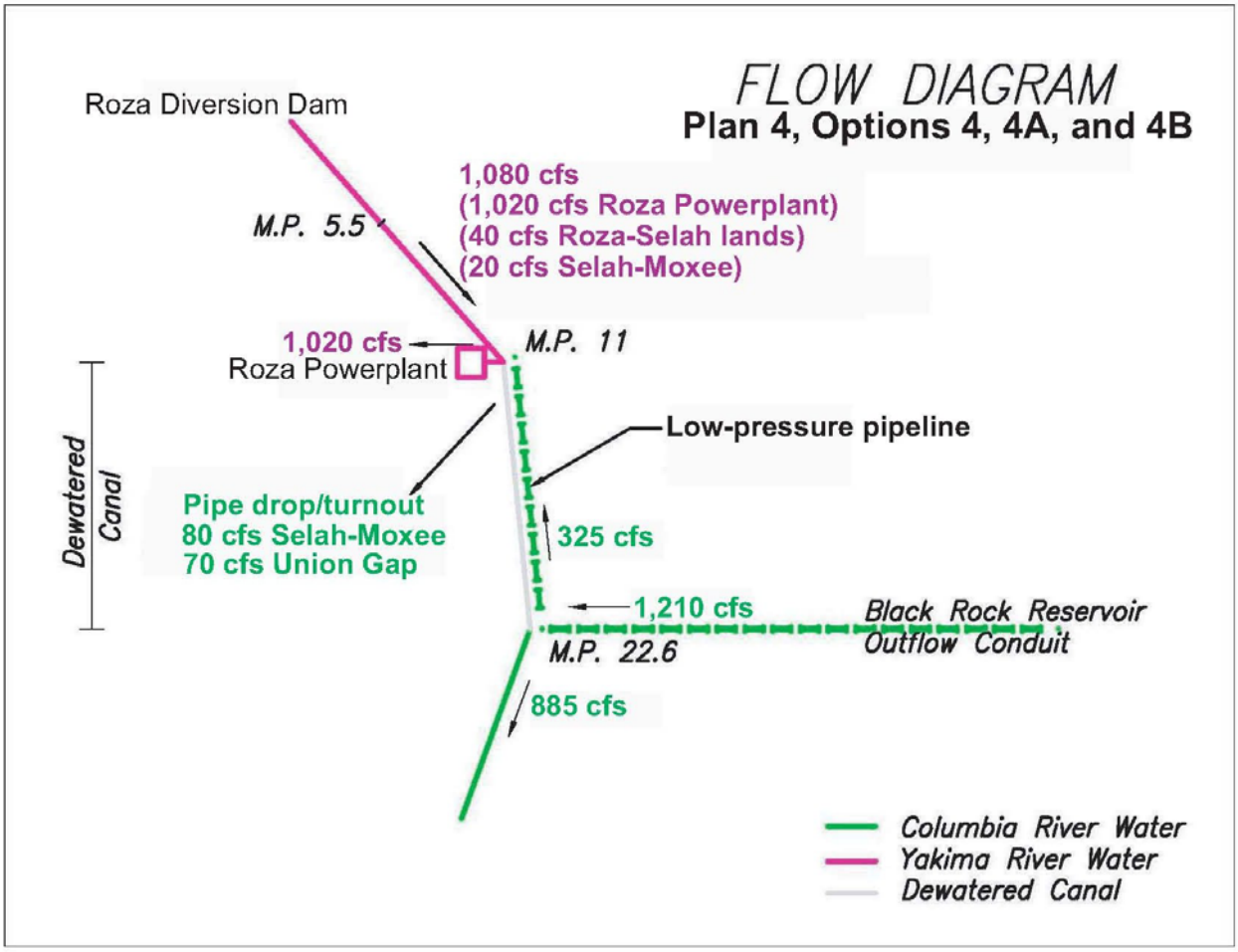


Figure 5-11. Flow diagram of upstream delivery plan 4

5.8.1.5 Upstream Plan 5 – 325-cfs Exchange Delivery With Checks and Relift Pumps

Upstream plan 5 (figure 5-12) would be similar to plan 4 and also deliver 325 cfs to the four upstream irrigation districts. However, this would be accomplished by installing checks and relift pumps to reverse the flow in Roza Canal from MP 22.6 to 11.7. Four new check structures and relift pumps would lift, in 5-foot increments, water discharged from the Black Rock outlet facility through a Black Rock powerplant. A terminal check would be also added at MP 11.7.

