Sites Program Management Team

Final Feasibility Level Basis of Design Report – HC Conveyance Facilities

Sites Reservoir Project

August 28, 2020

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1. Background

The initial concept of the Sites Reservoir Project was initiated in 1957 by the California Department of Water Resources and by the U.S. Bureau of Reclamation (Reclamation) in 1964. Over the decades, the project size and components have been revised and studied to gain a better understanding of the need and cost.

On August 26, 2010, the Sites Project Authority (Authority) was formed when seven regional entities, including several local water agencies and counties, executed the Joint Exercise of Powers Authority. The primary purpose of the Authority, as stated in the agreement, is to pursue the development and construction of the Sites Reservoir Project, which has long been viewed as an ideal location for additional off-stream storage to provide direct and real benefits to instream flows, the Delta ecosystem, and water supply.

Most recently, a feasibility study was completed by Reclamation that results in a project cost of roughly $5 billion. In September 2019, representatives from the Authority Board and Reservoir Committee began undertaking a value planning process to identify and evaluate additional alternatives that could make the project more affordable for the project's participants. This decision was based on ongoing discussions with permitting agencies, expected project cost, cost per acre-foot, and existing participation levels. An Ad Hoc Value Planning Workgroup was formed and continued to meet through April 2020. The workgroup produced the "Sites Project Value Planning Alternatives Appraisal Report" in April 2020 and recommended that the Authority adopt alternative VP7, which was a 1.5-million-acre-foot reservoir and project, with an estimated cost of $3 billion. The Authority approved moving forward with alternative VP7; the conveyance facilities described in this report reflect the factors and components of alternative VP7.

1.1 Project Description

The project consists of a large reservoir, ancillary roads, and conveyance facilities. The Authority decided to segregate the design of these facilities into an HR (Segment H Reservoir) segment that is responsible for design of the reservoir features, including several dams, inlet/outlet tunnels at Golden Gate Dam, as well as relocation of roads displaced by the reservoir. The other segment is known as the HC (Segment H Conveyance) segment and includes improvements to the two existing diversion canals from the Sacramento River to the Project Area (Tehama-Colusa Canal and Glenn-Colusa Irrigation District Canal), regulating reservoirs (existing Funks Reservoir and new Terminal Regulating Reservoir), two pumping generating plants (PGP), large-diameter pipelines from each PGP to Sites Reservoir, and a large-diameter pipeline to convey water from the Tehama Colusa Canal (TCC) to the Colusa Basin Drain or Sacramento River near Dunnigan, California. Detailed descriptions of each facility are provided in the next section.

1.1.1 General Description of Facilities

Following is a list of the individual new facilities and existing facilities requiring improvements.

- Improvements to the TCC Authority Red Bluff Pumping Plant on the Sacramento River
- Glenn-Colusa Irrigation District (GCID) Canal Improvements upstream of the TRR
- Terminal Regulating Reservoir (TRR)
- TRR PGP
- TRR Pipelines
- Funks Reservoir – Sediment Removal
- Funks PGP
- Funks Pipelines
- Western Area Power Administration (WAPA) or Pacific Gas and Electric (PG&E) Substation/Switchyard
- Power Transmission Lines
- Dunnigan Pipeline
- Administration and Operations Building

- Maintenance and Storage Building
- Access Roads

Improvements to the TCC Red Bluff Pumping Plant

The Red Bluff Diversion is located on the Sacramento River in Red Bluff, California. The facility includes a 2,500 cubic feet per second (cfs) capacity 1,180-foot-long fish screen structure, forebay, pumping plant (current capacity 2,000 cfs), an electrical switchyard, and a 660-foot-long access bridge, canal, and siphon under Red Bank Creek, to deliver water from the Sacramento River into the TCC and Corning Canal. This facility was constructed and put into operation in October 2012. The pumping plant was designed to accommodate the Sites Project and includes space to add two additional 250 cfs pumping units, bringing the total pumping capacity to 2,500 cfs.

GCID Canal Improvements

The GCID Main Canal delivers water from the Sacramento River to water users along its route, from its diversion point approximately 5 miles northwest of Hamilton City to southeast of the City of Williams. The canal is a 65 mile unlined earthen channel, with capacity varying from 3,000 cfs at the upstream end to 300 cfs at the southern terminus. Water conveyed by the canal is pumped by the Hamilton City Main Pump Station into the GCID Main Canal.

Improvements to the GCID Main Canal will include a 3,000 cfs headworks structure just downstream of the Hamilton City Diversion, the railroad siphon at Willows, and miscellaneous other structures yet to be identified.

TRR

This is a new reservoir that will be hydraulically connected to the GCID Canal a few miles east of Funks Reservoir.

TRR PGP

This is a pumping and generating plant that will be used to pump water from the TRR to the Sites Reservoir. This facility will also include hydroelectric turbines to generate electricity when flow is released from Sites Reservoir to the TRR and GCID Canal. As part of this PGP facility, there will also be an energy-dissipation facility that will allow releases back to the TRR as backup to the hydroelectric turbine facilities.

TRR Pipelines

These are two parallel, 12-foot-diameter pipelines used to convey water between the TRR PGP and the Sites Reservoir. These pipelines will connect from the piping manifold at TRR PGP to the downstream side of the two proposed 23-foot-diameter tunnels connected to the Site Reservoir inlet/outlet structure. The approximate length of these pipelines is 5 miles each.

Funks Reservoir

Reclamation constructed the Funks Reservoir in the mid-1970s with the intent of providing operational flexibility for the TCC. There are check structures on the TCC just upstream and downstream of the reservoir. The TCC is located about 1 mile east of the proposed Sites Reservoir. At the time of construction, the reservoir had a useable capacity of 1,170 acre-feet between operating levels of 199.5- and 205.2-feet elevation, and 1,080 acre-feet of inactive storage below elevation 199.5 feet, for a total capacity of 2,250 acre-feet; however, the addition of sediment from Funks Creek and the TCC have likely reduced the total storage volume. The spillway has a capacity of 2,500 cfs. The project will remove accumulated sediment to recapture the design storage volume.

Funks PGP

This is a pumping and generating plant that will be used to pump water from Funks Reservoir to the Sites Reservoir. This facility will also include hydroelectric turbines to generate electricity when flow is released from Sites Reservoir to Funks Reservoir and, ultimately, the TCC. There will also be an energy-dissipation facility as part of this PGP facility that will allow releases back to Funks Reservoir as backup to the hydroelectric turbine facilities.

Funks Pipelines

These are 2 parallel, 12-foot-diameter pipelines used to convey water between the Funks PGP and the Sites Reservoir. These pipelines will connect from the piping manifold at Funks PGP to the downstream side of the two proposed 23-foot-diameter tunnels connected to the Site Reservoir inlet/outlet structure. The approximate length of these pipelines is 1 mile each.

Dunnigan Pipeline

The Dunnigan pipeline consists of either a 9-foot-diameter or 10.5-diameter pipeline that will be used to release water from the TCC to the Sacramento River. The concept is to release flow from Sites Reservoir to Funks Reservoir, where the flow will then go south about 40 miles to near the end of the TCC. At this point, flow will be diverted into the Dunnigan pipeline, where flow will head either to the Colusa Basin (which flows to Sacramento River) or directly to the Sacramento River. The pipeline is about 4 miles long to the CBD, or 10 miles long if it goes directly to the Sacramento River.

WAPA or PG&E Substation/Switchyard

There are 230 kilovolt (kV) electrical transmission lines running near the proposed project area. Specifically, the WAPA transmission lines run very close to Funks Reservoir in a north-south direction, with a parallel 230 kV line owned by PG&E a few miles east of the WAPA transmission lines. It is anticipated that one of these transmission lines will be connected to provide power for the project, as well as receive generated electrical power from the hydroelectric turbines. Switchyards and substations will be needed to provide power to both the TRR and Funks sites.

Electrical Transmission Lines

Electrical transmission lines will be required to connect the WAPA or PGE 230 kV transmission lines to the TRR PGP and the Funks PGP.

Administration and Operations Building

At this time, staffing requirements for operating and maintaining the Sites facilities are unclear, but an administration and operations building is anticipated to be needed.

Maintenance and Storage Building

A building is also expected to be required to provide maintenance and storage associated with the Project.

Access Roads

Access to the proposed TRR site would likely be from McDermott Road, which lies adjacent to the proposed reservoir. Access to the Funks complex (PGP and Reservoir) is currently done using the O & M road along the TCC. A new access road will be required that allows larger equipment and year-round access. It is also anticipated that roads will be constructed within the TRR and Funks Pipeline easements, both to provide access to the pipelines and electrical power transmission lines, but also as a secondary access road to the project facilities.

2. Design Criteria

2.1 General Criteria

The purpose of this section is to provide the currently known design criteria used as the basis for design. There have been previous studies of these facilities that summarized design criteria, some of which are applicable to this current project; but, in many cases, the criteria have changed. Given that the design is in very early stages, these criteria are subject to change as the design progresses.

2.1.1 Civil – Site Design

The site design for this project will include grading, drainage, site security, and access roads. General criteria have been established for these features, as summarized in Table 1.

TABLE 1: SITE CIVIL DESIGN CRITERIA

Subject	Criteria
Coordinate System	
Horizontal Datum	NAD83, California State Plane Coordinates, Zone 2
Vertical Datum	NAVD 88
Drainage	
Depth	Minimum 1 foot
Ditch Drain Slopes	Minimum 0.5 percent
Ditch Side Slopes	Maximum 2H:1V
Roads – General	
Vehicle	WB-65 design
Standards	AASHTO – Green Book
Paved Roads	
Cross Slopes	Minimum 2 percent Maximum 4 percent
Grade	Maximum 6 percent
Widths	30 foot maximum 24 feet minimum
Unpaved Roads	
Fill Slopes	Recommended 4:1 Maximum 2:1
Cut Slopes	Recommended 4:1 Maximum 2:1

Table 1 will be revisited after the geotechnical field investigation is completed to confirm or revise.

Existing topography for this area was provided by the U.S. Bureau of Reclamation and was used in the previous feasibility study by AECOM. The mapping is set in NAD83 State Plane Coordinate system CA, Zone 2, U.S. survey feet. The vertical datum appears to be NAVD 88, based on a comparison of the Funks Reservoir Dam crest elevation with the elevation shown in the original as-built drawings for the Funks Dam from December 2, 1974 (which are currently assumed to be on NGVD29). We are proceeding with this design assumption, while awaiting confirmation from Reclamation on the vertical datum of the contours they provided.

2.1.2 Civil – Pipeline Design

This section describes the criteria for the transmission main pipelines, including Funks, TRR, and Dunnigan pipelines. Initially, pipe types being considered included: 1) reinforced-concrete cylinder pipe – American Water Works Association (AWWA) Standard C300-16, and AWWA M9-3rd edition; and 2) welded steel pipe (WSP) AWWA C200-05 and AWWA M11-4th edition. Based on experience, the application for this pipe and discussions with pipe manufactures for both reinforced-concrete cylinder pipe and WSP, the pipe anticipated for use on this project is WSP. Appendix A contains detailed design criteria for the transmission pipelines using WSP.

2.1.3 Mechanical Design

To be Added in Future Design Phases – see later sections for specific criteria for Funks and TRR.

2.1.4 Structural Design

The structures described in this document will be designed in accordance with the current governing codes and standards applicable to the construction of buildings, structures, and appurtenances in the State of California.

Reinforced Concrete Hydraulic Structures

Hydraulic structures will be designed in accordance with ACI 350, using the alternate service load method or the strength method with durability factors.

Miscellaneous Concrete Structures

Miscellaneous, non-hydraulic, concrete structures, including building slabs and foundations, will be designed in accordance with ACI 318.

Structural Steel

Structural steel will be designed in accordance with the latest edition of the American Institute of Steel Construction (AISC) Steel Construction Manual and the AISC Specification for Structural Steel Buildings. Design will use either the allowable stress design method or the load and resistance factor design. Structural steel bolted connections shall be designed in accordance with the RCSC Specification for Structural Joints Using High Strength Bolts.

Masonry Structures

Masonry structures shall be designed in accordance with the latest edition of The Masonry Society Building Code Requirements and Specifications for Masonry Structures (TMS 402). Design shall use either the allowable stress design method or the load and resistance factor design method.

Loads

Table 2 shows the structural loads.

TABLE 2: STRUCTURAL LOADS

Subject	Criteria
Gravity	
Dead	Weight of structure and permanent equipment
Floor Live	Superimposed uniform or concentrated loads
Roof Live	20 psf minimum
Conform to ASCE 7, Chapter 4 except:	Process Areas – 200 psf
	Electrical Rooms – 300 psf
	Where significant, use the actual weights of equipment

TABLE 2: STRUCTURAL LOADS

Subject	Criteria
Wind	
Power Generating Facilities: Conform to ASCE 7, Chapter 26-30:	Risk Category III
	Basic Wind Speed (ASCE 7-16 3 second gust) 99 MPH
	Exposure Category C
All other facilities: Conform to ASCE 7, Chapter 26-30:	Risk Category II
	Basic Wind Speed (ASCE 7-16 3 second gust) 93 MPH
	Exposure Category C
Earthquake	
Conform to following references:	CBC Chapter 16
	ASCE 7 Chapters 11-23
	Power Generating Facilities: Risk Category III All other facilities: Risk Category II
	Project Geotechnical and Seismicity Report
	Hydrodynamic Loads – ACI 350.3, Seismic Design of Liquid Containing Concrete Structures and ASCE 7-16 Chapter 15
Other	
Vehicle	AASHTO HS-20 Truck and Caltrans P13 permit vehicle
Lateral Earth Pressures	Conform to criteria listed in the project geotechnical report
Flood Protection	Design for 100-year frequency flood levels, including debris protection and location of all critical equipment (pumps, panels)
Load Combinations	Conform to ASCE 7 Chapter 2 for service and strength level combinations

psf = pounds per square foot

Materials

Table 3 shows the structural materials and properties.

TABLE 3: STRUCTURAL MATERIALS AND PROPERTY

Material	Property
Structural concrete	F’c = 4,500 psi at 28 days, normal weight
Reinforcing steel	ASTM A615, Grade 60, F_y = 60 kilopounds per square inch (ksi)
Reinforcing steel to be welded	ASTM A706, Grade 60, F_y = 60 ksi
Masonry	
Concrete masonry units (CMU)	TMS 6-13, 1.4.B.2, F’c = 2,000 psi
Mortar	ASTM C270, F’c = 1,800 psi
Grout	ASTM C476, F’c = 2,000 psi
Structural Steel	
Steel plates, shapes except W-shapes, and bars	ASTM A572 Gr. 50, F_y = 50 ksi or ASTM A36, F_y = 36 ksi
W-shapes and WT-shapes	ASTM A992, (Fy = 50 ksi)
Rectangular (and square) hollow structural sections (HSS)	ASTM A500 Grade C, F_y = 50 ksi
Round hollow structural sections (HSS)	ASTM A500 Grade C, F_y = 46 ksi
Steel pipe	ASTM A53, Grade B, F_y = 35 ksi

TABLE 3: STRUCTURAL MATERIALS AND PROPERTY

Material	Property
Steel pipe for sleeves and piles	API Specification for Pipeline 5L, PSl2; sleeve grade 52; piles grade 65
Stainless Steel	
Bars and shapes	ASTM A276, AISI Type 304L, F_y = 25 ksi
Plate	ASTM A167, AISI Type 304L, F_y = 25 ksi
Bolts and Rods	
High-strength steel bolts	ASTM A325, Type 1 bolts with A563 nuts
Steel bolts	ASTM A307, Grade B or A36
Anchor rods	ASTM F1554, Grades 36, 55, and 105 (hooked, headed, or threaded and nutted) as appropriate for application or ASTM A36 (threaded rods either plain or upset ends)
Stainless steel bolts	ASTM F593, AISI Type 304, Condition CW
Post-tensioning bars	ASTM A722, 150 ksi ultimate stress
Concrete-adhesive anchors	Stainless Steel Hilti or equal
Concrete-grouted anchors	Galvanized
Grating	
Grating for pedestrian loads	Galvanized steel
Grating vehicular loads	Galvanized steel
Fiberglass grating	Molded or pultruded fiberglass
Handrail and guardrail	Galvanized steel

2.1.5 Electrical Design

Table 4 summarizes electrical design criteria.

TABLE 4: ELECTRICAL DESIGN CRITERIA

Subject	Criteria	Comments
Utilization Voltage	• Motor control for large pump motors – 13,800 volts (V) • Motors for large pump motors – TBD • Motors 1HP to 400HP – 480V • Fractional HP motors – 120V, 208V, 480V, as required • HVAC – 120V, 208V, 480V, as required • Convenience Loads – 120V • Lighting – 120, 208V, as required	For large pump motors on variable-speed drives, motor voltage will be determined by equipment supplier as best suited for the motor\drive equipment selection. Voltage will be selected in the range of 4160V to 13,800V.
Medium-Voltage Switchgear	Metal-Clad Switchgear, 15kV, 3,000A maximum. Standards: Institution of Electrical and Electronics Engineers, Inc. (IEEE) C37.20.2.	
Medium-Voltage Motor Control	Voltage source, variable-speed drive, with the following features: • Input section for isolation, short circuit, and overload protection • 24-pulse or higher isolation transformer • Motor protection relay	Variable-speed drive allows pump to operate more efficiently, provides near unity power factor on the line side of the drive, and allows the motor voltage to be different than the line voltage for better coordination of motor and drive equipment selection.

General Requirements

An electrical building is planned to house electrical equipment for distribution, motor control, and hydro generation. The similarity between the Funks and TRR pumping generating plants will allow for similarity in the general arrangement of the electrical building and electrical equipment. Consistency between the designs of these facilities will be accommodated to the extent possible. Electrical equipment will be specified that complies with NEMA and American National Standards Institute (ANSI)\IEEE standards. UL-listed or label equipment, or equipment label or listed by a Nationally Recognized Testing Laboratory, will be specified where equipment with the required ratings can be provided in accordance with their respective safety standards.

Electrical power will be received from the substation at 13.8kV by two separate lineups of switchgear. The switchgear will distribute at this voltage for motor control, and connection of the hydro generation, and will be stepped down for low-voltage use. Three-phase 480V and 208/120V power will be provide for ancillary equipment.

Medium-voltage Switchgear

Two lineups of indoor, metal-clad, switchgear will divide power distribution as evenly as possible between the medium-voltage loads. A main-tie-main bus arrangement will be provided to isolate medium- and low-voltage buses and reduce the effect of a bus fault, feeder fault, or switchgear failure on the operation of equipment. A remote operating panel will be provided to allow operator control of circuit breakers. Arc-resistance switchgear and arc-flash energy hazard reduction techniques will be evaluated.

Medium-voltage, Variable-speed Drives and Motor Protection

Variable-speed drives will be provided for large induction motors. An input section, with disconnect switch and fused vacuum contactor or circuit breaker, will be specified to provide isolation from the medium-voltage switchgear without having to rack out a switchgear circuit breaker. Each drive will be a fully integrated system, capable of controlling medium-voltage power at its input and operating the driven equipment from the local control interface without external programmable devices. Each drive will have its own short-circuit and overload protection. Drives will be provided with a separate low-voltage power for ventilation and control. Line side harmonics will be mitigated with a 24-pulse or higher isolation transformer, which is standard for drives connected to 8,000-horsepower and higher motors.

Variable-speed drives for large motors can be provided with air or water cooling. Air-cooled drives typically require large air-conditioning systems to remove heat from the drive out of the building. Air-cooled technology is recommended to reduce cost and complexity of drive maintenance.

Motor protection will be provided by a separate motor protection relay (MPR) integrated into the drive system. The MPR will allow current- and temperature-based motor protections to be provided, independent of the variable-speed drive control. The MPR will provide connections for motor stator, motor bearing, and pump bearing temperature sensors, as well as for current transformers to measure phase, ground, and differential current.

Standby Power

Depending on operational requirements, standby uninterruptible power supplies and/or standby generation will be provided to maintain operation of plant control, communications, and ancillary equipment. Because of the size of the pumping units, no backup generation is planned for pumping facilities.

2.2 Red Bluff Diversion Improvements

The Red Bluff Diversion Pumping Plant that currently pumps water into the TCC has a capacity of 2,000 cfs. A portion of this flow also is diverted to the Corning Canal. It is anticipated that additional flow beyond the 2,000 cfs will be required to convey flow to Funks Reservoir and, ultimately, Site Reservoir. The current design involves adding two 250 cfs pumps to match the existing seven 250 cfs pumps. The existing pumping plant also includes two 125-cfs pumps. This will increase the capacity from 2,000 to 2,500 cfs.

Improvements will include the two pumps and motors, discharge piping, electrical switchgear, and instrumentation and control to program pump operation.

2.3 GCID Canal Improvements

GCID recently gave the design team a preliminary list of improvements resulting from the plan to use the GCID Canal to convey water for the Sites Project. These improvements include:

- Construct new Main Canal head gate
- Increase capacity of railroad siphon near Willows

Jacobs has been coordinating with GCID and expects to obtain more detail on the final list of required improvements and details associated with each. Once this information is obtained, Jacobs will be able to define design criteria. In some cases, other criteria present in this section for Sites specific facilities may apply, but all criteria will be defined once this information about GCID facilities is obtained.

2.4 Funks Area

2.4.1 Funks Reservoir

The existing Funks Reservoir is a regulating reservoir on the TCC. The reservoir will be used as a source of water to pump to and receive discharged water from Sites Reservoir. The Funks Reservoir water surface elevation (WSE) is controlled by a check structure (#16) where the TCC enters the reservoir and a check structure (#17) where the TCC leaves the Sites Reservoir. Therefore, the reservoir operational WSE can only vary slightly from the TCC, especially since the reservoir contains no irrigation specific turnouts. The reservoir WSE typically ranges from 200 to 205 feet, although the preferred operating range is 202 to 204 feet. The minimum WSE in the reservoir is 199 feet, in order to flow south in the TCC while providing the head required for downstream turnouts.

The initial volume of the Funks Reservoir is approximately 2,200 acre-feet, but only about 1,100 acre-feet is useable storage within the TCC operating range. Based on input from TCC operations staff, sediment has deposited in the reservoir and predominately on the western side, which has reduced the total storage. The Funks PGP is expected to be located on the western shore of where most of the sediment build-up has likely occurred. Other than reshaping the shoreline of the reservoir to accommodate the Funks PGP and possibly performing excavation and fill to place the TRR pipelines across the north side of the reservoir, the only improvement is to remove sediment. At this time, it is unclear how much sediment removal is required (this will be determined in the fall of 2020); however, the need to excavate sediment from the reservoir to an elevation of around 188 feet near the proposed PGP on the western side is anticipated. The excavation and reshaping of the reservoir bottom is necessary to allow the large flow to and from the PGP to be unimpeded. As the design progresses, additional information will determine the minimum reservoir depth near the Funks PGP.