This plan assumes Roza Powerplant would continue to operate. Roza Canal from MP 11.7 to MP 22.6 would be watered up by the delivery of Columbia River water as the result of the reverse flow operation.

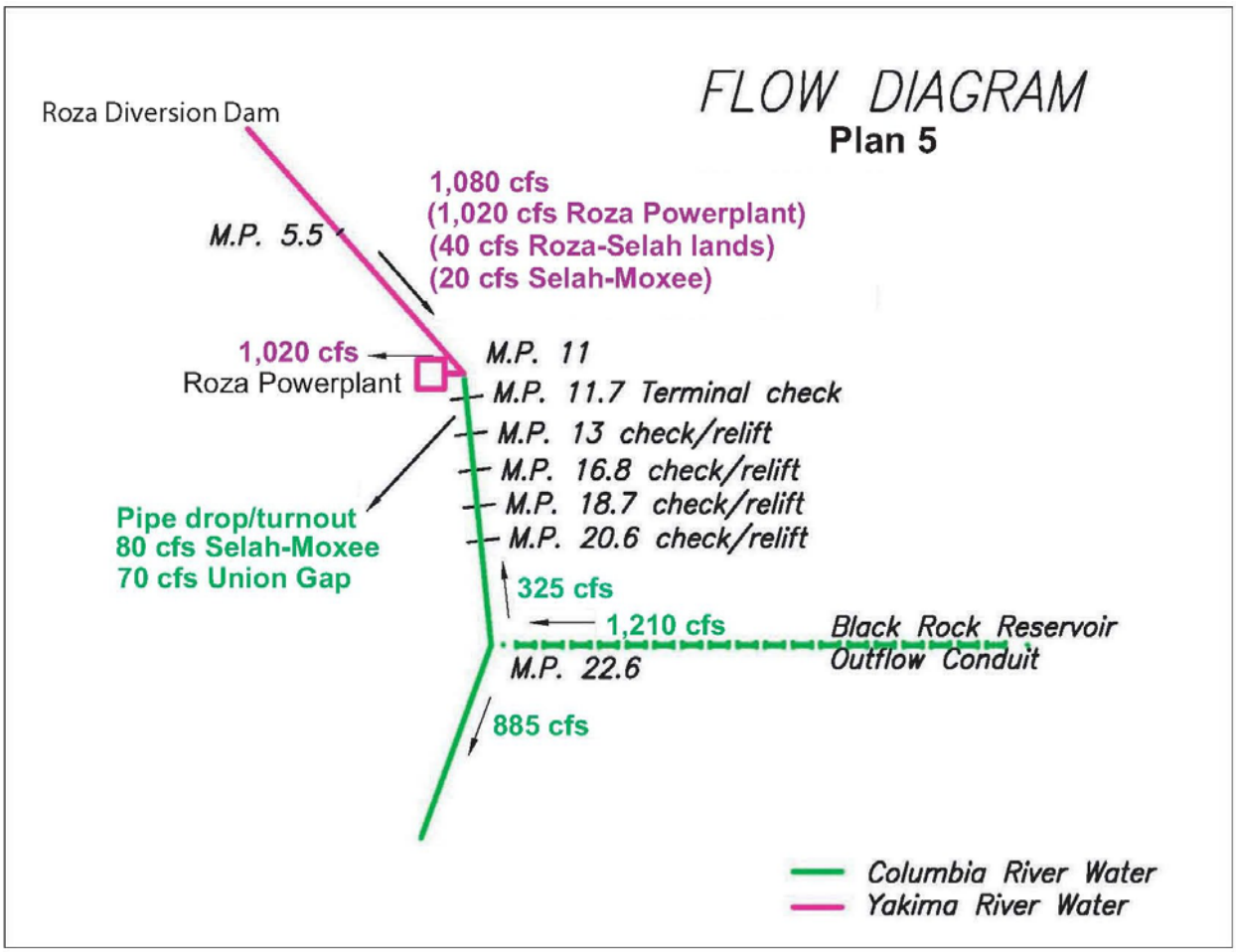


Figure 5-12. Flow diagram of upstream delivery plan 5

5.8.1.6 Upstream Plan 6 – 35-cfs Exchange

Under upstream plan 6 (figure 5-13), the only upstream water exchange would be with Roza Division. It would involve the delivery of 35 cfs of Columbia River water to meet a portion of the irrigation requirement at pumping plant No. 3 (65 cfs) at MP 22.5. Table 5-14 shows that Yakima River diversions would continue at 180 cfs.

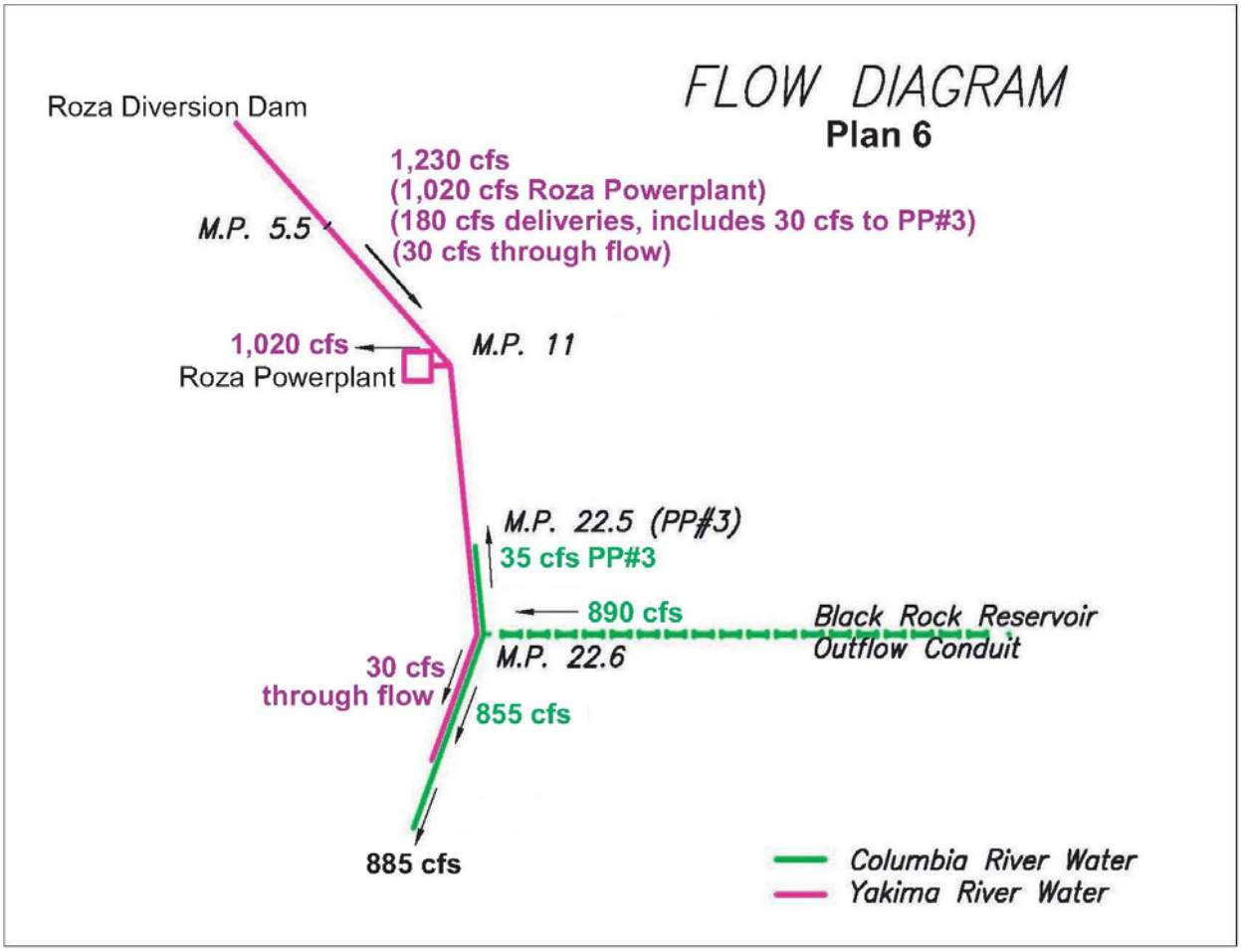


Figure 5-13. Flow diagram of upstream delivery plan 6

Table 5-14. Upstream delivery plan 6 Yakima River diversion requirements

Upstream from MP 11.0 (Roza-Selah lands)		40 cfs
MP 11.0 to MP 22.6 (total requirement)	175 cfs	
<less> pumping plant No. 3 exchange	–35 cfs	
Residual		140 cfs
Yakima River irrigation diversions		180 cfs

The 35 cfs from the Columbia River would be provided at pumping plant No. 3 from backflow of water discharged from the Black Rock outlet facility through a Black Rock powerplant into Roza Canal at MP 22.6. The Yakima River would provide the remaining 30 cfs required at

pumping plant No. 3. In addition to the 180-cfs Yakima River diversion to meet upstream irrigation requirements, 30 cfs would also be diverted as flow-through water to keep the canal from getting stagnant. This flow-through water would then be used for irrigation downstream from MP 22.6.

5.8.2 Plans for Delivery Downstream From MP 22.6 – Roza and Sunnyside Divisions

As described in the Roza [7] and Sunnyside [8] delivery system assessments, two plans for delivery downstream from Roza Canal MP 22.6 would involve Roza Irrigation District and Sunnyside Division. Service to Roza Division would be by means of the existing Roza Canal. Service to Sunnyside Division would be either by a new pressure-pipe delivery system extending through Konnowock Pass to Sunnyside Canal or by enlarging Roza Canal combined with a shorter pipeline to Sunnyside Canal. Either system would have a new Sunnyside powerplant at the discharge to Sunnyside Canal.