An additional benefit of placing the PGP on the western side is providing better circulation of flow in the reservoir, given the canal inflow and outflow are on the eastern side of reservoir. This design also uses the currently unusable storage volume below elevation 199 feet. Although the useable storage will not change, adding the PGP will allow almost all of the reservoir to have circulation due to the PGP since the PGP inlet floor elevation is near the bottom of the reservoir.

2.4.2 Funks Pumping Generating Plant

General Information

Table 5 overviews the components of the Funks area.

TABLE 5: FUNKS AREA

Subject	Criteria
Flows	
Pumping Plant	2,100 cfs
Generating Plant	1,000 cfs
Energy Dissipation	1,000 cfs

Consideration was given to using a single unit to provide both pumping and generation versus having separate units. As part of this analysis, a comparison of the units at the Gianelli PGP at San Luis Reservoir, which contain single units to pump and generate, versus the proposed Sites Project which recommends using separate pumps and generating turbines. This comparison is summarized in a Technical Memorandum contained in Appendix B.

Mechanical Design

Table 6 details the mechanical design of the pumping design components.

TABLE 6: FUNKS PUMPING UNIT

Subject	Criteria
Operational Criteria	
Pumping Plant	2,100 cfs
Pumps, Motors, and Ancillary Equipment	
Number of Pumping Units	13 (12 duty + 1 standby)
Capacity at Rated Point	175 cfs @ 320 feet
Static Head, Maximum	298 feet
Static Head, Minimum	135 feet
Rated Pump Efficiency	89 percent
Pump Type and Configuration	Vertical Mixed Flow, Multi-Stage
Pump Shaft Lubrication	Shaft-enclosing Tube with External water-flush, or Oil-drip System
Pump Shaft Seal	Packing
Pump Materials	
Discharge Head	Fabricated Steel, Epoxy L&C
Column	Fabricated Steel, Epoxy L&C
Shaft	17-4 PH Stainless Steel
Impellers	Silicon Bronze or Stainless Steel
Bowls	Cast or Ductile Iron
Impeller Wear Rings	Bronze
Bowl Wear Rings	Stainless Steel
Lineshaft Bearings	Water-Flush: Synthetic Rubber Oil-Drip: Bronze
Motor	
Size	8,000 horsepower
Type	Induction, Vertical Solid Shaft, High Thrust
Nominal Speed	505 rotations per minute (rpm)
Voltage	4,000 V or 13,200 V
Enclosure	WPII
Ambient Rating	50 degrees Celsius
Non-Reverse Ratchet	No

TABLE 6: FUNKS PUMPING UNIT

Subject	Criteria
Insulated Bearings and Shaft Grounding	Yes
Drive Type	Adjustable Speed Drives
Valves and Accessories	
Large Isolation Valves	Butterfly, Class 250B, per AWWA C504
Small Isolation Valves	Ball, Bronze or Stainless Steel
Large Check Valves	Tilting Disc w/Hydraulic Damper
Small Check Valves	Swing, Bronze
Air Valves	Cast or Ductile Iron with Stainless Steel Trim

Table 7 details the mechanical design of the generating unit components.

TABLE 7: FUNKS GENERATING UNITS

Subject	Criteria	
Operational Criteria		
Generating Flow	2,000 cfs (2 @ 1,000 cfs each)	
Turbines		
Type of Units	Francis Turbine	
Head, Maximum	289.5 feet	
Head, Minimum	125.4 feet	
Capacity at Rated Point	1,000 cfs	1,000 cfs
Head, Rated	255.9 feet	190.3 feet
Operational Head, Maximum	319 feet	238 feet
Operational Head, Minimum	153.5 feet	114 feet
Rated Turbine Output	20 megawatts (MW)	14.5 MW
Speed	360 rpm	300 rpm
Generator		
Size	*Waiting for Information from Manufacturers*	
Type		
Nominal Speed		
Voltage		
Valves and Accessories		
Turbine Isolation Valves	78” Ball, Hydraulic Operator	

Structural Design

The structures within the pumping generating plants will be designed to support and access the mechanical, electrical, and control equipment. The pumping generating plants include the following significant structures:

- Pump Station
- Turbine Generator Building
- Energy-Dissipation Valve Structure

Pump Station

The pump station structure will support the pumps at the edge of the reservoir. The pump station will be designed with specific attention given to pump vibration. The pump station will be sized to create a fundamental baseline frequency rate that provides at least 20% frequency separation from the pumps to avoid resonance. A trash rack will be installed at the front of the wet well to prevent debris from entering the wet well. The trash rack will be designed for a maximum head differential of 3 feet. Bulkhead slots will be provided at each wet well to allow bulkheads to be installed and isolate pumps bays for maintenance.

Turbine Generator Building

The turbine generator building will house the Francis turbine, generator, draft tube, turbine inlet valve, associated piping appurtenances, and mechanical and electrical equipment. The turbine will discharge into a draft tube prior to exiting into the reservoir. Consideration will be given for providing access for future maintenance and removal (via temporary crane and removable roof sections) of all major pieces of equipment.

The aboveground portion of the building will consist of CMU walls. The building is assumed to be without personnel and will provide only minimal ventilation and heating to suit the housed equipment. Unit control will be possible from local panels in the turbine generator buildings, but primary control is assumed to be remote.

Energy-Dissipation Valve Structure

The energy-dissipation valve structure includes a stilling basin and fixed-cone valve to dissipate energy before water enters the reservoir. The geometry of the stilling basin is sized in accordance with *Hydraulic Design of Stilling Basins and Energy Dissipators* (A.J. Peterka, USBR, 1984).

Electrical Design

All electrical equipment for the pumping units, as well as lower-voltage auxiliary power, will be placed inside a building. The objective is to place this building as close to the pumping units to provide shorter conductor lengths to the pumps. Having the motor control equipment and starters close to the pumping units also allows quick access for operational reasons.

2.4.3 Pipeline

The Funks dual 12-foot-inner-diameter (ID) pipelines connect to the 23-foot Sites Inlet/outlet tunnels, via a transition manifold, which includes isolation valves. From the connection, the pipelines generally run east in parallel with the TRR dual pipelines. After curving around Funks Creek and hilly areas, the Funks pipelines run south, deviating from the TRR pipelines alignment, to the Funks Pumping Generating Plant. The total length of the pipeline alignment is approximately 1 mile.

2.4.4 Administration and Operations Building

The administration and operations building contains the offices, control and communications rooms, and restrooms required to operate the project facilities. The building will be constructed with CMU walls and designed in accordance with the California Building Code. Finalization of the floorplan and color scheme for this building will be addressed in future design phases.

2.4.5 Maintenance and Storage Building

The maintenance and storage building include space for equipment storage and maintenance rooms to support the project facilities. The building will be constructed with CMU walls and designed in accordance with the California Building Code. Finalization of the floorplan and color scheme for this building will be addressed in future design phases.

2.5 TRR Area

2.5.1 TRR Reservoir

The TRR will be connected hydraulically to the existing GCID Canal. The GCID Canal is the conveyance source of water for the TRR and its PGP to pump water into Sites Reservoir. The GCID Canal is also the primary conveyance for releases of water from Sites Reservoir through the PGP and into the TRR. The GCID Canal operational ranges, capacities, and other operational constraints and considerations are the primary factors in the design criteria for the TRR.

The reach of the GCID Canal that is adjacent to TRR is delineated upstream by a check structure that is approximately 6 miles upstream of the location of TRR and is delineated downstream by a check structure adjacent to the southern end of the TRR (this check structure is at the siphon under Funks Creek). Within this reach of the GCID Canal, adjacent to TRR, the operating level (the WSE) within the canal, typically ranges from 123.0 to 123.2 feet in the summer, and typically goes no lower than 121.8 feet in the winter. The maximum design WSE is 124.0 feet, and the lowest WSE corresponds to a drained condition in the canal during the winter shutdown (January to February). The bottom of the canal is approximately elevation 112.7 feet, and the canal embankment crests range from about elevation 128 to 130 feet. The maximum flow capacity of the GCID Canal is understood to be approximately 1,800 cfs.

The TRR PGP is sized to correspond to the 1,800 cfs maximum capacity of the GCID Canal. A design criterion for the project is the ability to capture and store water from the GCID Canal in the TRR in the event the TRR PGP shuts down unexpectedly. That is, when the TRR PGP shuts down, terminating pumping from the TRR and GCID Canal (at rates up to 1,800 cfs), continuing flows from the GCID Canal will either be accommodated by the TRR or continue down the GCID Canal. The TRR is intended to accommodate (capture) those flows because that quantity/flow of water would otherwise not be used or captured downstream of this location. The GCID Canal can be operated to shut off these flows, but doing so takes at least 2 to 3 hours. Therefore, the corresponding design criterion for the TRR is to be able to accommodate inflows of up to 1,800 cfs, for up to 4 hours. This corresponds to a storage capacity (for these inflows alone) of approximately 600 acre-feet which the storage volume used for design of the TRR.

Additional storage capacity within the TRR requires either additional plan-area size (for hydraulic needs within this same elevation range within the TRR) or accommodation either above or below this elevation range, depending on the hydraulic constraints.

2.5.2 TRR Pumping Generating Plants

Mechanical Design

Table 8 details the components of the pumping units.

TABLE 8: TRR PUMPING UNITS

Subject	Criteria
Operational Criteria	
Pumping Plant	1,800 cfs
Pumps, Motors, and Ancillary Equipment	
Number of Pumping Units	13 (12 duty + 1 standby)
Capacity at Rated Point	150 cfs @ 420 feet
Static Head, Maximum	379 feet
Static Head, Minimum	216 feet
Rated Pump Efficiency	88 percent
Pump Type and Configuration	Vertical Mixed Flow, Multi-Stage
Pump Shaft Lubrication	Shaft-enclosing Tube with External Water-flush, or Oil-drip System
Pump Shaft Seal	Packing
Pump Materials	
Discharge Head	Fabricated Steel, Epoxy L&C
Column	Fabricated Steel, Epoxy L&C
Shaft	17-4 PH Stainless Steel
Impellers	Silicon Bronze or Stainless Steel
Bowls	Cast or Ductile Iron
Impeller Wear Rings	Bronze
Bowl Wear Rings	Stainless Steel

TABLE 8: TRR PUMPING UNITS

Subject	Criteria
Lineshaft Bearings	Water-Flush: Synthetic rubber Oil-Drip: Bronze
Motor	
Size	9,000 hp
Type	Induction, Vertical Solid Shaft, High Thrust
Nominal Speed	590 rpm
Voltage	4,000 V or 13,200 V
Enclosure	WPII
Ambient Rating	50 degrees Celsius
Non-Reverse Ratchet	No
Insulated Bearings and Shaft Grounding	Yes
Drive Type	Adjustable Speed Drive
Valves and Accessories	
Large Isolation Valves	Butterfly, Class 250B per AWWA C504
Small Isolation Valves	Ball, Bronze or Stainless Steel
Large Check Valves	Tilting Disc w/Hydraulic Damper
Small Check Valves	Swing, Bronze
Air Valves	Cast or Ductile Iron with Stainless Steel Trim

Table 9 details the components of the generating units.

TABLE 9: TRR GENERATING UNITS

Subject	Criteria
Operational Criteria	
Generating Flow	1,000 cfs (2 @ 500 cfs each)
Turbines	
Type of Units	Francis Turbine
Head, Maximum	370.1 feet
Head, Minimum	206.5 feet
Capacity at Rated Point	500 cfs
Head, Rated	290 feet
Operational Head, Maximum	330.7 feet
Operational Head, Minimum	211.8 feet
Rated Turbine Output	11.5 MW
Speed	514 rpm
Generator	
Size	*Waiting for Information from Manufacturers*
Type	
Nominal Speed	
Voltage	
Valves and Accessories	
Turbine Isolation Valves	60-inch Ball, Hydraulic Operator

Structural Design

The structures within the pumping generating plants will be designed to support and access the mechanical, electrical, and control equipment. The pumping generating plants include the following significant structures:

- Pump Station
- Turbine Generator Building
- Energy-Dissipation Valve Structure

Pump Station

The pump station structure will support the pumps at the edge of the reservoir. The pump station will be designed with specific attention given to pump vibration. The pump station will be sized to create a fundamental baseline frequency rate that provides at least 20% frequency separation from the pumps to avoid resonance. A trash rack will be installed at the front of the wet well to prevent debris from entering the wet well. The trash rack will be designed for a maximum head differential of 3 feet. Bulkhead slots will be provided at each wet well to allow bulkheads to be installed and isolate pumps bays for maintenance.

Turbine Generator Building

The turbine generator building will house the Francis turbine, generator, draft tube, turbine inlet valve, associated piping appurtenances, and mechanical and electrical equipment. The turbine will discharge into a draft tube prior to exiting into the reservoir. Consideration will be given for providing access for future maintenance and removal (via temporary crane and removable roof sections) of all major pieces of equipment.

The aboveground portion of the building will consist of CMU walls. The building is assumed to be without personnel and will provide only minimal ventilation and heating to suit the housed equipment. Unit control will be possible from local panels in the turbine generator buildings, but primary control is assumed to be remote.

Energy-Dissipation Valve Structure

The energy-dissipation valve structure includes a stilling basin and fixed-cone valve to dissipate energy before water enters the reservoir. The geometry of the stilling basin is sized in accordance with the Bureau of Reclamation's *Hydraulic Design of Stilling Basins and Energy Dissipators* (A.J. Peterka, USBR, 1984).

Electrical Design

All electrical equipment for the pumping units, as well as lower-voltage auxiliary power will be placed inside a building. The objective is to place this building as close to the pumping units to provide shorter conductor lengths to the pumps. Having the motor control equipment and starters close to the pumping units also allows quick access for operational reasons.

2.5.3 Pipeline

The TRR dual 12-foot-ID pipelines connect to the 23-foot-diameter Sites Reservoir intlet/outlet tunnels, via a transition manifold, which includes isolation valves. From the connection, the pipelines generally run east in parallel to the Funks dual pipelines. After curving around Funks Creek and hilly areas, the TRR pipelines cross across the top portions of the Funks Reservoir under the waterline. Northeast of Funks Reservoir, the pipelines then cross the TCC by means of a trenchless crossing. East of the TCC, the pipelines continue to run east, parallel to a drainage canal to the GCID Canal. The pipelines cross this GCID canal via trenchless methods before entering the TRR pumping generating plant.

2.6 Dunnigan Pipeline

The Dunnigan pipeline design will be in accordance with the requirements in Appendix A as previously referenced. The pipeline is anticipated to be gravity flow with the following water surface elevation assumptions used for calculating the pipe diameter.

- TCC at inlet structure – 160 feet

- Outlet structure at CBD – 32 feet
- Outfall structure at Sacramento River – 40 feet

Energy dissipation using fixed-cone valves will be required at the downstream end of the pipeline.

2.7 Electrical Supply

2.7.1 Point of Interconnection

The Preliminary 230kV Schematic Plan depicts the point of interconnection (POI) looping in and then back out of the new TRR substation (see Section 3). The TRR substation will then connect to the new Funks substation. This interconnection configuration is subject to approval by the Transmission Operator and the system operator, California Independent System Operator.

The POI, transmission and substation design criteria, dependent on the POI option, will incorporate the following references:

a. California General Order 95, Rules for Overhead Electric Line Construction
b. WAPA Service and Generation Interconnection Requirements
c. PG&E Interconnection Requirements
d. PG&E Substation Design Criteria

The latest edition and addenda of the following publications, as applicable, will be incorporated in the design specifications codes and standards sections.

- ANSI
- IEEE
- Association of Edison Illuminating Companies
- Transmission Interconnections Handbook
- North America Electric Reliability Corporation Standards
- National Fire Protection Agency 70 National Electric Code
- National Electrical Safety Code (ANSI C2)

2.7.2 Transmission Lines

Codes and Standards

In addition to the POI requirements, transmissions lines will be designed in accordance with the latest edition and addenda of the following publications, as applicable, which will be incorporated in the design specifications.

- California Building Code 2016, Title 24 Vol. 2
- ASCE-113, Substation Structure Design Guide
- ASCE/SEI 7-05, Minimum Design Loads for Buildings and Other Structures
- ANSI/AISC 41-10, Seismic Provisions for Structural Steel Buildings

2.7.3 Substations

The transmission operating voltage of 230 kV will be stepped down via transformer, to the 13.8 kV operating voltage of the turbine generators and the pump motors at each new pumping site. The proposed substations will use two 100 millivolt-ampere (MVA) transformers to step down the voltage. The transformers will be three winding, to reduce nominal current ratings below 3,000 amperes and minimize short-circuit levels to comply with Arc Flash requirements in accordance with Occupational Safety and Health Administration regulations. This configuration will allow two independent, double-ended 13.8 kV switchgear lineups to reliably connect the motors and generators to the transmission system.

The substation design will include that the primary safety equipment, including breakers and utility grade relays, to disconnect the interconnection facilities immediately upon a fault detection on the 230 kV transmission system and the 13.8 kV pumping station systems, to minimize potential loss of life and property.

When operating in the generation mode, the facility will automatically trip offline (disconnect from the transmission system) when the relays detect that power has been interrupted on the transmission line into the substation. Transmission line-protective equipment will perform one of the following, as stated anticipated to be in the Interconnection Agreement:

1. Automatically clear a fault and restore power.
2. Rapidly isolate only the faulted section so that the 230 kV system affected by any outage is minimized.

The protection system will be designed with sufficient redundancy such that the failure of any one component will permit the substations to be safely and reliably isolated from the transmission system under fault conditions. Fiberoptic cable will be used for communication protection between each pumping station and the POI.

The substations will include a control enclosure containing power, control, relaying, monitoring and communications. The control enclosure will contain redundant, protective relays and supervisory control and data acquisition (SCADA)/remote telemetry units for transmitting information. The control enclosure will be designed to meet the Bulk Power Protection Criteria of the North America Electric Reliability Corporation Standards, as well Federal Energy Regulatory Commission or Transmission Operator (TO)/California Independent Service Operators (CALISO)-specific requirements.

The TO has standardized their protection requirements; however, system variables will impact the protection requirements, such as generator size and type, number of generators, fault duties, line characteristics (such as, voltage, impedance, and ampacity) and pre-existing protection schemes. For example, high-speed fault clearing may or may not be required to minimize equipment damage and potential impact to system stability.

2.8 Rights-of-way and Easements

2.8.1 Pipelines

Rights-of-way (ROWs) and/or permanent easements (PE) will be required for long-term operation and maintenance of the large-diameter pipelines. In addition, a temporary construction easement (TCE) will be required during initial construction. Following construction, the TCE will no longer be needed and the ROW or PE will be used if repairs are required. Careful consideration must be given to provide widths of PE and TCE that are balanced between what is optimally needed for construction versus the cost of obtaining easements.

Exhibits have been developed to depict the preliminary proposed easements. However, these exhibits have not been thoroughly vetted and are not included at this time.

2.8.2 Transmission Lines

For the interconnect between Funks and TRR, the transmission lines are anticipated to be located parallel and within the same easement as the pipelines. Up to four 230 kV transmission lines are required for the project: two for the source supply to either Funks or TRR (depending on the option), and two for the Funks-to-TRR substations. The two, looped, source circuits will be installed on a set of common double-circuit, steel monopole structures (Figure 1) and have their own easement requirement because they are not parallel to any pipeline. The two Funks-to-TRR circuits will be installed on their own common set of double-circuit, steel monopole structures (Figure 2) within the pipeline easement.

In the sections where four circuits are required, specifically, for the approximately 1 mile between the existing WAPA 230 kV lines and the Funks substation (the WAPA option) or for the approximately 1.7 miles between the existing PG&E 230 kV lines and TRR Substation (the PG&E option), the routing and ROW have not been established. The ROW will depend on the substation locations, the substation orientation for ingress and egress of the transmission lines, and the physical location of the POI. The transmission lines may be located on a common ROW or on separate ROWs.

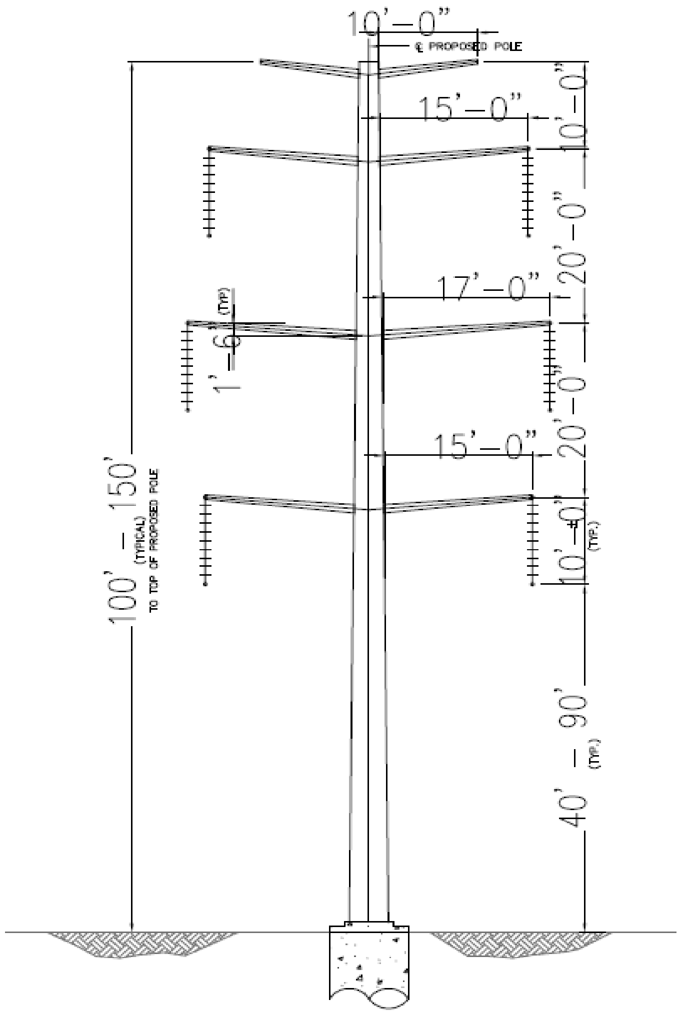


FIGURE 1: DOUBLE-CIRCUIT SOURCE TRANSMISSION POLES

For the approximately 2.9-mile section of the 230 kV Funks-to-TRR transmission lines between the existing WAPA 230 kV lines and TRR (under the WAPA option), or the approximately 2.2-mile section of the 230 kV Funks-to-TRR transmission lines between the existing PG&E 230 kV lines and Funks (under the PG&E option), where just one double-circuit line is required, the preliminary intent will be to route the transmission lines within the 100-foot-wide corridor running parallel to the TRR to Funks water pipeline ROW. The width of the corridor has been estimated based on a suitable edge distance of the lines to the northerly edge of the ROW, as required for trees, future buildings, wire blowout events occurring during strong wind conditions, and transmission line installation and maintenance; the width estimate is also based on suitable edge distance to the southerly edge of the ROW, as required for pipeline equipment access, worker safety, and foundational stability during water pipeline trenching operations. The structures will be spaced to reduce impacts on farming/grazing lands.

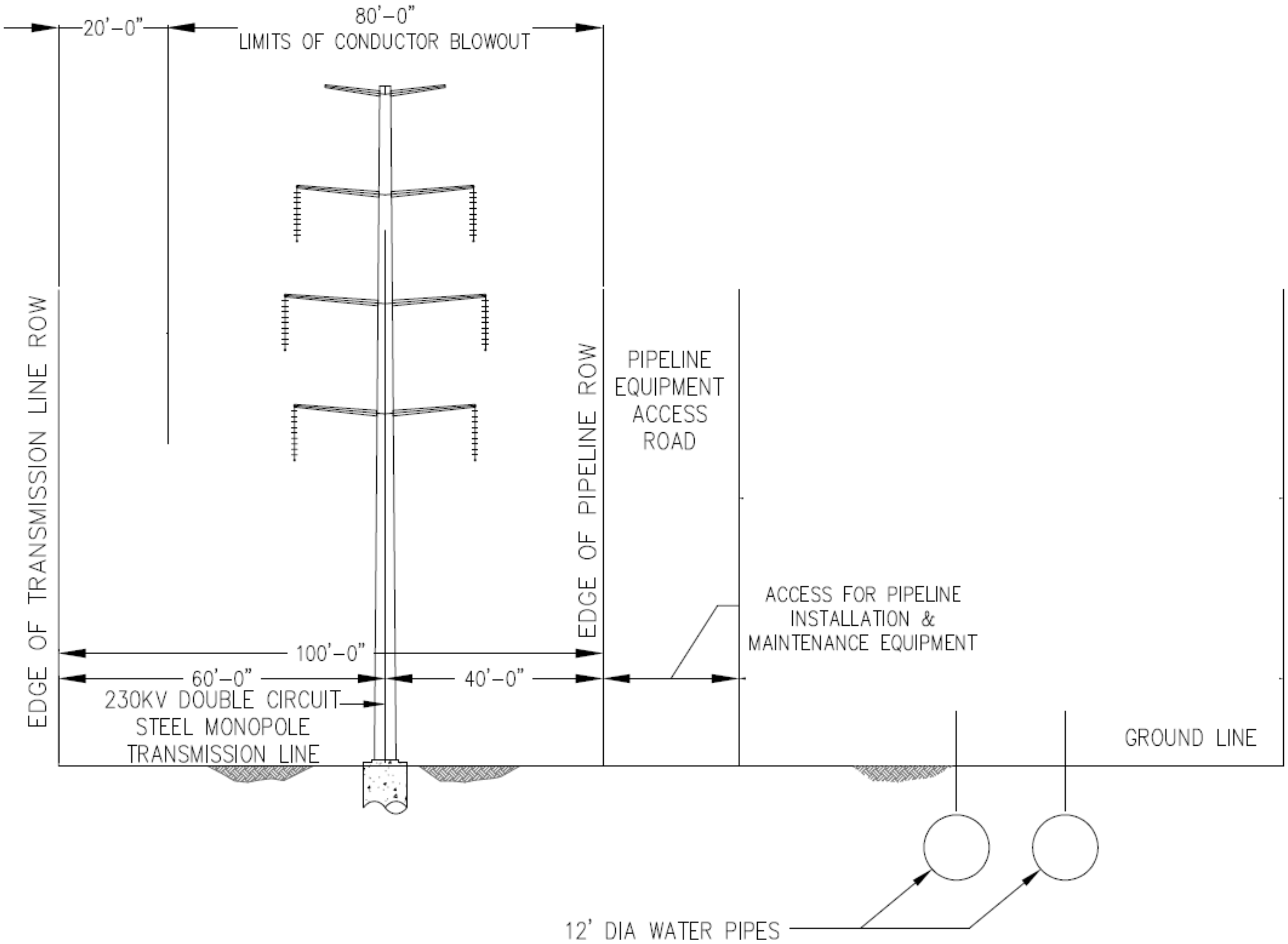


FIGURE 2: FUNKS-TRR INTERCONNECT IN PIPELINE EASEMENT

3. Description of Facilities

This section describes the proposed facilities where applicable. In most instances, the descriptions in this section are more clearly understand by looking at the drawings associated with this deliverable (a separate file).

3.1 Red Bluff Diversion

The Red Bluff Diversion is an existing facility that will require minimal improvements because provisions were made during original design and construction to add the two required pumps.

3.2 GCID Canal Improvements

Following are general descriptions of two improvements required for GCID facilities. These two description were taken from a 2017 Public Draft EIR/EIS for the Sites Project. Jacobs anticipates receiving more detail from GCID that will better help in defining the facility improvements required.

3.2.1 Main Canal Head Gate

The existing head gate structure will be left in place to continue to serve as a bridge between County Road 203 and County Road 205. The existing head gate structure will continue to operate during construction of the new head gate structure, and diversion activities will continue throughout construction. The existing head gate will not be adequate for proposed winter operation during high river flows because of the large head-drop (decrease of water elevation) across the structure during high river levels. A new head gate structure will be constructed upstream of the existing structure. The new head gate structure will include three automated gates (two vertical roller gates and one radial gate). The water level and flow control functions will involve operating conditions that would result in water surface drops across the head gate of between 3 and 15 feet, which will require a set of energy-dissipater blocks immediately below the gates to slow down and stabilize the water discharging under each gate. The canal reach immediately downstream of the new head gate structure will be lined with concrete for approximately 200 feet to prevent erosion resulting from the turbulent flow conditions.

3.2.2 Railroad Siphon

The Union Pacific Railroad siphon at Mile 26.6, near Willows, does not meet design and operation criteria for the Sites Project; the siphon will need to be replaced. The existing railroad siphon structure was built in the early 1900s and includes two 6-foot-diameter barrels and five 7.25-foot by 6-foot barrels. At maximum existing flows of approximately 2,000 cfs, the head loss across the railroad siphon, resulting from high flow velocity and poor entrance and exit transitions, reduces upstream canal freeboard to marginal conditions. The structure's age, hydraulic capacity restrictions, and use as a major transportation link lead to the recommendation for its replacement. The new structure will consist of three prefabricated box culverts. Typical future maximum velocity will be approximately 4 feet per second (fps), with 0.2 foot head loss.

The proposed replacement of the railroad siphon would require coordination and planning with railroad operators. Construction restrictions may exist regarding minimizing interference with regular railroad operations. To the extent possible, replacement of the railroad siphon would take place during periods of lowest train traffic, and railroad shutdown time would be minimized.

3.3 Funks Area

3.3.1 Pumping Generating Plants

The pumping generating plants will consist of the pumping plant, hydro-generating turbine(s), and the energy-dissipation structure.

Pumping Station

The pumping station will consist of 13 pumps in a single row, with 6 pumps feeding into one 12-foot-diameter pipe, 6 pumps feeding into the other pipe, and the final pump as a standby pump that feeds into either pipe. The pumps were sized in consultation with a pump manufacturer. Because of the large head fluctuation in the Sites Reservoir levels and resulting pumping heads, all pumps are currently anticipated to have a variable speed drive to adjust to the variable pumping heads, while staying within the pump operating range and efficiency. A technical memorandum was prepared to provide comparison of using adjustable speed drives versus only constant speed drives for this pumping plant. This TM can be found in Appendix C.

Generating Turbines

The hydro turbines at Funks have been initially designed in consultation with a manufacturer. The manufacturer determined that the wide range of generating head, in relation to the overall design head, prevents selection of a single turbine that covers the full range. The conceptual design is two turbines of a similar size, but with different speeds to use different design heads to cover the full range, with at least one unit and an overlapping unit, to have two functioning units in the middle head range.

The two generating units will be a 20 MW turbine (1,000 cfs at 255-foot head) and a 14.5 MW turbine (1,000 cfs at 190-foot head). The turbines are anticipated to be vertical Francis style. The draft tube discharge will need to be submerged, so the turbines will be in an underground structure with a roof.

Energy Dissipation

The energy-dissipation structure has been initially designed in consultation with a manufacturer. The manufacturer determined that a single 60-inch-diameter, fixed-cone valve will provide approximately 1,000 cfs discharge over the full range of head differential resulting from fluctuating reservoir levels. There will be a 60-inch-diameter, fixed-cone valve on each of the two 12-foot-diameter pipes, for a total of two 60-inch-diameter, fixed-cone valves and a total flow of 2,000 cfs.

Electrical

An electrical building will house most of the electrical equipment to protect it from the elements. The building size is about 50 feet wide by 280 feet in length. This building will be placed on top of the pumping unit intakes to save space and be close to the pumping units.

3.3.2 Reservoir

Improvements to the Funks Reservoir are anticipated to include removal of sediment build-up, as discussed previously. Regrading of the reservoir bottom near the proposed Funks PGP will also be necessary to provide a clear path of water to and from the PGP.

3.3.3 Pipeline

Preliminary hydraulic analysis indicates that two 12-foot-ID pipelines will be used to convey water between Funks and Sites Reservoir in both directions. The total length of the pipeline alignment is about 1 mile.

The two pipelines will connect two 23-foot-diameter inlet/outlet tunnels to the Site Reservoir, using large-diameter piping manifold. The design proposes to have the two Funks pipelines connect to one of the inlet/outlet tunnels.

3.4 TRR Area

3.4.1 Pumping Generating Plants

The pumping generating plants will consist of the pumping plant, hydro-generating turbines, and the energy-dissipation structure.

Pumping Station

The pumping generating plants will consist of the pumping station, hydro-generating turbine(s), and the energy-dissipation structure. The pumping station will consist of 13 pumps in a single row, with 6 pumps feeding into one 12-foot-diameter pipe, 6 pumps feeding into the other pipe, and the final pump as a standby pump that feeds into either pipe. The pumps were sized in consultation with a pump manufacturer. Because of the large head fluctuation in the Sites Reservoir levels and resulting pumping heads, all pumps are currently anticipated to have a variable-frequency drive to adjust to the variable pumping heads while staying within the pump operating range and efficiency. A technical memorandum was prepared to provide comparison of using adjustable speed drives versus only constant speed drives for this pumping plant. This TM can be found in Appendix C.

Generating Turbines

The hydro turbines at TRR have been initially designed in consultation with a manufacturer. The manufacturer determined that a single 11.5 MW turbine (500 cfs at 290-foot head) on each 12-foot-ID pipe will work under the variable head conditions. The turbines are anticipated to be vertical Francis style. The draft tube discharge will need to be submerged, so the turbines will be in an underground structure with a roof.

Energy Dissipation

The energy-dissipation structure has been initially designed in consultation with a manufacturer. The manufacturer determined that a single 36-inch-diameter, fixed-cone valve will provide approximately 500 cfs discharge over the full range of head differential, resulting from fluctuating reservoir levels. There will be a 36-inch-diameter, fixed-cone valve on each of the two 12-foot-ID pipes, for a total of two 36-inch-diameter, fixed-cone valves and a total flow of 1,000 cfs.

Electrical

An electrical building will house most of the electrical equipment to protect it from the elements. The building size is about 50 feet wide by 280 feet in length. This building will be placed on top of the pumping unit intakes to save space and be close to the pumping units.

3.4.2 Reservoir

This is an entirely new reservoir that is currently sited adjacent to the GCID canal just north of Funks Creek. The reservoir volume is anticipated to be about 600 acre-feet to accommodate about 4 hours of an 1,800 cfs canal flow from GCID if there is a shutdown and a place needed to store the water.

3.4.3 Pipeline

Preliminary hydraulic analysis indicates that two 12-foot-ID pipelines will be used to convey water between TRR and Sites Reservoir, in both directions. The total length of the pipeline alignment is about 4.5 miles.

The two pipelines will connect two 23-foot-diameter inlet/outlet tunnels to the Site Reservoir, using large-diameter piping manifold. It is proposed to have the two Funks pipelines connect to the other tunnel that is not connected to the Funks pipelines.

3.5 Administration/Operation and Maintenance/Storage Buildings

To be detailed in future design phases.

3.6 Dunnigan Pipeline

The Dunnigan Pipeline connects to the intake structure on the Tehama Colusa Canal. The Dunnigan Pipeline downstream termination point has two options. Option 1 goes from the TCC to the Colusa Basin Drain. Option 2 goes from TCC to the Sacramento River. Either of these pipelines will flow at 1,000 cfs and be based on gravity head from the TCC. Once the pipeline leaves the intake structure, it heads east, crossing and then paralleling Bird Creek. Soon after reaching Bird Creek, the pipeline will be tunneled under Interstate 5 and then Highway 99 and a railroad. The pipeline continues to head east along Bird Creek to the Colusa Basin Drain. The 9’ pipeline would end at this drain per and outlet structure. If the pipeline is to continue, it would cross the drain, head north, and then head east along rice fields to the Sacramento River. The pipeline will cross over Highway 45 and a levee and into the river through an outfall structure.

The proposed length of option 1 alignment is about 4 miles. Preliminary calculations show a 9-foot (108-inch) ID, with 2 tunneled crossings (I-5 and 99W/RR) that require 10.5-foot (126-inch) casings. The total length of pipeline is 20,000 feet, with 300-foot and 250-foot tunneled crossings. Two 60-inch-diameter, fixed-cone valves will be placed at the discharge to dissipate energy and adjust the flow.

The proposed length of the option 2 alignment is about 10 miles. Preliminary calculations show a 10.5-foot (126-inch) ID, with 3 tunneled crossings (I-5, 99W/RR, and CBD) that require 12-foot (144-inch) casings. The total length of pipeline is 51,600 feet, with 300-, 250-, and 250-foot tunneled crossings. Two 60-inch-diameter, fixed-cone valves will be placed at the discharge to dissipate energy and adjust the flow.

Both of these alignments are shown in the drawings, under separate cover.

An alternative pipeline alignment known as the Harrington Pipeline (located about 8 miles north of Dunnigan Pipeline) from the TCC to the Colusa Basin Drain was studied and summarized in a technical memorandum located in Appendix D. The TM recommends keeping the Dunnigan Pipeline to convey water back to the Sacramento River. The TM was presented to the Sites Ad Hoc Operations and Engineering Work Group on August 11, 2020 and they decided that the Harrington Pipeline was not a viable alternative.

The structures associated with the Dunnigan Pipeline include the following intake and discharge structures.

- TCC Intake Structure
- Colusa Basin Drain Outlet Structure
- Sacramento River Outfall Structure

3.6.1 TCC Intake Structure

The intake structure will be used to divert water from the existing concrete lined TCC into the Dunnigan Pipeline. The intake structure will be a concrete structure that supports the control gates and associated gate operators. A concrete bridge deck will provide vehicle access across the top of the structure. Stop log slots will be provided upstream and downstream to isolate the control gates for maintenance.

3.6.2 Colusa Basin Drain Outlet Structure

The outlet structure includes a stilling basin and two fixed-cone valves to dissipate energy before water enters the existing Colusa Basin Drain canal. The geometry of the stilling basin is sized in accordance with the Bureau of Reclamation’s Hydraulic Design of Stilling Basins and Energy Dissipators.

3.6.3 Sacramento River Outlet Structure

The outlet structure into the Sacramento River will include an exclusion barrier, sized per NOAA Fisheries design guidelines, to prevent the passage of anadromous fish. The exclusion barrier will consist of a combination velocity and vertical drop barrier, including a weir and concrete apron to prevent upstream passage. The minimum weir height relative to the apron below is 3.5 feet, with a maximum weir crest of 2 feet. The minimum apron slope in the downstream direction is 1:16, with a maximum flow depth of 0.5 foot. The end of the apron must be at least 1 foot above the high water surface elevation of the Sacramento River. Refinement of this structure design will be completed in the next design phase.

3.7 Electrical Supply

3.7.1 Point of Interconnection

The point of interconnection (POI) for the project will require that an Application for Interconnection Request be submitted and processed under the CALISO Interconnection Process. The location of the POI to either the WAPA or PG&E 230-kV transmission lines will depend on the results of a system impact study (SIS), which will be required to be performed by the Transmission Operators (TOs), in conjunction with the independent system operator of the transmission system.

The interconnection application process includes that the project enter into a SIS agreement, which requires the project to compensate the TO for its actual costs to undertake the SIS. The TO will include in the agreement a non-binding estimate of the cost and a timeframe for completing the SIS. The SIS report will state the results of the power flow, short circuit, and stability analyses, and will provide the requirements or potential impediments to the requested POI, including a preliminary indication of the cost and length of time necessary to correct any problems identified in the SIS. The SIS report will also provide a preliminary list of facilities required to be upgraded to accommodate the supply of power to the project.

The application process then entails the TO performing a facilities study (FS). The Project will enter into a FS agreement, which requires the project to compensate the TO for its actual costs to perform the FS. The TO will include in the agreement a non-binding estimate of the cost and timeframe for completing the FS. Upon completion of the FS, the TO will provide the FS report. The report will specify the estimated cost of the equipment, engineering, procurement, and construction work needed to implement the conclusions of the SIS. The FS report will also identify the electrical switching configuration of the connection equipment, including: the transformer, switchgear, meters, and other station equipment. The report shall include a +/- 20 percent cost estimate of facilities necessary for the interconnection, and an estimate of the time required to complete the construction and installation of such facilities.

The application process then entails the project entering into the Interconnection Agreement, (IA) with the TO. The IA will require that the actual costs associated with the equipment, environmental, engineering, procurement, construction, and any other work needed to accomplish the interconnection be payable by the project. The IA will specify the interconnection and network facilities that will be required to interconnect the project.

The California Independent System Operator interconnection procedures place applications into groups, known as clusters, for projects that are interconnecting in the same area to be studied together.

This process will begin in September 2020 when the Authority provides funding to PGE and WAPA.

3.7.2 Transmission Lines

A POI to a high-voltage electric transmission line will be required for the project. Interconnecting to the transmission system is necessary to provide for the supply of power to operate the large-horsepower pumps at both the Funks Reservoir, Funks PGP, and the TRR Pumping/Generation Plant. In addition, the interconnection to the transmission system will allow PGP and TRR to send energy produced to the transmission system during the periods when they are using their turbines/generators.

Several existing high-voltage transmission lines are in the vicinity of the project; all of these run north to south. These transmission lines include two 230 kV lines owned and operated by WAPA, and four 230 kV lines owned and operated by PG&E. WAPA and PG&E are defined as the TO and the Transmission Operator of their respective high-voltage transmission lines. Each of these lines is a potential POI source for the project. The Transmission Agency of Northern California, (TANC), owns a 500-kV, high-voltage transmission line that runs parallels to the WAPA lines; however, this transmission line is not considered to be a potential POI for the project.

See attached Figures 3 and 4 for schematic sketches showing the WAPA and PG&E alternative POI arrangements, and the required transmission line lengths to the proposed Funks and TRR substations. Refer to Section 3.6.1 for additional details. Under either alternative POI, the power will be delivered to the project via looped (two circuits) 230 kV to the supply power for the pumps. The looped circuits would also receive power from the Funks and TRR plants when their turbine/generators are operating. The looped circuits are typically installed on double-circuit steel monopole structures (poles), as shown in Figure 1. The poles would be approximately 100 to 150 feet high and supported atop reinforced concrete foundations that are augured and designed in accordance with the results of the geotechnical investigation. The conductor size will be designed to match or be larger than the existing conductor size of the transmission line, which will be interconnected via the loop. One or two fiberoptic cables can be used as shield wire, the size of which will be determined by fault current requirements and TO telecommunication requirements.

In addition to the loop POI design, there will be two additional 230 kV transmission line radial taps installed between the Funks and TRR substations. The transmission line structures for these lines will be double-circuit, steel monopole structures, as shown in Figure 2. The poles would be approximately 100 to 150 feet high and supported atop reinforced concrete foundations that are designed in accordance with the results of the geotechnical investigation. The conductor size is estimated to be 795 kcmil aluminum conductor steel reinforced (ACSR). In some sections of the transmission line; the double-circuit, monopole tap lines may share a common ROW with the double-circuit, monopole looped circuits (see Section 2.9).

The configuration of the transmission lines will depend on the selected POI, which is described in the following subsections.

WAPA POI Option

This option proposes to loop the existing WAPA 230 kV Keswick-O'Banion transmission line into and out of the Funks substation, as shown schematically on Figure 3. The length of the looped, double-circuit, steel monopole line will be approximately 1 mile, in a generally westerly direction to Funks. Two new, three-pole, single-circuit structures will be cut into the existing transmission line; the existing wires between these two structures will be removed. The new conductors in the first spans toward Funks will be installed low on the poles to achieve the proper clearances as they cross under the existing TANC 500 kV transmission line to two, new, double-pole, single-circuit (or one, new, double-pole, double-circuit) steel H-Frame structures. Minimum phase-to-phase and phase-to-ground clearances will be in accordance with WAPA and TANC standards, and the state of California Public Utility Commission Rules for Overhead Electric Line Construction. Conductors will match or exceed the conductor size of the existing WAPA line.

Two, new, 230 kV, radial lines will also be constructed between the Funks and TRR substations, on double-circuit, steel, monopole structures, for a length of approximately 3.9 miles. H-Frame construction will be used at the crossings below the TANC 500 kV Line, the two WAPA 230 kV lines, and the four PG&E lines.

Although the structures between Funks and TRR will be designed to accommodate two 230 kV circuits, it is possible that only one circuit will be initially installed. The conductor will be 795 kcmil ACSR.