Maximum Roza Division irrigation requirements downstream from MP 22.6 are 885 cfs. This requirement could be met entirely by Columbia River water provided from the Black Rock outlet facility without incurring additional costs for construction of water delivery facilities. This would be done by releasing water from the Black Rock outlet facility through a new Black Rock powerplant into Roza Canal. This exchange water then would be conveyed to existing pumping plants and turnouts for the irrigation of upslope and downslope lands.

The preliminary design discharge and generation capacities of the Black Rock powerplant would be 900 cfs (23 MW) to 1,500 cfs (38 MW) depending on the Sunnyside Division water delivery plan discussed below. (Section 5.7.3.1 describes the Black Rock powerplant options.) The following describes the two appraisal-level water delivery plans for Sunnyside Division.

5.8.2.1 Downstream Plan 1 – Pipeline From Black Rock Outlet Facility

Downstream plan 1 would involve a buried steel pipeline extending from the Black Rock outlet facility bifurcation works generally following Roza Canal across orchard lands to the top of Konnowock Pass and then downhill parallel to Konnowock Pass Road. The pipeline, with a total length of about 6.5 miles, would discharge into Sunnyside Canal at MP 3.83. A check structure constructed in Sunnyside Canal at MP 3.83 would prevent water from backing up in the canal. At Sunnyside Canal, a powerplant and a bypass structure would dissipate the excess pressure. A 12-foot-diameter steel pipeline would keep velocities under 12 feet per second. The pipeline would be sized using a 1,262-cfs flow capacity; the design of the turbine would be based on a 900-cfs flow.[4] The output of the powerplant (at the design head of 435 feet) would be 29.5 MW.