PG&E POI Option

This option proposes to loop the existing PG&E 230 kV Colusa-Vaca-Dixon #3 transmission line into and out of the TRR substation, as shown schematically on Figure 4. The length of the looped, double-circuit, steel monopole line will be approximately 1.7 miles, in a generally easterly direction to TRR. Two, new, monopole-pole, single-circuit structures will be cut into the existing transmission line; and the existing wires between these two structures will be removed. Conductors will match or exceed the conductor size of the existing PG&E line.

Two new 230 kV radial lines will also be constructed between the TRR and Funks substations, on double-circuit, steel monopole structures, for a length of approximately 3.9 miles. Single- or double-circuit, steel, H-Frame pole construction will be used at the crossings below the existing TANC 500 kV Line, the two WAPA 230 kV lines, and the four PG&E lines, to achieve proper minimum phase-to-phase and phase-to-ground clearances, in accordance with WAPA and TANC standards, and the State of California Public Utility Commission Rules for Overhead Electric Line Construction.

Although the structures between Funks and TRR will be designed to accommodate two 230 kV circuits, it is possible that only one circuit will be initially installed. The conductor will be 795 kcmil ACSR.

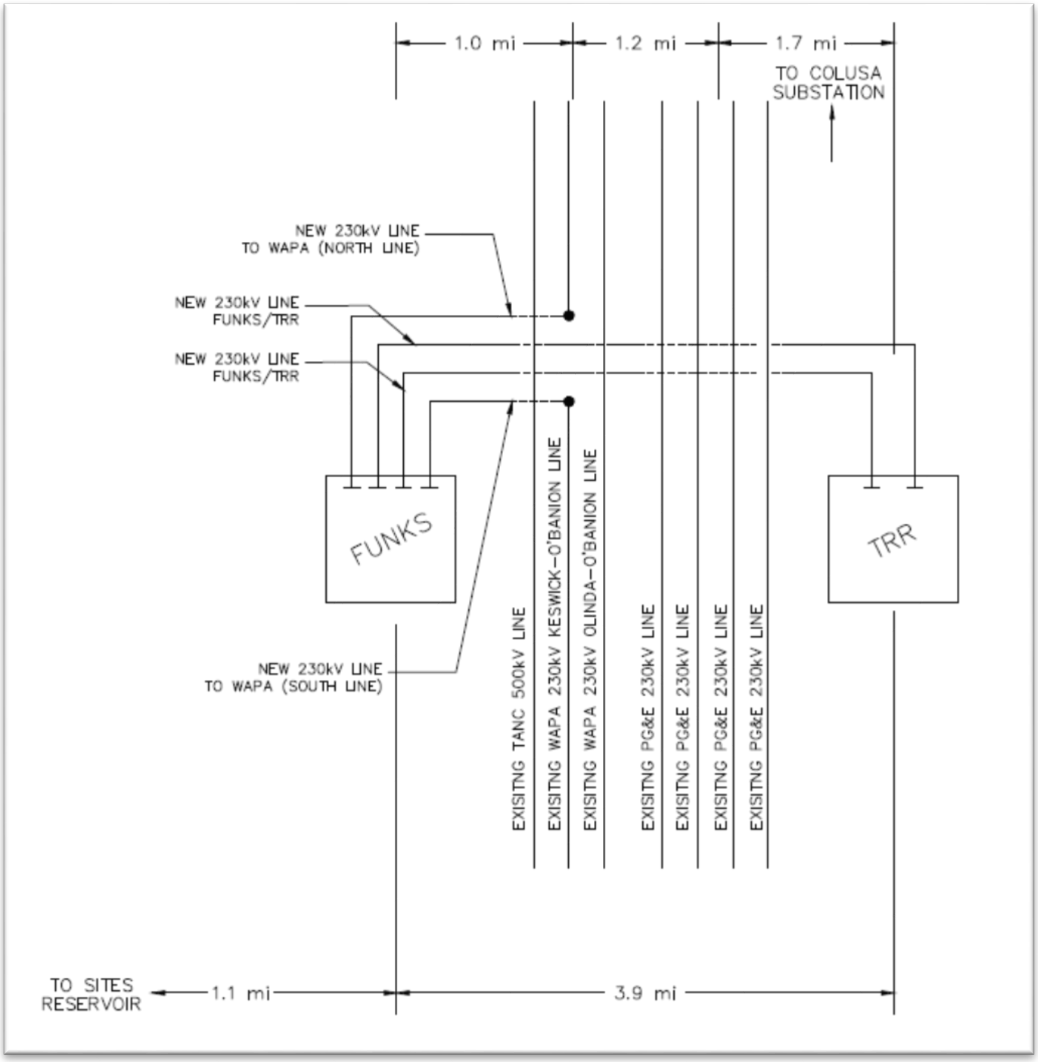


FIGURE 3: SCHEMATIC OF WAPA POI OPTION

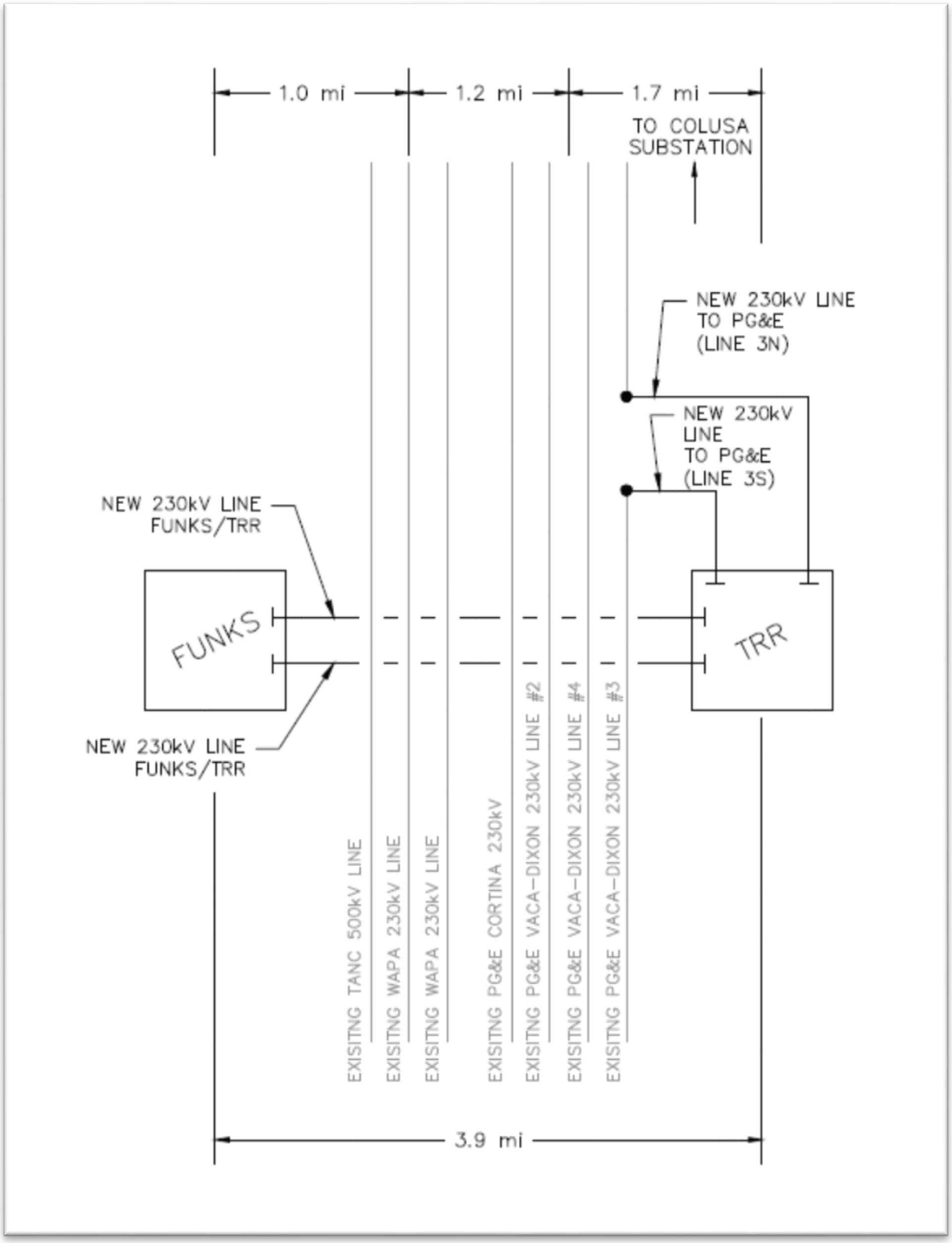


FIGURE 4: SCHEMATIC OF PG&E POI OPTION

3.7.3 Substations

Each pumping/hydroelectric generator substation at TRR and Funks Reservoir will have a new 230 kV to 13.8 kV substation. The substations will service a net pumping energy demand, estimated at 80 MVA at Funks and 90 MVA at the TRR site, totaling 170 MVA of demand load.

In terms of generation, estimates indicate that Funks Reservoir will have a net generating capacity of 55.0 MVA and TRR will have 31 MVA. The project's total net generating capacity to the grid is estimated to be 86 MVA.

The project estimated pumping energy requirements and power generation are summarized as shown in Tables 10 and 11.

TABLE 10: PROJECT PUMPING SUMMARY

Site	Net Pumping Power (MW)	Other Auxiliary Loads (MW)	Transformer and T Line Losses (MW)	Total Pumping Power (MW)	Total Pumping Power @ 0.85 Power Factor (PF) (MVA)
Funks	67.1	1	0.1	68.2	80.2
TRR	75.4	1	0.1	76.5	90.0
Total	**142.4**			**144.7**	**170.2**

TABLE 11: PROJECT GENERATING SUMMARY

Site	Net Generating Power (MW)	Other Auxiliary Loads (MW)	Transformer and T Line Losses (MW)	Total Power Generation (MW)	Total Power Generation @ 0.85 PF (MVA)
Funks	48.1	1	0.1	47.0	55.3
TRR	27.4	1	0.1	26.3	31.0
Total	**75.5**			**73.3**	**86.2**

The substations will be designed to for the total pumping power requirements (import) or total generation requirements (export).

3.8 Site Civil and Roadway Improvements

3.8.1 Site Civil

Funks Area

The proposed Funks PGP site is located in Colusa County, on the northwestern side of the existing Funks Reservoir. Access is provided to both the southern and northern ends of the site, as described in the roadway improvement section.

Asphalt concrete-paved, onsite, vehicular access will be provided between the proposed PGP and substation, with facility spacing to accommodate an operational crane with outriggers extended. Asphalt concrete-paved, onsite parking and vehicular access will also be provided at the two buildings on site, the maintenance and storage building, and the administrative and operations building. Additional gravel parking will be provided near the pumping generating plant.

The proposed substation and overall site will be enclosed by a security fence (6-foot-tall, chain-link fabric with 1-foot of three-strand barbed wire on top) with 30-foot-wide, double-swing access gates on the southern and northwestern sides. The switchyard will have internal gates as well as one external gate to accommodate access requirements.

Site drainage will be conveyed offsite directly into the Funks Reservoir via gentle swales or overland flow. Offsite stormwater runoff will be collected on the western side of the site in a ditch and conveyed around the site and into the Funks Reservoir.

The proposed Funks PGP site is in a FEMA Area of Minimal Flood Hazard, Zone X.

TRR Area

The proposed TRR is located in Colusa County, north of the GCID Canal and just West of McDermott Road. The site will be accessed via a maximum 30-foot-wide asphalt concrete, paved road from McDermott Road. Paved parking will be provided near the pumping generating plant.

Vehicular access will be provided inside the proposed switchyard, as well as to the pumping generating plant and inlet and outlet structures, with facility spacing to accommodate an operational crane with outriggers extended. The proposed switchyard and overall site will be fenced with 7-foot chain-link fence and access gates on the southern and eastern sides.

Site drainage will be conveyed offsite to the existing GCID Canal or directly into the TRR via gentle swales or overland flow.

The proposed TRR site is located within a designated Federal Emergency Management Agency (FEMA) Special Flood Hazard Areas, Zone A, Without Based Flood Elevation. A base flood elevation will need to be determined prior to project approval.

3.8.2 Roadway Improvements

Funks Area

The Funks PGP site will be accessed via a 30-foot-wide asphalt, concrete-paved road from Maxwell Sites road to the south. Existing gravel and roads will be improved to be 30 feet wide with asphalt concrete surfacing for the southern access; these will be relocated through the PGP site. A 30-foot-wide gravel bypass road may be provided to the west of the site. On the northern side of the site, the existing dirt road will be improved to be a 30-foot-wide gravel road that will follow the existing road alignment until it reaches the TRR pipeline. At that location, a new 30-foot-wide access road will be built alongside the Funks and TRR pipelines to the connection with the Sites tunnels.

Most of the road is within a FEMA Area of Minimal Flood Hazard, Zone X, but a portion of the existing gravel road to the northwest of the PGP site and adjacent to the existing creek is located within a FEMA Special Hazard Flood Area without Base Flood Elevation, Zone A. This portion may need to be raised if all-season access from that direction will be required.

TRR Area

The TRR and site improvements will be accessed via the existing McDermott Road. No roadway improvements are anticipated on existing roads.

3.9 Summary of Other Analysis

Additional analysis was conducted for the Sites project as follows and found in Appendices to this report:

- Utilizing proposed pipeline and other facilities to convey emergency drawdown of Sites Reservoir (Appendix E)
- Option to provide 10 cfs of base flow to the head of Funks Creek (Appendix F)
- Evaluation of potential hydroelectric revenue (Appendix G)

4. Acronyms and Abbreviations

ACSR	aluminum conductor steel reinforced
AISC	American Institute of Steel Construction
ANSI	American National Standards Institute
Authority	Sites Project Authority
CAISO	California Independent System Operator
CBC	California Building Code
cfs	cubic foot per second
CMU	concrete masonry unit
FEMA	Federal Emergency Management Agency
FS	facilities study
GCID	Glenn-Colusa Irrigation District
IA	Interconnection Agreement
ID	inner diameter
IEEE	Institution of Electrical and Electronics Engineers, Inc.
JPA	Joint Powers Authority
ksi	kilopounds per square inch
kV	kilovolt
MPR	motor protection relay
MVA	millivolt-ampere
MW	megawatt
PE	permanent easement
PF	power factor
PG&E	Pacific Gas & Electric
PGP	Pumping Generating Plant
POI	point of interconnection
Reclamation	U.S. Bureau of Reclamation
ROW	right-of-way
rpm	rotation per minute
SIS	system impact study
TANC	Transmission Agency of Northern California
TCC	Tehama-Colusa Canal
TCE	temporary construction easement
TO	Transmission Operator
TRR	Terminal Regulating Reservoir

V	volt
WAPA	Western Area Power Administration
WSE	Water Surface Elevation
WSP	welded steel pipe

Appendix A
Pipeline Design Criteria

TRANSMISSION PIPELINES DESIGN CRITERIA

Subject	Criteria	Comments/Reason
Survey and Mapping		
Horizontal Drawing Scale	1 inch = 200 feet (ft)	
Vertical Drawing Scale	1 inch = 20 ft	
Photo Plans	0.2-ft pixel resolution at 50 scale	
Contours	2 ft minor 10 ft major	
Pipeline Sizing/Hydraulics		
Maximum Velocity (normal)	10 ft per second	
Maximum Velocity (emergency)	35 ft per second	
Largest Standard Pipe	12-foot inside diameter	This is largest common pipe to be transported as fabricated. Larger pieces are considered special and excluded from this requirement.
Head Loss	Dynamic head determined using Hazen-Williams value of: C_{HW} = 120 (high friction case) C_{HW} = 145 (low friction case)	
Pipe Inside Diameter	Varies	
Pressure Considerations	Working Pressure = varies Design Pressure = varies Applies for design of the pipeline fittings, specials, and appurtenances Maximum Surge = not to exceed 1.33 times the design pressure Test Pressure = not to exceed 1.25 times the design pressure	
Drawings	Hydraulic profile(s) will be included in the drawings.	
Horizontal Alignment		
References	American Water Works Association (AWWA). date. *Concrete Pressure Pipe (M9) 3rd Edition*. American Water Works Association (AWWA). Date. *Steel Pipe – A Guide for Design and Installation (M11) 5th Edition*.	
Drawing Layout	Stationing will be shown on the drawings. Northing/Easting will be shown at horizontal points of intersection (HPIs).	
	Locations of appurtenances shown on the drawings.	
Horizontal Bends	Combine horizontal and vertical angle points wherever possible and call out as "Combined Bend" in the plan view. Bends less than or equal to 3.75° or ¾" maximum pullout can be made with standard joint deflection in the field at the joints that fall on either side of the HPI.	

TRANSMISSION PIPELINES DESIGN CRITERIA

Subject	Criteria	Comments/Reason
	Larger bends than standard joint deflections can be made using a beveled end joint on either side of the location of the HPI. Maximum bevel at any joint not to exceed 5°. Bends greater than 5° are made using fabricated elbows. Horizontal curves are identified on the drawings.	
Vertical Alignment		
References	American Water Works Association (AWWA). date. *Concrete Pressure Pipe (M9) 3rd Edition*. American Water Works Association (AWWA). Date. *Steel Pipe – A Guide for Design and Installation (M11) 5th Edition*.	
Drawing Layout	Minimal slopes shown in ft/ft between vertical points of intersection (VPIs). VPI elevations control actual slope. Minimum slope = 0.001 ft/ft or 0.1%. Avoid flat (0% slope) reaches. Surface slopes greater than 10% may require trench cutoff walls. Minimum cover over pipe is 6 ft unless otherwise approved. Pipe stationing and centerline elevations will be shown at VPIs.	
Utility Crossings	Maintain a minimum clearance (as coordinated with utility owner) between utilities crossing the water pipeline to be identified in future submittals.	
Vertical Bends and Curves	Combine horizontal and vertical angle points wherever possible and call out as combined point of intersection in the plan view. Bends less than or equal to 5° can be made with standard joint deflection in the field at the joints that fall on either side of the HPI.	
	Larger bends than standard joint deflections can be made using a beveled end joint on either side of the location of the VPI. Maximum bevel at any joint not to exceed 5°. Bends greater than 5° are made using fabricated elbows.	
Pipe Material		
References	For steel coil, refer to American Society for Testing and Materials (ASTM) A1018/A1018M for specification of structural steel. For steel plate, refer to ASTM A516/A516M for specification of structural steel.	

TRANSMISSION PIPELINES DESIGN CRITERIA

Subject	Criteria	Comments/Reason
	American Society of Civil Engineers. Date. *Steel Penstock Design Manual of Practice (MOP) 79*. American Water Works Association (AWWA). Date. C200-05.	
Material	In accordance with AWWA C200: Coils: ASTM A1018/A1018M Structural Steel Grade 36, Type 2 Modified 1. Manganese: 1.5% maximum 2. Aluminum: 0.02% minimum 3. Phosphorus: 0.025% maximum 4. Sulphur: 0.015% maximum Plate: ASTM A516/A516M Grade 65 Yield strength: 42 kilopounds per square inch (ksi), minimum Tensile strength: 63 ksi, minimum Maximum measured yield strength 85% of measured tensile strength Min Elongation: 21% in 2-in gauge length Carbon equivalent (CE) <0.45	
	Fully killed, fine-grained practice, continuous cast Toughness: 25 ft-lbs at 30°F for pipe wall thickness 7/16 in or above, per Charpy Test	
Minimum Wall Thickness	See Pipe Structural Section.	
Standard Barrel Length	40 ft maximum.	
Linings and Coatings	To be determined.	
Pipe Structural Section		
References	AWWA M11 5th Edition. AWWA C200-05. ASCE MOP No. 79 – Steel Penstocks.	
Procedure (see brief procedures below)	1. Minimum handling 2. Hoop stress (Barlow) 3. External loading (Spangler, soil buckling, highway, railroad, construction, and vacuum) 4. Longitudinal stress 5. Biaxial stress 6. Collapse (Stewart's) 7. Mitered bend wall thickness	
Internal Pressure	Values determined using pump station hydraulic analysis and system surge analysis.	
Factory Test Pressure	In accordance with AWWA C200 5.3.	
(1) Minimum Handling	thickness = D/240	
(2) Hoop Stress	Barlow formula	

TRANSMISSION PIPELINES DESIGN CRITERIA

Subject	Criteria	Comments/Reason
	P=2tS/D, where S = the allowable hoop stress as described in the right-hand column. AWWA M11 §4.1.	
(3) External Loading Spangler Soil Buckling	Deadload (DL): based on actual unit weight of backfill material. AWWA M11 §6.4. Live load (LL): based on HS-20 highway loads and E-80 railroad loads, per AWWA M11 Chapter 6. Construction loading per AWWA M11 Chapter 6 using extreme external LL from a large loader. AWWA M11 §6.8. Pipe is required to withstand full vacuum in the soil buckling analysis. External load combinations applied for determining soil buckling are as follows: 1. DL + LL (Construction) 2. DL + LL (HS-20) 3. DL + LL (E-80) 4. DL + PV (Internal Vacuum Pressure)	
(4) Longitudinal Stresses	Evaluate for the following two conditions: 1) warm weather temperature installation; 2) bulkhead stresses. Pipe wall and joint selection must be consistent with the condition that proves to be the limiting case. For longitudinal joint stresses, joint efficiencies are: e = 0.45 (single-welded lap) e = 0.55 (double-welded lap) e = 0.70 (butt weld, no radiographic test [RT]) e = 0.85 (butt weld, partial RT) e = 1.00 (butt weld, full RT) AWWA C206 for guidance on welding procedures.	
Warm Weather Installation	Warm weather installation allowable working longitudinal stress is the lesser of: 1. Tensile strength divided by 2.4, or 2. Yield strength divided by 1.5, or 3. The lesser of the above multiplied by the joint efficiency to obtain the allowable working longitudinal stress The allowable working longitudinal stress has to be greater than the longitudinal stress as calculated per AWWA M11, Chapter 8. The temperature stress is divided by the thermal load stress multiplier to calculate the longitudinal stress.	

TRANSMISSION PIPELINES DESIGN CRITERIA

Subject	Criteria	Comments/Reason
Bulkhead Stresses	Allowable working longitudinal stress is the lesser of tensile strength divided by 2.4, yield strength divided by 1.5, or allowable hoop stress multiplied by the joint efficiency. The allowable working longitudinal stress has to be greater than the bulkhead stress, which is 50% of the hoop stress.	
(5) Biaxial Stress Cold Weather Installation	Cold weather installation allowable working biaxial stress is the lesser of yield strength divided by 1.5, or tensile strength, divided by 2.4. The lesser of the above two values is multiplied by the joint efficiency of the spiral weld. The allowable working biaxial stress has to be greater than the Von Mises Biaxial Stress as directed by ASCE Manual 79, Chapter 3. The temperature stress is divided by the thermal load stress multiplier to calculate the longitudinal stress and the biaxial stress. Spiral Weld Joint Efficiency = 0.85	
(6) Collapse	Stewart's Formula: Collapsing pressure determined using AWWA M11 §4.4. Stewart's Formula only applies in locations listed in the column to the right and if negative pressures are calculated from the surge analysis at those locations.	
(7) Mitered Bends	Thickness of mitered bends determined using procedure in AWWA M11 Chapter 9.	
Soil Loads	In accordance with depth per unit soil weights from geotechnical reports.	
Thrust Forces for Bends and Valves	AWWA M11 Chapter 13.	
Pipe Joints		
References	AWWA C200-05. AWWA C207. AWWA C208. AWWA C219. American Society of Mechanical Engineers (ASME) Boiler and Pressure Vessel Code (BPVC) Section VIII, Division 1, Part 2. ASME 16.5 and ASME 16.47.	
Joint Types	The standard field joint is single-welded lap. Other welded joints include double-welded lap, butt joint welds, and butt strap welds. Flanged joints or restrained flexible joints may be used at connections as described below. Strength of welded joint efficiency as stated in AWW M11.	

TRANSMISSION PIPELINES DESIGN CRITERIA

Subject	Criteria	Comments/Reason
	Butt strap welds have the same joint efficiency as a double-welded lap joints.	
Special Joints	Flexible-type pipe coupling with harness restraints per modified AWWA M11 and AWWA C219. Flexible-type couplings configured as a dismantling joint in accordance with AWWA C219. Provide with suitable thrust restraining capability. Insulating joints depend on soil corrosivity. Generally, provide insulating joints at: • Connections to pump stations • Between pipe in low-resistivity soils and pipe in high-resistivity soils • Upon entering and/or exiting vaults • Connections to electrically grounded equipment (valves) within vaults	
	Flanged joints: • Flanged joints are used at combination air-release vacuums (CARVs), blowoffs, and other locations as needed. • ASME B16.5 and ASME B16.47, class 150 or class 300, bolted flanges for pipe where appropriate Restrained flexible joints: • Restrained flexible joints are used downstream of the blowoff riser pipe and blowoff pump well.	
Thrust Restraint	Do not use unrestrained joint system unless it has been verified that no forces must be restrained at a particular joint. Use full force (PA) as basis for thrust design. Flexible couplings: anchor each side of coupling for resultant axial thrust force. Fittings: for fully restrained pipeline consider longitudinal force as described in AWW M11. Restrained Dismantling Joint: anchor each side of coupling across the joint for resultant axial thrust force.	

TRANSMISSION PIPELINES DESIGN CRITERIA

Subject	Criteria	Comments/Reason
Elbows, Tees, Wyes, Bends	Comply with AWWA C208. Minimum radius = 2.5 times pipe OD unless designed for smaller radius per AWWA C208. Elbows: • 2-piece (0° to 22.5° deflection angle) • 3-piece (22.5°to 45° deflection angle) • 4-piece (45° to 67.5° deflection angle) • 5-piece (67.5° to 90° deflection angle)	
Nozzles, Dished Heads, and Test Heads	Design in accordance with ASME BPVC, Section VIII, Division 1.	
Pipeline Testing		
References	AWWA C206. AWWA M11. ASME BPVC, Section IX. American Welding Society (AWS) D1.1 for field welds.	
Hydrotesting	AWWA M11 Chapter 12. Test pressure not to exceed 125% of the conservative design pressure. Hydrostatic testing required for the main pipeline and includes the blowoff isolation valve, CARV, and related appurtenances. Maximum filling velocity not to exceed 1.0 feet per second, with calculated based on the full area of pipe. Hydrotesting of the blowoff piping and related appurtenances downstream of the blowoff isolation valve will not be required. Pipe with welded joints and flange joints to have zero allowable leakage.	
Field Welded Joint Testing	Test in accordance with ASME BPVC Section IX for shop welds and AWS D1.1 for field welds. Field Single-Welded Lap Joints: Visual inspection of 100% of welds plus 100% full circumference liquid penetrant (PT) or magnetic particle (MT) test on single lap welds. Field Double-Welded Lap Joints and Butt Strap Joints: Visual inspection of 100% of welds plus test double-welded lap joints and butt strap joints by pressurizing welds to 40 pounds per square inch and painting welds with soap solution. In addition, perform 100% full circumference PT or MT test on double-lap welds and butt strap joints.	

TRANSMISSION PIPELINES DESIGN CRITERIA

Subject	Criteria	Comments/Reason
	Field Butt Joint Joints: Visual inspection of 100% of welds plus full circumference spot RT test on butt joint welds.	
Protective Coatings and Linings		
References	AWWA C209 AWWA C214 AWWA C216 AWWA C222.	
Pipe Coating	Tape Coating System per AWWA C217, AWWA C216, AWWA C214 and AWWA C209. Polyurethane Coating System per AWWA C222 except as modified herein: Self-priming, plural component, 100 percent solids, polyurethane, suitable for burial or immersion, and the product of one of the following approved manufacturers: a. Futura Coatings (Protec II), Hazelwood, Missouri. b. Chemline (Chemthane 2261/2265), St Louis, Missouri.	
Pipe Lining	Cement Mortar Lining per AWWA C205. Velocity from 0 to 15 feet per seconds (fps) meets requirements for cement mortar lining. Velocities greater than 15 fps are polyurethane lined per AWWA C222	
Pipe Fittings	Coat buried connections, flanges, and any other irregular shapes with petrolatum or wax tape per AWWA C217. Use filler to create a smooth regular surface before applying petrolatum or wax tape. Coat buried welded joints with heat-shrinkable sleeves per AWWA C216. Wax tape coat buried accessways, per AWWA C217.	
Pipeline Appurtenances		
References	AWWA M11, 5th Edition. ASME BPVC, Section VIII, Division 1.	
General	No above ground pipeline appurtenances shall be constructed within Jurisdictional Waters of the United States or non-jurisdictional wetland.	

TRANSMISSION PIPELINES DESIGN CRITERIA

Subject	Criteria	Comments/Reason
Manway Access	Accessways are provided every 2000 feet +/- Accessways are provided within 20 ft of vertical bends at the top of pipeline slopes that are greater than 12 percent grade. Accessways not associated with CARV vaults are buried. Accessways are combined with CARV vaults where possible to reduce number of buried access manways. Mainline accessways are minimum 30-in diameter with a blind flange top access. Accessway outlets are designed per AWWA M11.	
Pipeline Isolation Valves	There are no mainline isolation valves on the pipeline except where required for operations.	
Combination Air Valves (Air/Vacuum Release)	Design air valves for the following conditions: 1. To evacuate air during pipeline filling at a filling velocity of 1 fps. 2. To allow air to enter during pipeline draining at rates defined in the Blowoff operations sequence. 3. To meet the requirements defined by surge analysis. 4. To allow air to enter to prevent vacuum conditions during a 24-in diameter rupture. 5. Intermediate air valves, as described in AWWA M-51 to release air at locations other than highpoints will not be installed. Sizing the CARV is based on the following additional criteria: • Maximum differential pressure across the CARV during pipe draining or emergency pipeline rupture as recommended by valve manufacturer. • Maximum differential pressure across the CARV during filling as recommended by valve manufacturer. • Minimum seating pressure above CARV orifice as recommended by valve manufacturer.	

TRANSMISSION PIPELINES DESIGN CRITERIA

Subject	Criteria	Comments/Reason
Blowoff Assemblies	Design flow for blowoffs are based on the smallest of: • Resulting flow if the maximum velocity in the main pipeline during draining is 2 ft/s • Resulting flow based on the discharge drainage channel not exceeding a maximum allowable downstream discharge capacity equal to the channel's 2-year storm event • Resulting flow if the maximum velocity in the blowoff piping is 12 ft/s	
	Every low point within the system is designed with a blowoff. Minimum cover above blowoff piping is 2.5 ft. Blowoff piping minimum slope from pump well is 1%.	
Earthwork and Trench		
Site Preparation	Construction activities will not be allowed outside of designated work limits as shown on the drawings.	
Trench	Method of excavation to be determined during final design using recommendations presented in the applicable geotechnical report(s). Minimum trench width: • When controlled low strength material (CLSM) is used for pipe zone material, minimum trench width is the pipe outer diameter plus 12 in clear (horizontally) on each side (typical). • When granular fill is used for pipe zone material, minimum trench width is the pipe outer diameter plus 18 in clear (horizontally) on each side (typical).	
Groundwater	Groundwater is expected to be encountered, mostly at drainage crossings.	
Pipe Bedding Material	CLSM or a well-graded granular material to be used as bedding material. In areas where CLSM is required for pipe zone material, CLSM will be required as bedding material. The water pipeline will be supported beneath the haunches with sand bags or native material when CLSM is used. In areas requiring over excavation for trench stabilization, use a suitable foundation stabilization prior to placing bedding material.	

TRANSMISSION PIPELINES DESIGN CRITERIA

Subject	Criteria	Comments/Reason
Pipe Zone Material	The pipe zone is defined as the area from the bottom of the pipe bedding to a minimum of 12 inches above the top of pipe, including the full width of the trench. CLSM or a well-graded granular material is used as backfill within the pipe zone unless otherwise specified in applicable geotechnical report(s) or hydrologic/scour report(s). CLSM is used as backfill from the bottom of the pipe bedding to 12-inches above the crown of the pipe, including the full width of the trench, for any road crossing from right-of-way to right-of-way.	
Trench Zone	The trench zone is from the top of the pipe zone to the bottom of the specified surface restoration, including the full width of the trench. Native or imported soil, loam, or other material suitable for use as backfill and is required to meet the requirements of applicable permits. Marking tape is required to be installed in the center of the trench 1 ft above the top of the pipe at the pipe centerline.	
Minimum Depth of Pipeline	Generally minimum bury depth is 6 ft. Some locations, including drainage crossings, road crossings, and utility crossings may require an increased minimum bury depth. See Vertical Alignment criteria for minimum bury depths.	
Disposal of Excess Trench Material	Contractor to submit excavation and disposal plans per the specifications.	
Finished Grading	Restore site to pre-construction conditions; no enclosed depressions allowed and no alterations to existing drainage ways allowed. Restoration is required to meet the requirements of applicable permits.	
Revegetation Requirements	Revegetate per local jurisdictional standards, and applicable permit requirements.	
Trenchless Crossings		
References	CI/ASCE 36-01, *Standard Construction Guidelines for Microtunneling.* ASCE 27-00, *Standard Practice for Design of Precast Concrete Pipe for Jacking in Trenchless Construction.*	

TRANSMISSION PIPELINES DESIGN CRITERIA

Subject	Criteria	Comments/Reason
Borings and Sampling	A minimum of two soil borings should be completed at each trenchless crossing: one located at the launching pit/shaft and one at the receiving pit/shaft. The need for additional borings should be evaluated based on geotechnical recommendations. Continuous sampling should extend from at least 10 ft above to 10 ft below the proposed pipeline zone. Provide a 5 ft minimum sample interval at all other depths within the borehole.	CI/ASCE 36-01, *Standard Construction Guidelines for Microtunneling*, suggests that a typical average final boring spacing should be on the order of 300 ft. Larger or smaller spacing may be appropriate depending on geologic variability and uncertainties remaining after initial phase borings are completed.
Piezometers	Piezometers should be installed on borings where ground water is expected. Consult geotechnical engineer regarding piezometers in saturated, clayey soils.	Groundwater levels and level fluctuations with time and seasonal precipitation and stream level changes are important to determine. Piezometers are monitored at a regular interval (at least bi-monthly) during the design phase such that at least 6 months of monitoring may be reported.
Permeability	Slug testing is completed within installed piezometers to indicate ground mass permeability.	Ground mass permeability is very important for proper design and management of shafts, launch and reception portals and trenchless construction.
Groundwater Quality	Water samples are properly taken and sent to an analytical laboratory to test for pH, corrosiveness, and dissolved methane, hydrogen sulfide, or volatile organic compounds (VOCs).	Corrosivity data is needed for casing and pipeline design. Extent of dissolved methane, hydrogen sulphide and VOCs will provide an indication on tunnel classification as non-gassy, potentially gassy, or gassy per Occupational Safety and Health Administration (OSHA) regulations.
Laboratory Testing of Soil Samples	At a minimum, representative granular samples are tested to determine grain size distribution and distribution of fines if over 10%. Representative cohesive samples are tested to determine water content, Atterberg Limits, and Unconfined Compressive Strength.	Laboratory test data is needed to help with soil unified soil classification system (USCS) classifications and to provide data for assessment of soil mass groutability, slurry penetration into ground mass, frac-out risk, overload factors, convergence, and Tunnelman's Ground Classifications.

TRANSMISSION PIPELINES DESIGN CRITERIA

Subject	Criteria	Comments/Reason
Settlement Trough and Potential Impacts	A settlement trough analysis is completed in accordance with New & O'Reilly, 1992 or an equivalent method. Risk of damage is assessed for the resulting settlement predictions.	Settlement trough analyses are made for volume losses that are achievable and reasonable for the trenchless method selected. If greater than acceptable settlements and damage risk results, more restrictive trenchless methods can be specified to reduce settlement trough volumes or ground improvement or compensation grouting methods can be specified to isolate the facilities of concern from the ground movement zone.
Heave and Frac-Out	Calculations and assessments are completed to determine risk of heave damage or frac-out (hydraulic fracturing) from excessive slurry pressures, excavation chamber air pressures or contact grouting pressures associated with directional drilling or microtunneling operations.	A high slurry pressure during directional drilling or microtunneling can result in frac-out and flow of bentonite or polymer slurry into Fountain Creek. Excess slurry or contact ground pressures could result in unacceptable heave to the freeway pavement or railroad tracks.
Control of Water and Face Stability	An assessment is made on feasibility and potential impacts of dewatering of shaft or shaft and trenchless zones. This assessment considers the potential consequences of severe face instability and ground loss resulting in sinkholes. If dewatering is unlikely to reduce face instability risk, specifications will require a trenchless method with active face support such as achievable with microtunneling.	Trenchless methods might include directional drilling, open-face pipe jacking or microtunneling. Open-face tunneling would require dewatering to lower groundwater levels to below the tunnel zone. A dewatering viability assessment considers cost, difficulty of dewatering interfaces between low and high permeability ground, and potential impacts of dewatering such as water supply well disruptions, migration of contaminated groundwater or drawdown-settlement.
Shaft Flooding	A construction phase maximum possible flood level is determined. If one or more shafts are located within the flood zone, measures to minimize the risk of shaft and tunnel flooding are designed or specified (such as higher shaft top elevations, and temporary berms around shaft rims).	Surface flooding of shafts or tunnels can have catastrophic impacts on health and safety and tunneling equipment resulting in high impact costs and major delays.
Drive Length and Jacking Force	If pipe jacking or microtunneling is selected, a jacking force analysis is completed to help determine viable drive lengths and any requirements involving intermediate jacking stations. The analysis should use the Bennett-Cording method or similar.	Drive length should be evaluated by final designer to determine if a single drive pipe jack or microtunnel is adequate of if intermediate jacking stations are needed.

TRANSMISSION PIPELINES DESIGN CRITERIA

Subject	Criteria	Comments/Reason
GDR and GBR	The initial phase site investigation (SI) work is be presented in a geotechnical data report (GDR) prepared in accordance with Underground Technology Research Council (UTRC), 1997. Essex, R.J. (ed.). Geotechnical Baseline Reports (GBRs) for Underground Construction – Suggested Guidelines and Practices. Reston: American Society of Civil Engineers. During final design a GBR is prepared by the final design team. The GBR is also prepared in accordance with UTRC, 2006.	Proper preparation of a GDR and GBR that are consistent with the specifications and other contract documents on risk sharing is critically important for trenchless construction. The GBR is prepared by the engineer and not the SI firm responsible for the GDR. Final design will incorporate baseline parameters (such as boulders, ground type quantities, and maximum groundwater levels).
Specifications	Specifications are prepared with a proper balance between performance and prescriptive wording for the risks involved. Trenchless specifications are likely to cover the following: • Auger boring, tunneling or microtunneling • Workshaft excavation and construction • Ground support systems • Control of water • Ground improvement (grouting) at portals and critical zones (might also include compensation grouting below freeway or railroad tracks) • Contact grouting of initial casing • Carrier pipe installation and backfilling with low density cellular grout • Geotechnical instrumentation and monitoring for protection of adjacent property	Performance specifications allow the most contractor flexibility and ingenuity and possibly the best price, but in a low-bid environment may result in excessive risk taking and a high change of construction problems from use of inappropriate equipment or methods. Fully prescription specifications essentially tell the contractor what to do and result in too much risk for the owner and engineer. Generally, a balance between performance and prescriptive requirements will result in more equitable bidding and risk sharing. Generally, more specifications on specific topics are preferable to fewer specifications covering multiple topics. Better specification clarity and conciseness is achievable with this approach.
Drawings	Trenchless and trenchless monitoring details will be provided during final design. Details will include minimum grout and lubrication port requirements for casing pipe, carrier pipe blocking and annular space requirements, shaft portal ground improvement, compensation grouting or ground improvement below roadways and railroads (if required) and geotechnical instrumentation details.	Final structural design of the carrier pipe will be conducted during final design and follow industry standards. Drawings for the casing or carrier pipe may not show final structural design depending on pipe type. A good guideline for division of responsibility and level of detail in design and submittals is given in ASCE 27-00, *Standard Practice for Design of Precast Concrete Pipe for Jacking in Trenchless Construction*.

Appendix B
Gianelli PGP Comparison to Sites PGP

Gianelli PGP Comparison to Sites PGPs
Technical Memorandum (Final)

To:	Henry Luu/HDR
Date:	August 28, 2020
From:	Jeff Smith/Jacobs
Quality Review by:	Peter Rude/Jacobs
Authority Agent Review by:	TBD
Subject:	Comparison of Gianelli PGP (San Luis Reservoir) to Sites Proposed PGPs

1.0 Purpose

The Sites Reservoir Project includes the Funks and the Terminal Regulating Reservoir (TRR) Pumping Generating Plants (PGPs), which will include large pumps and separate hydroelectric turbines. The purpose of this technical memorandum is: (1) to compare the proposed Sites PGPs to the existing Gianelli PGP located at San Luis Reservoir near Santa Nella, California; and (2) to see what can be learned from Gianelli PGP. This request was initiated at the July 1, 2020, Ad hoc Operations and Engineering Workgroup meeting of the Reservoir Committee. Information for Gianelli PGP was obtained from the U.S. Bureau of Reclamation and California Department of Water Resources websites.

2.0 Equipment Comparison

The following table compares various PGP features.

PUMPING AND GENERATING COMPARISON

Feature	Gianelli	Funks	TRR
Pumping System			
Pumping Units			
Duty	8	12	12
Standby	Unsure	1	1
Per Unit			
Power (horsepower)	63,000	8,000	9,000
Flow (cubic feet per second [cfs])	1,375	175	150

PUMPING AND GENERATING COMPARISON

Feature	Gianelli	Funks	TRR
Total (duty)			
Power (horsepower)	504,000	96,000	108,000
Flow (cfs)	11,000	2,100	1,800
Maximum Head (feet)	290	320	420
Generating System			
Generating Units			
Duty	8	2	2
Standby	Unsure	0	0
Per Unit			
Power (kilowatts)	53,000	21,000	13,500
Flow (cfs)	1,640	1,000	500
Total (duty)			
Max Power (kilowatts)	424,000	42,000	27,000
Flow (cfs)	13,120	2,000	1,000
Maximum Head (feet)	290	280	360

3.0 Discussion of Comparison

A comparison of the Gianelli PGP at San Luis Reservoir to the proposed Sites project PGPs (Funks and TRR) shows that Gianelli is considerably larger, even though the heads are comparable. For the pumping condition, each unit is seven to eight times larger than the Sites PGPs. For the generating condition, the Gianelli units are approximately three to four times larger than the Sites generating units. The source of information did not provide a distinction on whether all eight Gianelli units are duty or whether seven are duty and one is standby.

It is important to note that the Gianelli units are combination pump-turbine units that provide both pumping and generating by operating the unit's impellers either forward or reverse. Alternatively, the current Sites design has separate units for pumping and generating, with 12 units at each PGP for pumping and 2 units for generating. Pump-turbine units are very complex and required special custom engineering that is very costly and lengthy. As a result, pump-turbine units are more commonly found in facilities that generate 400 megawatts (MW) or more, which is consistent with the Gianelli facility.

Preliminary calculations indicate Funks generating 42 MW and the TRR generating 27 MW. Discussions with manufacturers and a Jacobs hydroelectric expert confirm the use of pump-turbine units on small generating facilities, like Funks and TRR, are not warranted. The use of separate pumping and generating units as currently planned and presented in our July 23, 2020, deliverable demonstrate the proper engineering approach.

Appendix C
Constant Speed versus Adjustable Speed Pumps and Motors Comparison

Constant-speed versus Adjustable-speed Pumps and Motors Comparison Technical Memorandum (Final)

To:	Henry Luu/HDR
CC:	
Date:	August 28, 2020
From:	Mike Riess/Jacobs, Jeff Smith/Jacobs
Quality Review by:	Bill Misslin/Jacobs
Authority Agent Review by:	TBD
Subject:	Constant-speed versus Adjustable-speed Pumps and Motors Comparison

1.0 Background

Sites Project Authority (Sites) adopted the recommended project (VP7) as provided in the *Sites Project Value Planning Alternatives Appraisal Report*, dated April 2020, to reduce the program cost from $5.2 billion to $3.0 billion. The VP7 project includes major changes to the pumping conditions associated with the Funks and Terminal Regulating Reservoir (TRR) Pumping Generating Plants (PGPs), notably the significantly higher pumping heads because both are now pumping directly to the Sites Reservoir. Design pumping flows and maximum pumping heads for Funks PGP are 2,100 cubic feet per second (cfs) and 317 feet; flows and maximum pumping heads for TRR PGP are 1,800 cfs and 420 feet.

2.0 Purpose

At the July 1, 2020, Ad hoc Operations and Engineering Workgroup Meeting of the Reservoir Committee, the Conveyance Team provided an overview of the proposed PGPs, including 12 duty and 1 standby pump for each PGP. The Conveyance Design Team stated that the wide range of flows and pumping heads will require the use of adjustable-speed drives for each pump. A Workgroup member requested consideration of use of constant-speed drives. The purpose of this technical memorandum is to summarize an analysis of using constant-speed versus adjustable-speed drives for the Funks and TRR PGPs.