A 210-foot static hydraulic head at the beginning of the pipeline would deliver water to Sunnyside Canal. This would result in a minimum 438-foot hydraulic head at the new Sunnyside

[4] This capacity is representative of the Sunnyside Division's April water rights. The 900-cfs design flow permits the powerplant to operate at full capacity for most of the irrigation season and reduces equipment costs.

powerplant and a maximum 743-foot hydraulic head with Black Rock reservoir at maximum water surface elevation 1778 feet. The pipeline would be designed to the maximum hydraulic head and, in this respect, would be similar to upstream plan 4, option 4B, which would rely on the pipe wall thickness to handle the system pressure. This is a conservative approach at this stage of the Storage Study, and further analysis may result in other ways to handle this significant pressure.

Most of the discharge from the powerplant would flow into Sunnyside Canal for downstream delivery. However, a small number of Sunnyside Division water users upstream from this point would be served by delivery of 17 to 20 cfs by a pumping plant constructed at MP 3.83 and a buried PVC pipeline extending about 3.2 miles on the right embankment of Sunnyside Canal.

5.8.2.2 Downstream Plan 2 – Modified Roza Canal and New Pipeline

Downstream plan 2 would include a modified Roza Canal and a new pipeline to convey water to Sunnyside Canal. Beginning at MP 22.6, modifications would consist of a new siphon, enlargement of Roza Canal, and construction of a new tunnel No. 5 to carry an additional 1,262-cfs flow to about MP 29.2. At this point, the exchange water would be routed into wasteway No. 3, which would be enlarged from 1,252 cfs to 2,514 cfs to carry the additional flow. Five new check/drop structures would be also installed.

At about 1.75 miles from the headworks of the wasteway, a new turnout would divert Sunnyside Division exchange water into a 12-foot-diameter pipeline extending a little over 0.75 miles to Sunnyside Canal MP 3.83. Water would discharge into Sunnyside Canal through a new Sunnyside powerplant described for downstream plan 1. However, the output of the downstream plan 2 powerplant would be 15 MW at the design head of 221 feet. Deliveries to water users upstream from MP 3.83 would be handled in the same manner as upstream plan 1.

5.8.3 Preliminary Reactions to Appraisal-Level Delivery Plans

Input received from the potential water exchange participants as to concerns and preferences are noted below. Appraisal-level delivery plans affected by the concerns and preferences are also noted.

5.8.3.1 Roza Division Input

- Desire to keep Roza Powerplant in operation and is averse to a plan that would result in hydroelectric generation being terminated.

  ***Comment**: All upstream plans, except plan 1, assume continued diversion for hydroelectric generation at Roza Powerplant.*

- Prefer that Roza Canal be watered up during the irrigation season because the reinforced canal lining was not designed with expansion joints. If the canal were dewatered, there would be a tendency for thermal expansion and buckling of the lining during hot weather. Appraisal-level

delivery plans that include dewatering of the canal may need to consider installation of expansion joints at appropriate intervals.

> ***Comment**: Upstream plans 5 and 6 assume Roza Canal remains watered up from Roza Diversion Dam to the canal's end near Prosser; upstream plans 2, 3, and 4 assume Roza Canal would be watered up from Roza Diversion Dam to MP 11.0 and from MP 22.6 to the canal's end near Prosser.*

- Receptive to delivery plans that would convey water to Selah-Moxee and Union Gap by a new mainline delivery system or by Roza Canal.

> ***Comment**: Upstream plans 4 and 5 include water service to Selah-Moxee and Union Gap.*

- Current operational practice is to flush and prime Roza Canal prior to the irrigation season by diversion of Yakima River March flood flows. These flows then discharge directly to the Yakima River.

> ***Comment**: Because of the direct discharge to the Yakima River, it would be undesirable to use Columbia River water for flushing and priming. The current operational practice would need to continue to rely on Yakima River diversions to prime the canal. Then deliveries from the Black Rock alternative could commence.*

- Wish to keep the existing system in place and operational as a back up to the Black Rock alternative.

> ***Comment**: There is no intent to decommission the existing facilities. Continued operational reliability and integrity of the existing system would need to be ensured.*

5.8.3.2 Sunnyside Division Input

Sunnyside Division has both technical and policy issues as follows:

<u>Technical Issues</u>

- Prefer the delivery system be built with equipment redundancy to ensure service in the event of equipment failure.

- Design velocities of the mainline delivery system (up to 12 feet per second) seem excessive.