This analysis required a modeling effort to determine where the pumps will provide coverage for all the various operating points. Good engineering practice is to operate the pumps within their preferred operating region (POR) where there is less wear and tear on the equipment. However, manufacturers also define an allowable operating region (AOR) within which operating is acceptable, but the AOR comes at the sacrifice of additional wear and tear and lower pump efficiency. Operating points outside the AOR and POR are generally deemed as not acceptable.

3.0 Modeling Analysis

Hydraulic modeling of both the Funks and TRR pumping systems was completed using AFT Fathom (v. 10) hydraulic modeling software. The current layout for the two PGPs is almost identical, so only the Funks PGP layout is shown in plan view on Figure 1. Figure 2 shows the overall system piping schematic layout used for the modeling effort from both Funks PGP and TRR PGP through to Sites Reservoir inlet/outlet tower. Figures 3 through 6 provide pump curves and pumping system curves. Attachment A contains the model data input, such as pipe diameter, pipe length, pipe number, and other information.

3.1 Pump Generating Plant Criteria

The following are common criteria used for both PGPs:

- Pipe Friction Factor (Hazen-Williams) = 130 or 150
- Sites Reservoir Maximum Water Surface Elevation = 498 feet
- Sites Reservoir Minimum Water Surface Elevation = 340 feet

3.1.1 Funks PGP

The Funks PGP modeling assumptions for the system and pump are included in Table 1.

TABLE 1: FUNKS PUMP DESIGN CRITERIA

Subject	Criteria
Maximum Flow	2,100 cfs
Number of Pumping Units	13 (12 duty + 1 standby)
Capacity at Rated Point	175 cfs @ 320 feet
Static Head, Maximum	298 feet
Static Head, Minimum	135 feet
Rated Pump Efficiency	89 percent
Pump Type and Configuration	Vertical mixed flow, multi-stage
Motor Size	8,000 horsepower
Motor Type	Induction, vertical solid shaft, high thrust
Nominal Speed	505 rotations per minute (rpm)

Figure 3 provides pump performance information for the Funks pump and includes various characteristics, such as full-speed pump curve, efficiency curve, horsepower requirements, preferred operating region (POR),and AOR.

3.1.2 TRR PGP

The TRR PGP modeling assumptions for the system and pump are included in Table 2.

TABLE 2: TRR PUMP DESIGN CRITERIA

Subject	Criteria
Maximum Flow	1,800 cfs
Number of Pumping Units	13 (12 Duty + 1 Standby)
Capacity at Rated Point	150 cfs @ 420 ft
Static Head, Maximum	379 feet
Static Head, Minimum	216 feet
Rated Pump Efficiency	88 percent
Pump Type & Configuration	Vertical Mixed Flow, Multi-Stage
Motor Size	9,000 hp
Motor Type	Induction, Vertical Solid Shaft, High Thrust
Nominal Speed	590 rpm

Figure 4 provides pump performance information for the TRR pump and includes various characteristics such as full speed pump curve, efficiency curve, horsepower requirements, POR, and allowable operating region (AOR).

3.2 Modeling Conditions

The Fathom model was used to simulate the highest and lowest static head conditions for each of the PGPs. Table 3 summarizes the conditions used in the modeling exercise. The low static and high static conditions for each PGP set the system boundaries for pump selection.

TABLE 3: SUMMARY OF MODELING CONDITIONS

	Funks PGP		TRR PGP	
Criteria	High Static	Low Static	High Static	Low Static
Sites Reservoir Level (feet)	498	340	498	340
Funks Reservoir Level (feet)	199	205	199	N/A
TRR Reservoir Level (feet)	119	N/A	124	119
Pipeline Friction Coefficient	130	150	130	150
Funks PGP Operating	Yes	Yes	Yes	No
TRR PGP Operating	Yes	No	Yes	Yes

3.3 Modeling Results

High and low static pumping scenarios were modeled to develop the system curves on each composite pump as shown in Figures 5 and 6. For each PGP, representative pump curves are superimposed over the respective system curves to display parallel pump behavior from single-pump to 12-pump operation. Isoefficiency lines corresponding to the pump POR and AOR are superimposed over the system curves to indicate the region and quality of flow coverage when each pump is operated by an adjustable-speed drive (ASD). Single-pump operation at a reduced speed, corresponding to the intersection of minimum AOR and the low head system curve, is shown to indicate the minimum recommended pump flow when considering only hydraulic criteria (other criteria may govern pump minimum speed).

Figures 5 and 6 depict the operational gaps – areas where the pumps are not operating within the POR or the AOR. The information contained in Figures 5 and 6 can be challenging to interpret, unless the reader is well versed in pump design. In simple terms, the potential operating area is vast and contained between the upper high head system curve and the lower Low Head System Curve, and between the minimum flow near zero and the maximum flow along the horizontal graph line. Each pump type has a minimum and maximum POR (see Figures 3 and 4). On Figures 5 and 6, the minimum is shown as a green line and the maximum is shown as a blue line.

Figure 5 provides the results of using ASDs to cover the entire operating range. As shown on Figure 5, the currently selected pump covers almost the entire operating region within the POR of the pumps at minimum flow (with one pump operating), to the maximum flow (with 12 pumps operating). On each end of the operating area is a very small area (shown in solid blue) where the pumps will operate in the allowable operating range to meet this design condition. There are also two areas of AOR operation between pumps 1 and 2 and between pumps 2 and 3. There is also a very small operating area (shown in solid red) at high flow and lowest head where pump operation is not allowed. Jacobs is confident that we can work with the pump manufacturers to slightly modify this pump to operate within this solid red area (not allowable operational area).

3.3.1 TRR PGP

Figure 6 shows the results of using the ASDs to cover the operating area. The results show that this pump can cover the entire area, with a small exception when flows are very low (below 100 cfs).

4.0 Constant-speed versus Adjustable-speed Drives

4.1 General Overview

The information in this memorandum has primarily focused on mechanical aspect of pump station design, but there are also differences between electrical design for ASDs and constant-speed pumps. This section presents discussion for both design disciplines.

4.1.1 Mechanical Design

For best efficiency and equipment longevity, pumps should be operated within the POR. Pumps may operate outside of the POR and within the AOR, but this course is not recommended unless unavoidable, because both efficiency and pump life will be reduced. Adjustable-speed pumping permits operators or automated control systems to more easily keep pumps within the POR for almost the entire operating area.

Using a constant-speed pump is applicable when a relatively constant operating point and somewhat constant flow exist. Both the Funks PGP and TRR PGP will have variable flow and variable head conditions that will make using constant-speed pumps essentially impossible.

Although the system and pump curves provided in Figures 5 and 6 contain many lines to interpret, they show that constant-speed pumps can only operate along the vertical curved lines; points between these lines are conditions that cannot be met by constant-speed pumps. The use of constant-speed pumps will not allow the PGP to match the flows from the Tehama-Colusa Canal and Glenn-Colusa Irrigation District Canal to pump into Sites Reservoir. At Funks PGP, constant speed pumps would operate outside the AOR when the pumps are operating at a head lower than 240 feet. At the TRR, constant speed pumps would operate outside the AOR when the pumps are operating at a head lower than 270 feet.

4.1.2 Electrical Design

The turbine generator and utility requirements will drive the method of grounding used on the switchgear. Constant-speed motors will be subject to the system grounding chosen, which may not be desirable for medium-voltage motors, where low impedance grounding is the preferred option. ASDs with isolation/phase shifting transformers will isolate the motors from the system grounding.

The two common types of motors to consider for this project include synchronous motors and induction motors. Given that using constant-speed pumps is essentially impossible, the use of induction motors is recommended because they work well with ASDs and are less expensive than synchronous motors.

Using ASDs with isolation transformers allows for flexibility with motor voltage selection, potentially saving considerable costs with coordinating a motor and pump.

4.2 Funks PGP

The pump-system curve for Funks (Figure 5) shows representative pumps operating in parallel and at a common pump speed (all pumps on ASDs and all pumps driven at the same speed), with flow coverage predominantly within the POR, from 100 cfs to approximately 1,600 cfs (design flow is 2,100 cfs). When total flow exceeds this 1,600 cfs, a region of operation is revealed within the AOR that is most pronounced at lower head conditions. Also, a small region of operation outside of the POR and AOR exists, from approximately 2,000 to 2,100 cfs; but this area is limited to extreme low head conditions. Jacobs can work with pump manufacturers to refine pump selection, having a POR envelope further “out” on the pump curve to cover up to the 2,100 cfs design flow under all head conditions.

As part of this analysis, Jacobs looked at using a combination of ASD pumps and constant-speed pumps. Applying one or more constant-speed pumps to operate in conjunction with ASD pumps, the full-speed pump head at which the minimum and maximum POR flow rates occur were evaluated relative to the system curves. The currently selected pump has a head of 350 feet at minimum POR flow, and 288 feet at the maximum POR flow. Relative to the system curves, there is a very limited range of static head conditions that would support use of a constant-speed pump operating within the POR (less than 10 percent of the static range – the area below the solid horizontal red line is outside of the AOR). If constant-speed pumps could operate in the AOR, then the range of operation would still be quite limited (less than 50 percent of the static range).

4.3 TRR PGP

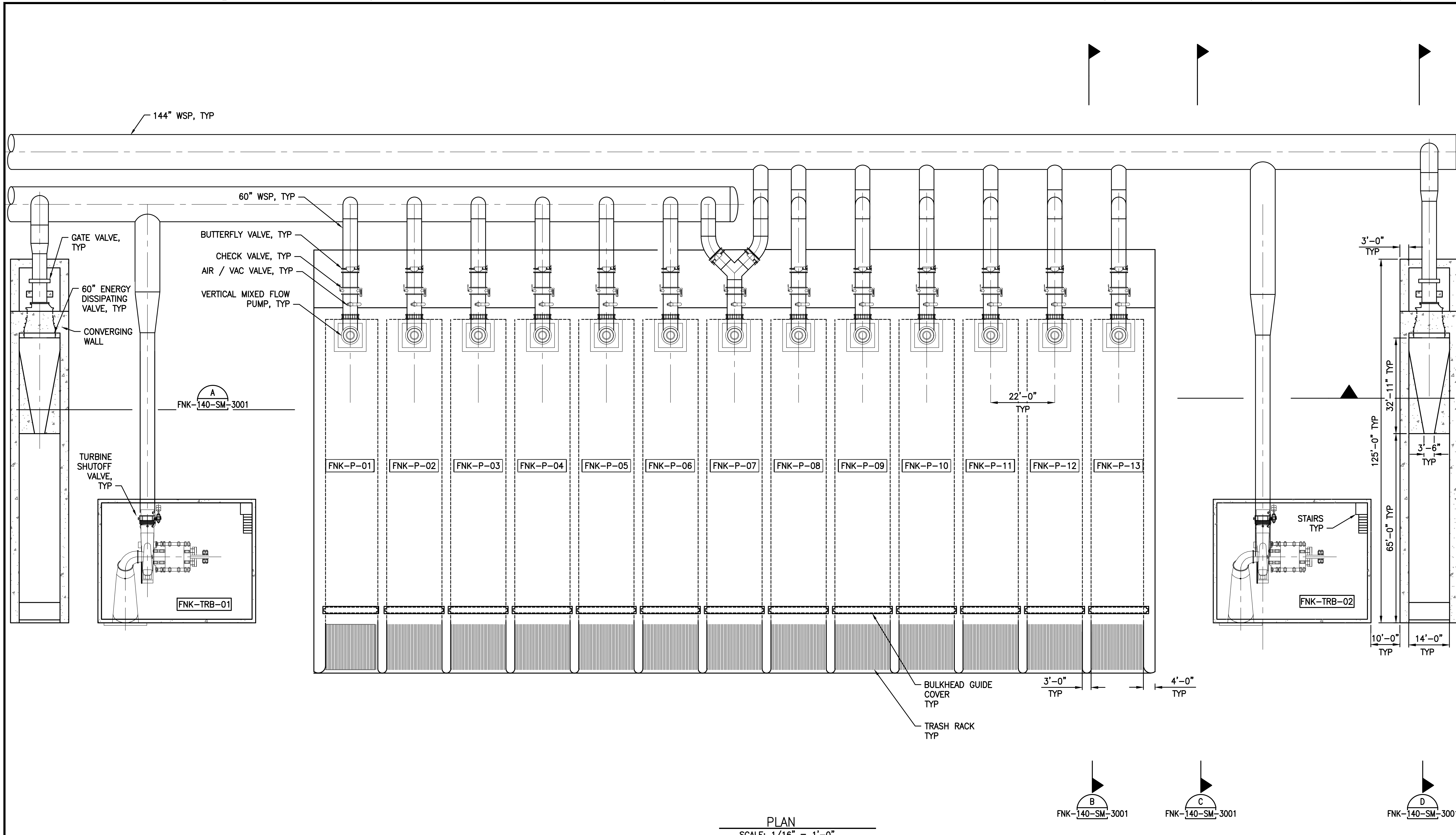
The pump-system curve (Figure 6) shows representative pumps operating in parallel and at a common pump speed (all pumps on ASDs and all pumps driven at the same speed), with flow coverage within the POR across a flow rate of 100 to 1,800 cfs.

As part of this analysis, Jacobs looked at using a combination of ASD and constant-speed pumps. Applying one or more constant-speed pumps to operate in conjunction with ASD pumps, the full-speed pump head at which the minimum and maximum POR flow rates occur were evaluated relative to the system curves. The currently selected pump has a head of 422 feet at minimum POR flow, and 338 feet at the maximum POR flow. Relative to the system curves, there is a very limited range of static head conditions that would support use of a constant-speed pump operating within the POR (less than 50 percent of the static head range – the area below the solid horizontal red line is outside of the AOR). If constant-speed pumps could operate in the AOR, then the range of operation would still be quite limited (less than 80 percent of the static range).

5.0 Recommendation

The primary purpose of this task was to evaluate whether constant-speed pumps could be used for the PGPs, as opposed to the currently recommended ASD pumps. What this exercise showed is that constant speed pumps would operate outside of the AOR and POR at lower system head conditions and therefore not recommended. Use of constant-speed pumps will limit the operational points for the system, reduce the overall pumping efficiency, provide unnecessary wear and tear on the pumps, and limit suppliers. Given the wide variation in pumping head resulting from fluctuations in Sites Reservoir water levels and variations in flow from the Tehama-Colusa Canal and Glenn-Colusa Irrigation District Canal, Jacobs recommends using all ASDs for both PGPs. Although installation of ASDs may add capital costs of approximately $10 to $12 million for both PGPs, the reduced operational cost for more efficient pumping and reduced wear and tear will lead to overall reduced costs over the life of the project.

Figures



PLAN
SCALE: 1/16" = 1'-0"

I/R	DATE	BY	CHK	APPR	DESCRIPTION

DESIGNED BY: KELLOGG/SMITH
DRAWN BY: S HAUNG
CHECKED BY:
IN CHARGE: P RUDE
DATE: 07-20-2020

JACOBS
2525 AIRPARK DR
REDDING CA, 96001
PHONE: (530) 243-5831

REGISTERED PROFESSIONAL ENGINEER
STATE OF CALIFORNIA

Sites

SITES RESERVOIR PROJECT

FIGURE 1
FUNKS/TRR PGP LAYOUT

VERIFY SCALES
BAR IS ONE INCH ON ORIGINAL DRAWING. ADJUST SCALE FOR REDUCED PLOTS

DRAWING NO.
FNK-140-SM-2001

PLOTTED BY:CAVE, DAVID/RDD -July 23, 2020-3:44:42 PM
DRAWING: FNK-140-SM-2001_D3380600.dwg
JACOBS PROJECT # D3880600

DRAFT - PROJECT FEASABILITY STUDY - NOT FOR CONSTRUCTION

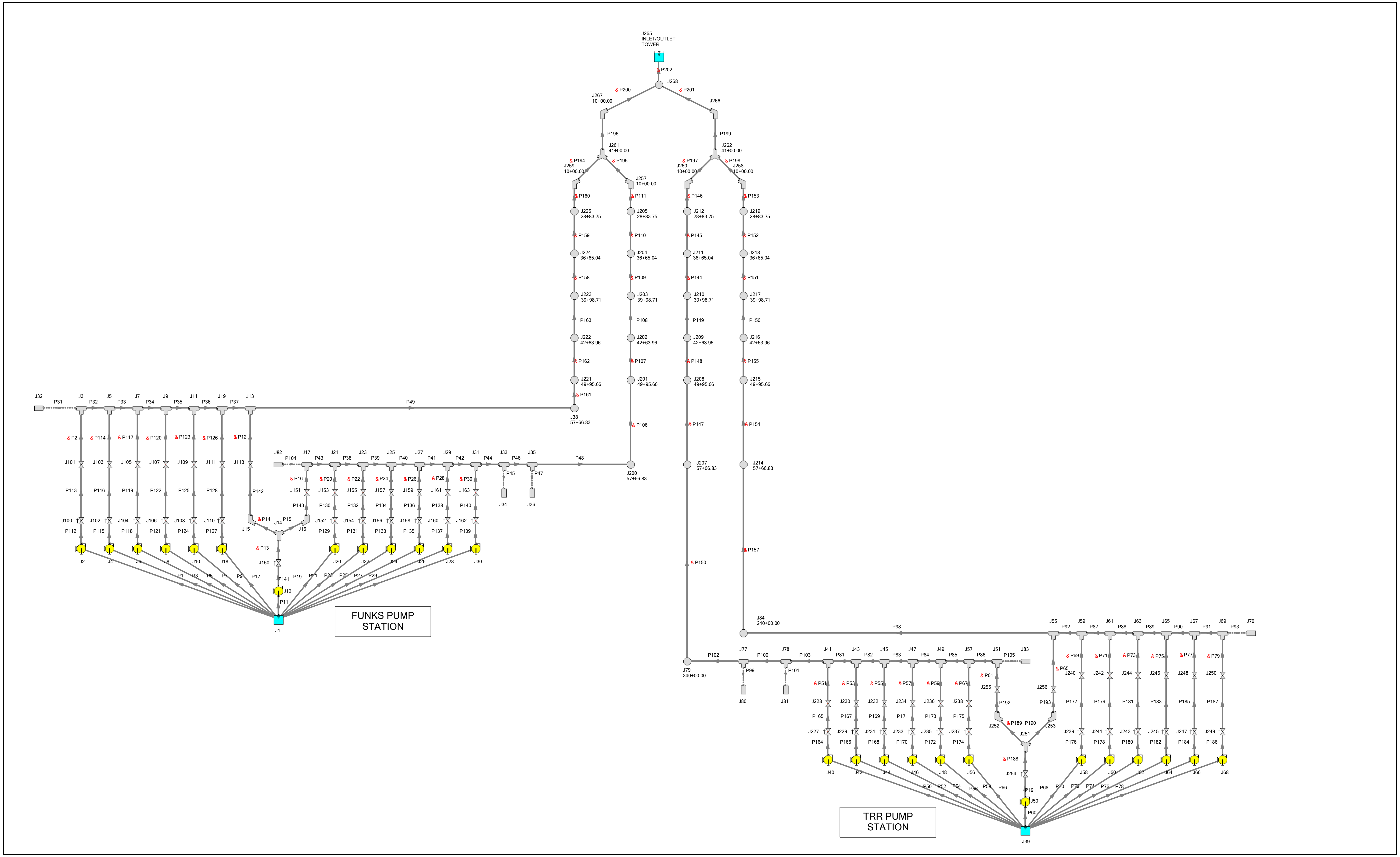


FIGURE 2: HYDRAULIC MODEL SCHEMATIC

Representative Pump - Funks Pump Station

Model: VCW	**Size: 67WMCC**	**60 Hz**	**RPM: 505**	**Stages: 3**

Job/Inq.No.:	Funks Reservoir				
Purchaser:	SITES				
End User:	TBD	Issued by:	Carlos Preciado	Rev.:	0
Item/Equip.No.:	ITEM 001-012 Large	Quotation No.:	CP2018-06-25	Date:	6/30/2020
Service:	Vertical Pumps				
		Certified By:			

Operating Conditions | **Pump Performance**

Liquid:	Water	Published Efficiency:	89.0%	Specific Speed:	3,684 (RPM,GPM,ft)
Temp.:	70.0 °F	Rated Pump Efficiency:	89.0%	Min Hydraulic Flow:	39,273 GPM
S.G.:	1.000	Rated Total Power:	7126.6 hp	Shut off Head:	481.2 ft
Flow:	78,545 GPM	Min. Motor Rating:	7840 hp		
TDH:	320.0 ft	Imp Dia.:	40.27 in	Qty:	12
NPSHa:	48.3 ft	NPSHr:	21.1 ft		

GOULDS PUMPS VERTICAL PUMP DIVISION INDUSTRY, CALIFORNIA	CENTRIFUGAL PUMP CHARACTERISTICS Performance Standard: ANSI/HI 14.6-2011, grade 2B	Curve #:	E60367WMPF
		RPM 505	
		Model:	VCW
		Size:	67WMCC - 2.82

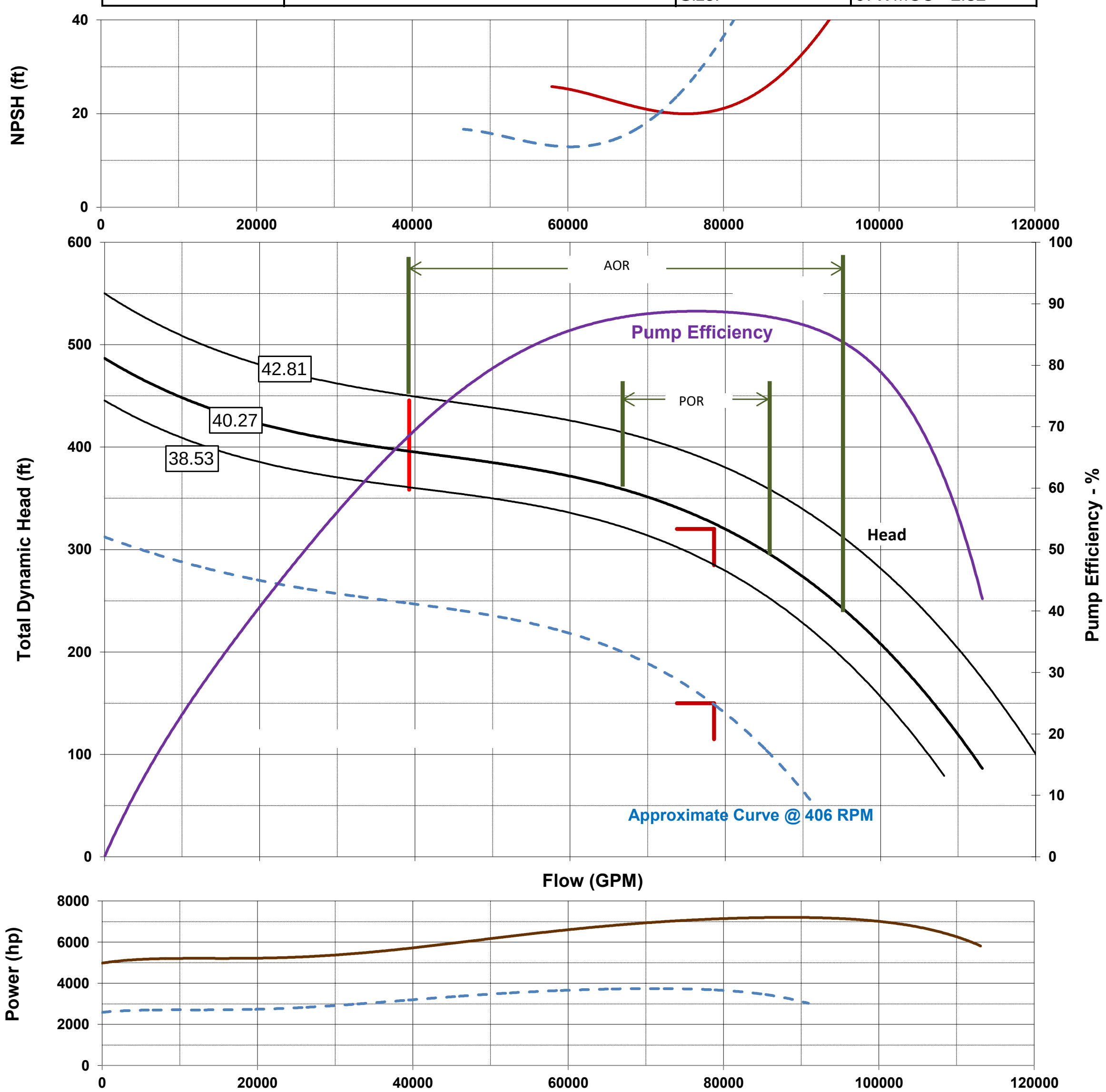


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FIGURE 3: FUNKS PUMP CURVE

Representative Pump - TRR Pump Station

Model: VCW	Size: 63WMCC	60 Hz	RPM: 590	Stages: 3

Job/Inq.No.:	TRR Reservoir				
Purchaser:	SITES				
End User:	TBD	Issued by:	Carlos Preciado	Rev.:	0
Item/Equip.No.:	ITEM 001-012 Large	Quotation No.:	CP2018-06-25	Date:	6/30/2020
Service:	Vertical Pumps				
		Certified By:			

Operating Conditions | **Pump Performance**

Liquid:	Water	Published Efficiency:	88.7%	Specific Speed:	3,680 (RPM,GPM,ft)
Temp.:	70.0 °F	Rated Pump Efficiency:	88.7%	Min Hydraulic Flow:	40,395 GPM
S.G.:	1.000	Rated Total Power:	8044.4 hp	Shut off Head:	577.9 ft
Flow:	67,325 GPM	Min. Motor Rating:	8849 hp		
TDH:	420.0 ft	Imp Dia.:	37.77 in	Qty:	12
NPSHa:	43.5 ft	NPSHr:	24.6 ft		

GOULDS PUMPS VERTICAL PUMP DIVISION INDUSTRY, CALIFORNIA	CENTRIFUGAL PUMP CHARACTERISTICS Performance Standard: ANSI/HI 14.6-2011, grade 2B	Curve #: E60363WMPF RPM 590 Model: VCW Size: 63WMCC - 2.6463

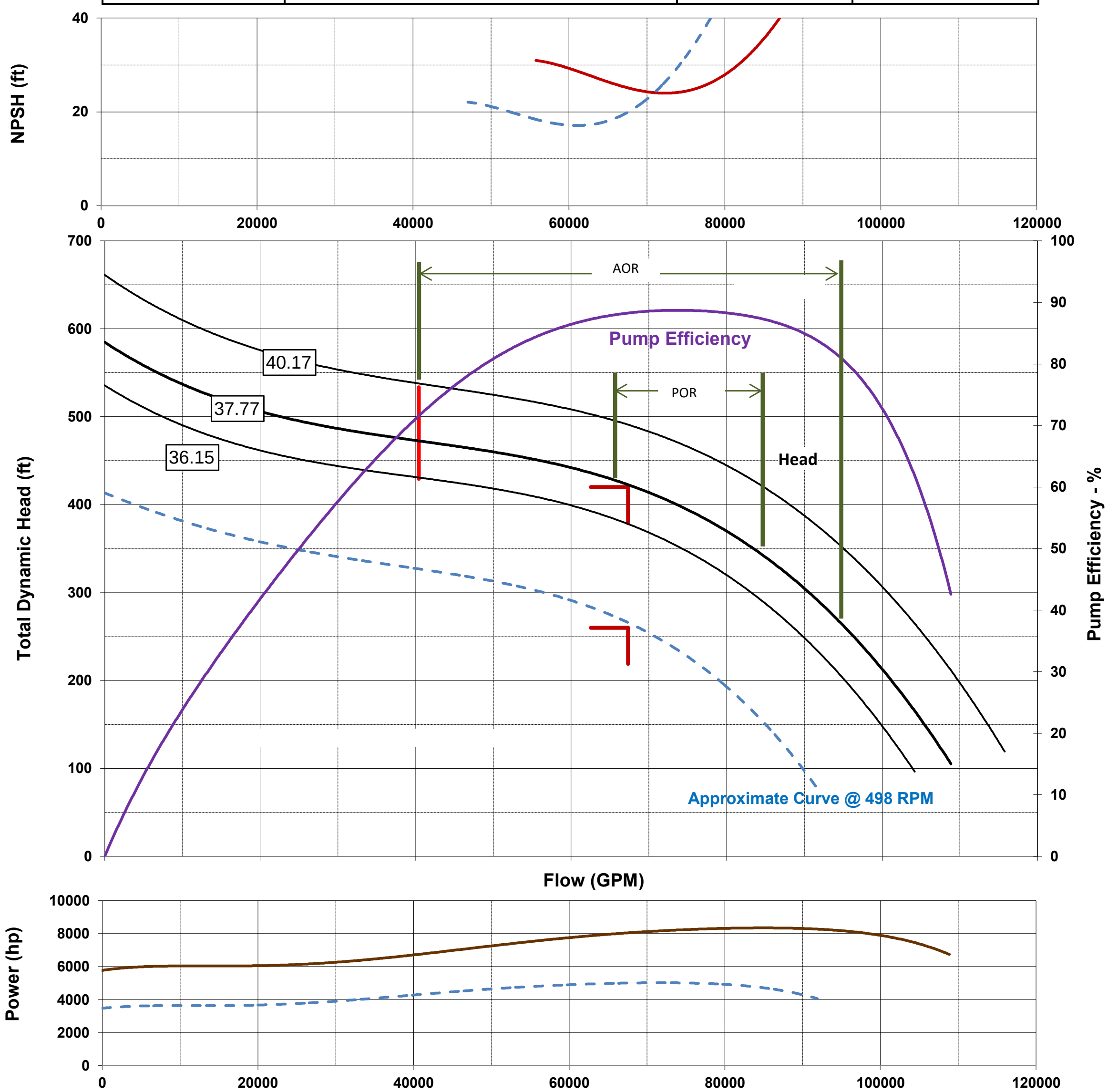

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FIGURE 4: TRR PUMP CURVE

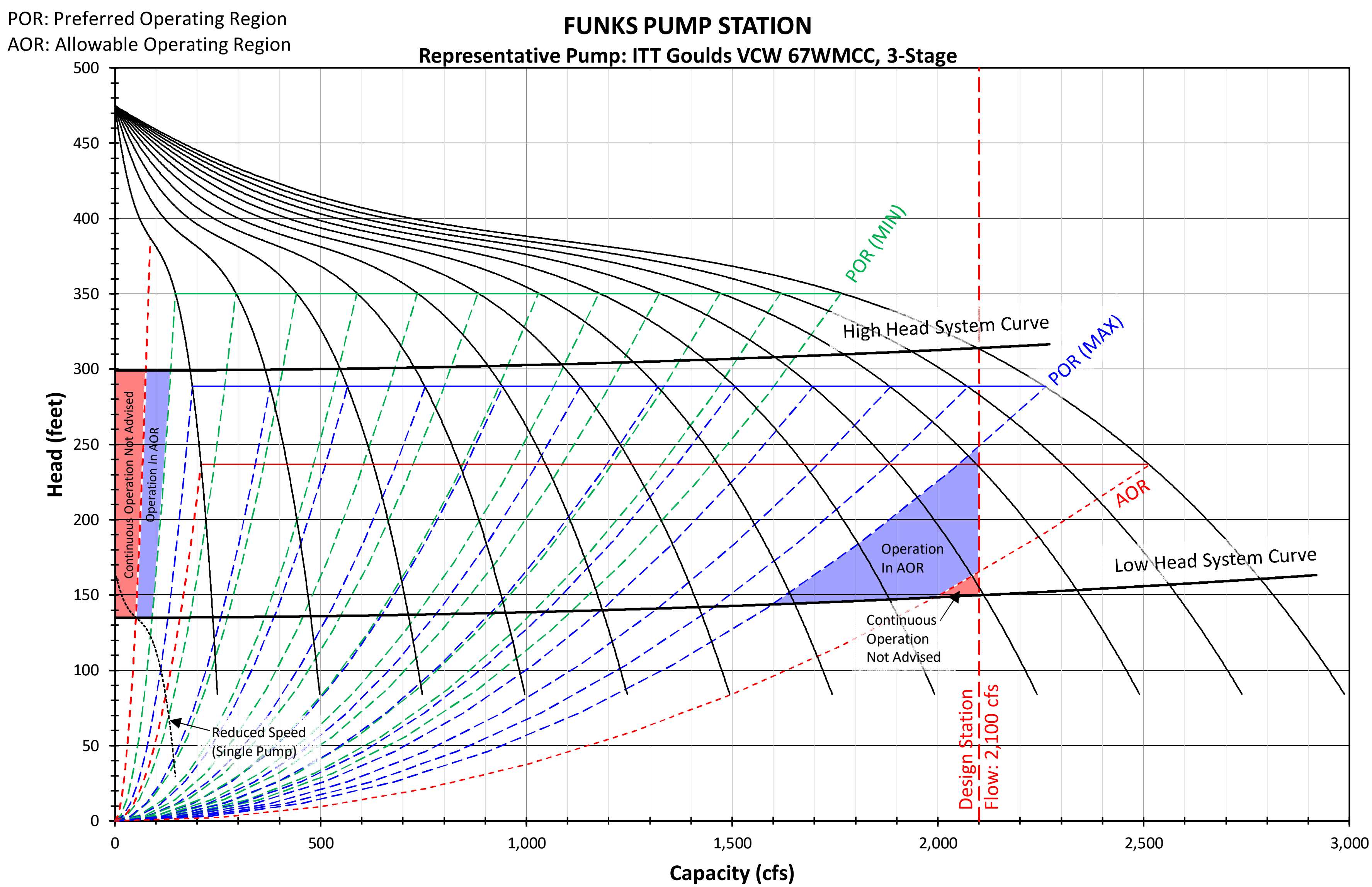


FIGURE 5: FUNKS PUMPING SYSTEM CURVES

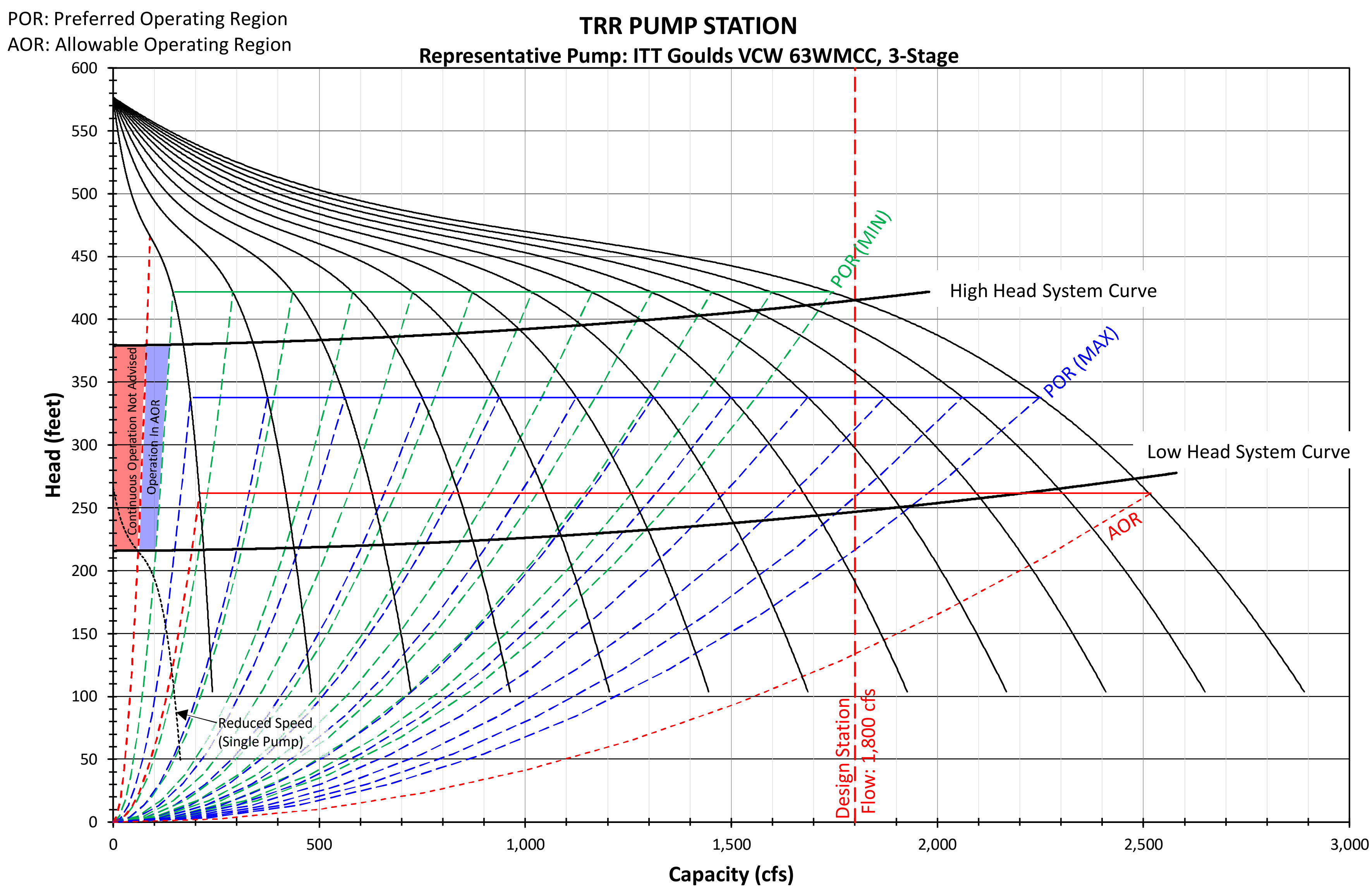
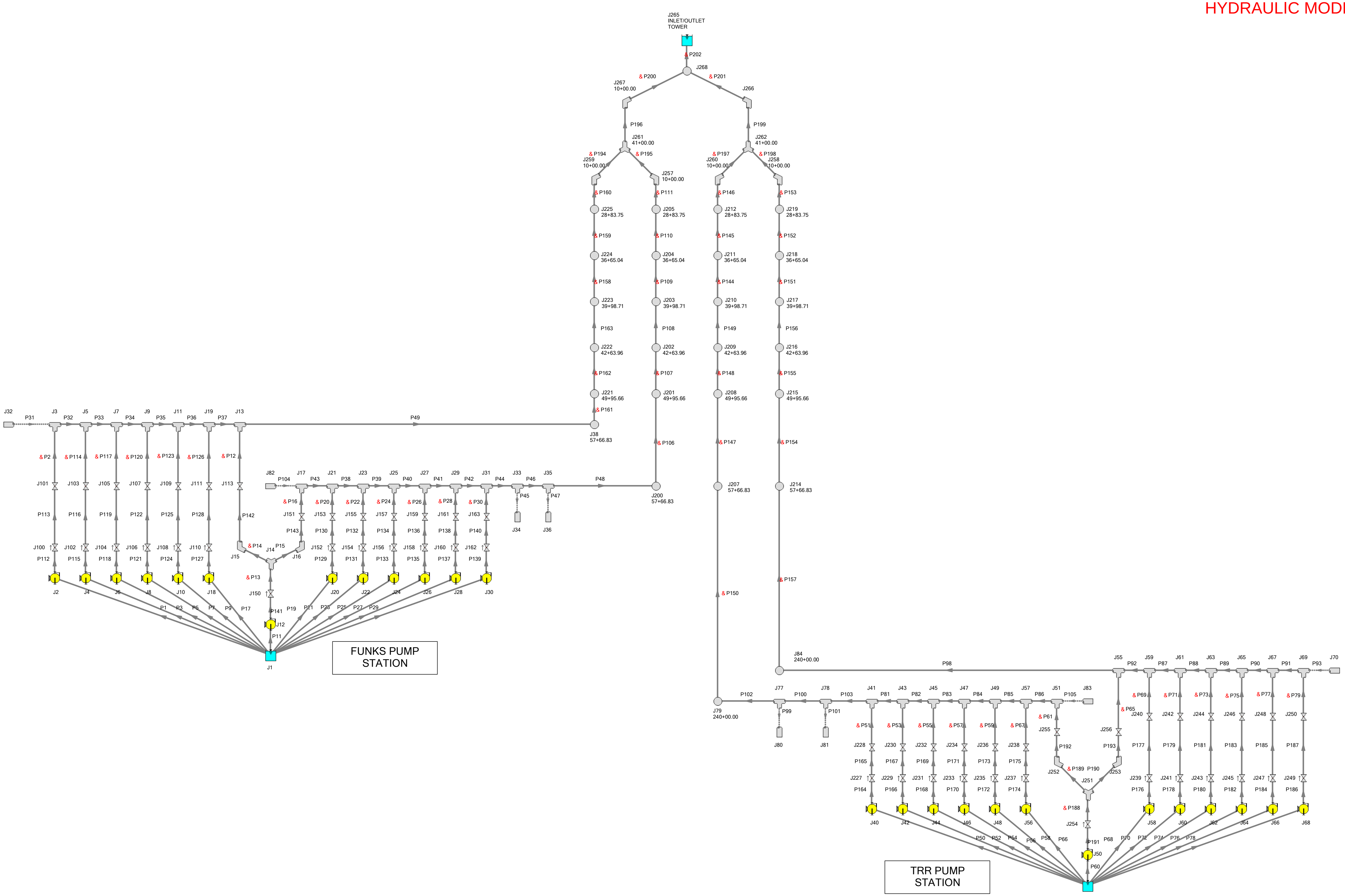


FIGURE 6: TRR PUMPING SYSTEM CURVES

Attachment A
Hydraulic Model Data

HYDRAULIC MODEL SCHEMATIC



HYDRAULIC MODEL INPUT DATA

General

Title: AFT Fathom Model
Input File: C:\Users\shussain.JEG\Documents\CH2MHILL\Sites Reservoir\Sites_Reservoir_PS_ Hydraulics.fth

Number Of Pipes= 189
Number Of Junctions= 164

Pressure/Head Tolerance= 0.0001 relative change
Flow Rate Tolerance= 0.0001 relative change
Temperature Tolerance= 0.0001 relative change
Flow Relaxation= (Automatic)
Pressure Relaxation= (Automatic)

Constant Fluid Property Model
Fluid Database: AFT Standard
Fluid: Water at 1 atm
Max Fluid Temperature Data= 212 deg. F
Min Fluid Temperature Data= 32 deg. F
Temperature= 70 deg. F
Density= 62.30841 lbm/ft3
Viscosity= 2.360044 lbm/hr-ft
Vapor Pressure= 0.3615736 psia
Viscosity Model= Newtonian
Apply laminar and non-Newtonian correction to: Pipe Fittings & Losses, Junction K factors, Junction Special Losses, Junction Polynomials
Corrections applied to the following junctions: Branch, Reservoir, Assigned Flow, Assigned Pressure, Area Change, Bend, Tee or Wye, Spray Discharge, Relief Valve

Ambient Pressure (constant)= 1 atm
Gravitational Acceleration= 1 g
Turbulent Flow Above Reynolds Number= 4000
Laminar Flow Below Reynolds Number= 2300