- Delivery pressure should be at least 40 pounds per square inch.

- The upper 3.83 miles of Sunnyside Canal and the current Yakima River diversion works would need to remain in place and operable for the months of March and October. This would require a structure to permit downstream passage of water during certain months of the year.

Policy Issues

- How would construction and operation and maintenance costs be allocated for the new delivery system?
- Would the delivery system be reserved or transferred works?
- The delivery raises key issues for linking Roza and Sunnyside Canal operations.
- A fundamental issue is the effect of the potential water exchange on the priority dates of the Sunnyside Division's water rights and the current contracts between Reclamation and Sunnyside Division entities.

> ***Comment**: Future phases of the Storage Study would address the above issues if the Black Rock alternative proceeds to the next phase.*

5.8.3.3 Selah-Moxee Irrigation District Input

The appraisal-level design for upstream plans 4 and 5 would include the delivery of Columbia River water to Selah-Moxee. The district's main canal diverts from the Yakima River near Pomona, runs parallel to and downslope of Roza Canal, tunnels through the Yakima Ridge, and ends in the southeast side of Moxee Valley. For design purposes, it was assumed the district's water demand was about 100 cfs with 80 cfs required downstream from the Yakima Ridge tunnel. The Yakima River would deliver the remaining 20 cfs. The appraisal-level design would limit service with use of Columbia River exchange water to the area downstream from the tunnel.

5.8.3.4 Union Gap Irrigation District Input

Appraisal-level design for upstream plans 4 and 5 also would include the delivery of Columbia River water to Union Gap. Currently, the district diverts from the Yakima River downstream from Pomona; its main canal is parallel to and downslope of Roza and Selah-Moxee Canals. After passing through Union Gap and to its end point north of Zillah, the canal is upslope of Sunnyside Canal.

All of the Union Gap service area lies downstream from Roza Canal MP 11.0. The district has indicated an interest in receiving Columbia River water as a full in-lieu supply as long as there would be no additional cost to the district.

> ***Comment**: The question of allocation of project costs is beyond the scope of this Assessment.*

5.8.4 Delivery System Conclusions

5.8.4.1 Upstream From MP 22.6

Of the six appraisal-level plans developed for delivery of Columbia River water upstream from MP 22.6, only plans 4 and 5 would provide service to Roza, Terrace Heights, Selah-Moxee, and Union Gap Irrigation Districts. Roza Irrigation District has indicated a preference for plans that would not dewater any portion of Roza Canal. Only plan 5 would be viable under this condition while including all the irrigation districts.

Further, with the Black Rock alternative, there would be the need to maintain the continued operational viability of Roza Canal for use in the event of some type of system outage that would preclude delivery of exchange water. Any water delivery plan that precludes such emergency operation would be of concern. This could impact the viability of plan 5 unless the checks were capable of passing approximately 1,100 cfs downstream from MP 11.0.

Based on this information, it may be desirable to forego delivery of any Columbia River exchange water upstream from MP 22.6. Under this scenario, about 325 cfs would not be exchanged. However, this decision should be deferred pending examination of the benefits associated with inclusion of such service; this Assessment does not include a benefit analysis.

Upstream delivery plans 4, 4A, and 5 should receive further evaluation. The dewatering of Roza Canal under the current plan 4 would, however, have to be assessed further. Plan 6 would involve no delivery system construction costs, but because only 35 cfs would be associated with this exchange, and the potential for operational concerns, further analyses of this plan may not be warranted.

5.8.4.2 Downstream From MP 22.6

At this time, there is potential for a water exchange with Roza and Sunnyside Divisions downstream from Roza Canal MP 22.6. The potential 885-cfs downstream water exchange with Roza Division could be incorporated with each of the upstream plans and should receive further evaluation.

For the potential 1,262-cfs water exchange with Sunnyside Division, both plans 1 and 2 should receive further evaluation. Plan 2 would involve policy issues of modifying Roza Canal to permit conveyance of Sunnyside Division's exchange water from MP 22.6 to MP 29.2. The joint use of Roza Canal must be addressed if the Black Rock alternative proceeds to the next phase of the Storage Study.