Pipes

Pipe	Name	Pipe Defined	Length	Length Units	Hydraulic Diameter	Hydraulic Diam. Units	Friction Data Set	Roughness	Roughness Units	Losses (K)	Initial Flow	Initial Flow Units	Junctions (Up,Down)	Geometry	Material	Size	Type	Special Condition
1	Pipe	Yes	0.1	feet	60	inches	Unspecified	130	C Hazen-Williams	0			1, 2	Cylindrical Pipe	User Specified			None
2	Pipe	Yes	36	feet	60	inches	Unspecified	130	C Hazen-Williams	0.708			101, 3	Cylindrical Pipe	User Specified			None
3	Pipe	Yes	0.1	feet	60	inches	Unspecified	130	C Hazen-Williams	0			1, 4	Cylindrical Pipe	User Specified			None
5	Pipe	Yes	0.1	feet	60	inches	Unspecified	130	C Hazen-Williams	0			1, 6	Cylindrical Pipe	User Specified			None
7	Pipe	Yes	0.1	feet	60	inches	Unspecified	130	C Hazen-Williams	0			1, 8	Cylindrical Pipe	User Specified			None
9	Pipe	Yes	0.1	feet	60	inches	Unspecified	130	C Hazen-Williams	0			1, 10	Cylindrical Pipe	User Specified			None
11	Pipe	Yes	0.1	feet	60	inches	Unspecified	130	C Hazen-Williams	0			1, 12	Cylindrical Pipe	User Specified			None
12	Pipe	Yes	22.5	feet	60	inches	Unspecified	130	C Hazen-Williams	0.168			113, 13	Cylindrical Pipe	User Specified			None
13	Pipe	Yes	6	feet	60	inches	Unspecified	130	C Hazen-Williams	0.24			150, 14	Cylindrical Pipe	User Specified			Closed
14	Pipe	Yes	9.5	feet	60	inches	Unspecified	130	C Hazen-Williams	0.3			14, 15	Cylindrical Pipe	User Specified			Closed
15	Pipe	Yes	9.5	feet	60	inches	Unspecified	130	C Hazen-Williams	0			14, 16	Cylindrical Pipe	User Specified			Closed
16	Pipe	Yes	17	feet	60	inches	Unspecified	130	C Hazen-Williams	0.168			151, 17	Cylindrical Pipe	User Specified			None
17	Pipe	Yes	0.1	feet	60	inches	Unspecified	130	C Hazen-Williams	0			1, 18	Cylindrical Pipe	User Specified			None
19	Pipe	Yes	0.1	feet	60	inches	Unspecified	130	C Hazen-Williams	0			1, 20	Cylindrical Pipe	User Specified			None
20	Pipe	Yes	26	feet	60	inches	Unspecified	130	C Hazen-Williams	0.54			153, 21	Cylindrical Pipe	User Specified			None
21	Pipe	Yes	0.1	feet	60	inches	Unspecified	130	C Hazen-Williams	0			1, 22	Cylindrical Pipe	User Specified			None
22	Pipe	Yes	26	feet	60	inches	Unspecified	130	C Hazen-Williams	0.54			155, 23	Cylindrical Pipe	User Specified			None
23	Pipe	Yes	0.1	feet	60	inches	Unspecified	130	C Hazen-Williams	0			1, 24	Cylindrical Pipe	User Specified			None
24	Pipe	Yes	26	feet	60	inches	Unspecified	130	C Hazen-Williams	0.54			157, 25	Cylindrical Pipe	User Specified			None
25	Pipe	Yes	0.1	feet	60	inches	Unspecified	130	C Hazen-Williams	0			1, 26	Cylindrical Pipe	User Specified			None
26	Pipe	Yes	26	feet	60	inches	Unspecified	130	C Hazen-Williams	0.54			159, 27	Cylindrical Pipe	User Specified			None
27	Pipe	Yes	0.1	feet	60	inches	Unspecified	130	C Hazen-Williams	0			1, 28	Cylindrical Pipe	User Specified			None
28	Pipe	Yes	26	feet	60	inches	Unspecified	130	C Hazen-Williams	0.54			161, 29	Cylindrical Pipe	User Specified			None
29	Pipe	Yes	0.1	feet	60	inches	Unspecified	130	C Hazen-Williams	0			1, 30	Cylindrical Pipe	User Specified			None
30	Pipe	Yes	26	feet	60	inches	Unspecified	130	C Hazen-Williams	0.54			163, 31	Cylindrical Pipe	User Specified			None
31	Pipe	Yes	113	feet	144	inches	Unspecified	130	C Hazen-Williams	0			32, 3	Cylindrical Pipe	User Specified			None
32	Pipe	Yes	22	feet	144	inches	Unspecified	130	C Hazen-Williams	0			3, 5	Cylindrical Pipe	User Specified			None
33	Pipe	Yes	22	feet	144	inches	Unspecified	130	C Hazen-Williams	0			5, 7	Cylindrical Pipe	User Specified			None
34	Pipe	Yes	22	feet	144	inches	Unspecified	130	C Hazen-Williams	0			7, 9	Cylindrical Pipe	User Specified			None
35	Pipe	Yes	22	feet	144	inches	Unspecified	130	C Hazen-Williams	0			9, 11	Cylindrical Pipe	User Specified			None
36	Pipe	Yes	22	feet	144	inches	Unspecified	130	C Hazen-Williams	0			11, 19	Cylindrical Pipe	User Specified			None
37	Pipe	Yes	13	feet	144	inches	Unspecified	130	C Hazen-Williams	0			19, 13	Cylindrical Pipe	User Specified			None
38	Pipe	Yes	22	feet	144	inches	Unspecified	130	C Hazen-Williams	0			21, 23	Cylindrical Pipe	User Specified			None
39	Pipe	Yes	22	feet	144	inches	Unspecified	130	C Hazen-Williams	0			23, 25	Cylindrical Pipe	User Specified			None
40	Pipe	Yes	22	feet	144	inches	Unspecified	130	C Hazen-Williams	0			25, 27	Cylindrical Pipe	User Specified			None
41	Pipe	Yes	22	feet	144	inches	Unspecified	130	C Hazen-Williams	0			27, 29	Cylindrical Pipe	User Specified			None
42	Pipe	Yes	22	feet	144	inches	Unspecified	130	C Hazen-Williams	0			29, 31	Cylindrical Pipe	User Specified			None
43	Pipe	Yes	13	feet	144	inches	Unspecified	130	C Hazen-Williams	0			17, 21	Cylindrical Pipe	User Specified			None
44	Pipe	Yes	49.5	feet	144	inches	Unspecified	130	C Hazen-Williams	0			31, 33	Cylindrical Pipe	User Specified			None
45	Pipe	Yes	1	feet	144	inches	Unspecified	130	C Hazen-Williams	0			33, 34	Cylindrical Pipe	User Specified			None
46	Pipe	Yes	57	feet	144	inches	Unspecified	130	C Hazen-Williams	0			33, 35	Cylindrical Pipe	User Specified			None
47	Pipe	Yes	1	feet	60	inches	Unspecified	130	C Hazen-Williams	0			35, 36	Cylindrical Pipe	User Specified			None
48	Pipe	Yes	375	feet	144	inches	Unspecified	130	C Hazen-Williams	0			35, 200	Cylindrical Pipe	User Specified			None
49	Pipe	Yes	612	feet	144	inches	Unspecified	130	C Hazen-Williams	0			13, 38	Cylindrical Pipe	User Specified			None
50	Pipe	Yes	0.1	feet	60	inches	Unspecified	130	C Hazen-Williams	0			39, 40	Cylindrical Pipe	User Specified			None

Pipe	Name	Pipe Defined	Length	Length Units	Hydraulic Diameter	Hydraulic Diam. Units	Friction Data Set	Roughness	Roughness Units	Losses (K)	Initial Flow	Initial Flow Units	Junctions (Up,Down)	Geometry	Material	Size	Type	Special Condition
51	Pipe	Yes	52	feet	60	inches	Unspecified	130	C Hazen-Williams	0.54			228, 41	Cylindrical Pipe	User Specified			None
52	Pipe	Yes	0.1	feet	60	inches	Unspecified	130	C Hazen-Williams	0			39, 42	Cylindrical Pipe	User Specified			None
53	Pipe	Yes	52	feet	60	inches	Unspecified	130	C Hazen-Williams	0.54			230, 43	Cylindrical Pipe	User Specified			None
54	Pipe	Yes	0.1	feet	60	inches	Unspecified	130	C Hazen-Williams	0			39, 44	Cylindrical Pipe	User Specified			None
55	Pipe	Yes	52	feet	60	inches	Unspecified	130	C Hazen-Williams	0.54			232, 45	Cylindrical Pipe	User Specified			None
56	Pipe	Yes	0.1	feet	60	inches	Unspecified	130	C Hazen-Williams	0			39, 46	Cylindrical Pipe	User Specified			None
57	Pipe	Yes	52	feet	60	inches	Unspecified	130	C Hazen-Williams	0.54			234, 47	Cylindrical Pipe	User Specified			None
58	Pipe	Yes	0.1	feet	60	inches	Unspecified	130	C Hazen-Williams	0			39, 48	Cylindrical Pipe	User Specified			None
59	Pipe	Yes	52	feet	60	inches	Unspecified	130	C Hazen-Williams	0.54			236, 49	Cylindrical Pipe	User Specified			None
60	Pipe	Yes	0.1	feet	60	inches	Unspecified	130	C Hazen-Williams	0			39, 50	Cylindrical Pipe	User Specified			None
61	Pipe	Yes	24.5	feet	60	inches	Unspecified	130	C Hazen-Williams	0.168			255, 51	Cylindrical Pipe	User Specified			Closed
65	Pipe	Yes	24.5	feet	60	inches	Unspecified	130	C Hazen-Williams	0.168			256, 55	Cylindrical Pipe	User Specified			Closed
66	Pipe	Yes	0.1	feet	60	inches	Unspecified	130	C Hazen-Williams	0			39, 56	Cylindrical Pipe	User Specified			None
67	Pipe	Yes	52	feet	60	inches	Unspecified	130	C Hazen-Williams	0.54			238, 57	Cylindrical Pipe	User Specified			None
68	Pipe	Yes	0.1	feet	60	inches	Unspecified	130	C Hazen-Williams	0			39, 58	Cylindrical Pipe	User Specified			None
69	Pipe	Yes	52	feet	60	inches	Unspecified	130	C Hazen-Williams	0.54			240, 59	Cylindrical Pipe	User Specified			None
70	Pipe	Yes	0.1	feet	60	inches	Unspecified	130	C Hazen-Williams	0			39, 60	Cylindrical Pipe	User Specified			None
71	Pipe	Yes	52	feet	60	inches	Unspecified	130	C Hazen-Williams	0.54			242, 61	Cylindrical Pipe	User Specified			None
72	Pipe	Yes	0.1	feet	60	inches	Unspecified	130	C Hazen-Williams	0			39, 62	Cylindrical Pipe	User Specified			None
73	Pipe	Yes	52	feet	60	inches	Unspecified	130	C Hazen-Williams	0.54			244, 63	Cylindrical Pipe	User Specified			None
74	Pipe	Yes	0.1	feet	60	inches	Unspecified	130	C Hazen-Williams	0			39, 64	Cylindrical Pipe	User Specified			None
75	Pipe	Yes	52	feet	60	inches	Unspecified	130	C Hazen-Williams	0.54			246, 65	Cylindrical Pipe	User Specified			None
76	Pipe	Yes	0.1	feet	60	inches	Unspecified	130	C Hazen-Williams	0			39, 66	Cylindrical Pipe	User Specified			None
77	Pipe	Yes	52	feet	60	inches	Unspecified	130	C Hazen-Williams	0.54			248, 67	Cylindrical Pipe	User Specified			None
78	Pipe	Yes	0.1	feet	60	inches	Unspecified	130	C Hazen-Williams	0			39, 68	Cylindrical Pipe	User Specified			None
79	Pipe	Yes	52	feet	60	inches	Unspecified	130	C Hazen-Williams	0.54			250, 69	Cylindrical Pipe	User Specified			None
81	Pipe	Yes	22	feet	144	inches	Unspecified	130	C Hazen-Williams	0			43, 41	Cylindrical Pipe	User Specified			None
82	Pipe	Yes	22	feet	144	inches	Unspecified	130	C Hazen-Williams	0			45, 43	Cylindrical Pipe	User Specified			None
83	Pipe	Yes	22	feet	144	inches	Unspecified	130	C Hazen-Williams	0			47, 45	Cylindrical Pipe	User Specified			None
84	Pipe	Yes	22	feet	144	inches	Unspecified	130	C Hazen-Williams	0			49, 47	Cylindrical Pipe	User Specified			None
85	Pipe	Yes	22	feet	144	inches	Unspecified	130	C Hazen-Williams	0			57, 49	Cylindrical Pipe	User Specified			None
86	Pipe	Yes	13	feet	144	inches	Unspecified	130	C Hazen-Williams	0			51, 57	Cylindrical Pipe	User Specified			None
87	Pipe	Yes	22	feet	144	inches	Unspecified	130	C Hazen-Williams	0			61, 59	Cylindrical Pipe	User Specified			None
88	Pipe	Yes	22	feet	144	inches	Unspecified	130	C Hazen-Williams	0			63, 61	Cylindrical Pipe	User Specified			None
89	Pipe	Yes	22	feet	144	inches	Unspecified	130	C Hazen-Williams	0			65, 63	Cylindrical Pipe	User Specified			None
90	Pipe	Yes	22	feet	144	inches	Unspecified	130	C Hazen-Williams	0			67, 65	Cylindrical Pipe	User Specified			None
91	Pipe	Yes	22	feet	144	inches	Unspecified	130	C Hazen-Williams	0			69, 67	Cylindrical Pipe	User Specified			None
92	Pipe	Yes	13	feet	144	inches	Unspecified	130	C Hazen-Williams	0			59, 55	Cylindrical Pipe	User Specified			None
93	Pipe	Yes	49.5	feet	144	inches	Unspecified	130	C Hazen-Williams	0			70, 69	Cylindrical Pipe	User Specified			None
98	Pipe	Yes	1096	feet	144	inches	Unspecified	130	C Hazen-Williams	0			55, 84	Cylindrical Pipe	User Specified			None
99	Pipe	Yes	1	feet	144	inches	Unspecified	130	C Hazen-Williams	0			77, 80	Cylindrical Pipe	User Specified			None
100	Pipe	Yes	37	feet	144	inches	Unspecified	130	C Hazen-Williams	0			78, 77	Cylindrical Pipe	User Specified			None
101	Pipe	Yes	1	feet	60	inches	Unspecified	130	C Hazen-Williams	0			78, 81	Cylindrical Pipe	User Specified			None
102	Pipe	Yes	1078	feet	144	inches	Unspecified	130	C Hazen-Williams	0			77, 79	Cylindrical Pipe	User Specified			None
103	Pipe	Yes	70	feet	144	inches	Unspecified	130	C Hazen-Williams	0			41, 78	Cylindrical Pipe	User Specified			None
104	Pipe	Yes	1	feet	144	inches	Unspecified	130	C Hazen-Williams	0			82, 17	Cylindrical Pipe	User Specified			None
105	Pipe	Yes	0.1	feet	60	inches	Unspecified	130	C Hazen-Williams	0			83, 51	Cylindrical Pipe	User Specified			None

Pipe	Name	Pipe Defined	Length	Length Units	Hydraulic Diameter	Hydraulic Diam. Units	Friction Data Set	Roughness	Roughness Units	Losses (K)	Initial Flow	Initial Flow Units	Junctions (Up,Down)	Geometry	Material	Size	Type	Special Condition
106	Pipe	Yes	771	feet	144	inches	Unspecified	130	C Hazen-Williams	0.1752876			200, 201	Cylindrical Pipe	User Specified			None
107	Pipe	Yes	732	feet	144	inches	Unspecified	130	C Hazen-Williams	0.3505752			201, 202	Cylindrical Pipe	User Specified			None
108	Pipe	Yes	265	feet	144	inches	Unspecified	130	C Hazen-Williams	0			202, 203	Cylindrical Pipe	User Specified			None
109	Pipe	Yes	334	feet	144	inches	Unspecified	130	C Hazen-Williams	0.1537168			203, 204	Cylindrical Pipe	User Specified			None
110	Pipe	Yes	781	feet	144	inches	Unspecified	130	C Hazen-Williams	0.1752876			204, 205	Cylindrical Pipe	User Specified			None
111	Pipe	Yes	1884	feet	144	inches	Unspecified	130	C Hazen-Williams	0.4827213			205, 257	Cylindrical Pipe	User Specified			None
112	Pipe	Yes	9	feet	60	inches	Unspecified	130	C Hazen-Williams	0			2, 100	Cylindrical Pipe	User Specified			None
113	Pipe	Yes	6.5	feet	60	inches	Unspecified	130	C Hazen-Williams	0			100, 101	Cylindrical Pipe	User Specified			None
114	Pipe	Yes	36	feet	60	inches	Unspecified	130	C Hazen-Williams	0.708			103, 5	Cylindrical Pipe	User Specified			None
115	Pipe	Yes	9	feet	60	inches	Unspecified	130	C Hazen-Williams	0			4, 102	Cylindrical Pipe	User Specified			None
116	Pipe	Yes	6.5	feet	60	inches	Unspecified	130	C Hazen-Williams	0			102, 103	Cylindrical Pipe	User Specified			None
117	Pipe	Yes	36	feet	60	inches	Unspecified	130	C Hazen-Williams	0.708			105, 7	Cylindrical Pipe	User Specified			None
118	Pipe	Yes	9	feet	60	inches	Unspecified	130	C Hazen-Williams	0			6, 104	Cylindrical Pipe	User Specified			None
119	Pipe	Yes	6.5	feet	60	inches	Unspecified	130	C Hazen-Williams	0			104, 105	Cylindrical Pipe	User Specified			None
120	Pipe	Yes	36	feet	60	inches	Unspecified	130	C Hazen-Williams	0.708			107, 9	Cylindrical Pipe	User Specified			None
121	Pipe	Yes	9	feet	60	inches	Unspecified	130	C Hazen-Williams	0			8, 106	Cylindrical Pipe	User Specified			None
122	Pipe	Yes	6.5	feet	60	inches	Unspecified	130	C Hazen-Williams	0			106, 107	Cylindrical Pipe	User Specified			None
123	Pipe	Yes	36	feet	60	inches	Unspecified	130	C Hazen-Williams	0.708			109, 11	Cylindrical Pipe	User Specified			None
124	Pipe	Yes	9	feet	60	inches	Unspecified	130	C Hazen-Williams	0			10, 108	Cylindrical Pipe	User Specified			None
125	Pipe	Yes	6.5	feet	60	inches	Unspecified	130	C Hazen-Williams	0			108, 109	Cylindrical Pipe	User Specified			None
126	Pipe	Yes	36	feet	60	inches	Unspecified	130	C Hazen-Williams	0.708			111, 19	Cylindrical Pipe	User Specified			None
127	Pipe	Yes	9	feet	60	inches	Unspecified	130	C Hazen-Williams	0			18, 110	Cylindrical Pipe	User Specified			None
128	Pipe	Yes	6.5	feet	60	inches	Unspecified	130	C Hazen-Williams	0			110, 111	Cylindrical Pipe	User Specified			None
129	Pipe	Yes	9	feet	60	inches	Unspecified	130	C Hazen-Williams	0			20, 152	Cylindrical Pipe	User Specified			None
130	Pipe	Yes	6.5	feet	60	inches	Unspecified	130	C Hazen-Williams	0			152, 153	Cylindrical Pipe	User Specified			None
131	Pipe	Yes	9	feet	60	inches	Unspecified	130	C Hazen-Williams	0			22, 154	Cylindrical Pipe	User Specified			None
132	Pipe	Yes	6.5	feet	60	inches	Unspecified	130	C Hazen-Williams	0			154, 155	Cylindrical Pipe	User Specified			None
133	Pipe	Yes	9	feet	60	inches	Unspecified	130	C Hazen-Williams	0			24, 156	Cylindrical Pipe	User Specified			None
134	Pipe	Yes	6.5	feet	60	inches	Unspecified	130	C Hazen-Williams	0			156, 157	Cylindrical Pipe	User Specified			None
135	Pipe	Yes	9	feet	60	inches	Unspecified	130	C Hazen-Williams	0			26, 158	Cylindrical Pipe	User Specified			None
136	Pipe	Yes	6.5	feet	60	inches	Unspecified	130	C Hazen-Williams	0			158, 159	Cylindrical Pipe	User Specified			None
137	Pipe	Yes	9	feet	60	inches	Unspecified	130	C Hazen-Williams	0			28, 160	Cylindrical Pipe	User Specified			None
138	Pipe	Yes	6.5	feet	60	inches	Unspecified	130	C Hazen-Williams	0			160, 161	Cylindrical Pipe	User Specified			None
139	Pipe	Yes	9	feet	60	inches	Unspecified	130	C Hazen-Williams	0			30, 162	Cylindrical Pipe	User Specified			None
140	Pipe	Yes	6.5	feet	60	inches	Unspecified	130	C Hazen-Williams	0			162, 163	Cylindrical Pipe	User Specified			None
141	Pipe	Yes	8	feet	60	inches	Unspecified	130	C Hazen-Williams	0			12, 150	Cylindrical Pipe	User Specified			None
142	Pipe	Yes	2	feet	60	inches	Unspecified	130	C Hazen-Williams	0			15, 113	Cylindrical Pipe	User Specified			Closed
143	Pipe	Yes	2	feet	60	inches	Unspecified	130	C Hazen-Williams	0			16, 151	Cylindrical Pipe	User Specified			None
144	Pipe	Yes	334	feet	144	inches	Unspecified	130	C Hazen-Williams	0.1537168			210, 211	Cylindrical Pipe	User Specified			None
145	Pipe	Yes	781	feet	144	inches	Unspecified	130	C Hazen-Williams	0.1752876			211, 212	Cylindrical Pipe	User Specified			None
146	Pipe	Yes	1884	feet	144	inches	Unspecified	130	C Hazen-Williams	0.4827213			212, 260	Cylindrical Pipe	User Specified			None
147	Pipe	Yes	771	feet	144	inches	Unspecified	130	C Hazen-Williams	0.1752876			207, 208	Cylindrical Pipe	User Specified			None
148	Pipe	Yes	732	feet	144	inches	Unspecified	130	C Hazen-Williams	0.3505752			208, 209	Cylindrical Pipe	User Specified			None
149	Pipe	Yes	265	feet	144	inches	Unspecified	130	C Hazen-Williams	0			209, 210	Cylindrical Pipe	User Specified			None
150	Pipe	Yes	18233	feet	144	inches	Unspecified	130	C Hazen-Williams	2.610465			79, 207	Cylindrical Pipe	User Specified			None
151	Pipe	Yes	334	feet	144	inches	Unspecified	130	C Hazen-Williams	0.1537168			217, 218	Cylindrical Pipe	User Specified			None
152	Pipe	Yes	781	feet	144	inches	Unspecified	130	C Hazen-Williams	0.1752876			218, 219	Cylindrical Pipe	User Specified			None

Pipe	Name	Pipe Defined	Length	Length Units	Hydraulic Diameter	Hydraulic Diam. Units	Friction Data Set	Roughness	Roughness Units	Losses (K)	Initial Flow	Initial Flow Units	Junctions (Up,Down)	Geometry	Material	Size	Type	Special Condition
153	Pipe	Yes	1884	feet	144	inches	Unspecified	130	C Hazen-Williams	0.4827213			219, 258	Cylindrical Pipe	User Specified			None
154	Pipe	Yes	771	feet	144	inches	Unspecified	130	C Hazen-Williams	0.1752876			214, 215	Cylindrical Pipe	User Specified			None
155	Pipe	Yes	732	feet	144	inches	Unspecified	130	C Hazen-Williams	0.3505752			215, 216	Cylindrical Pipe	User Specified			None
156	Pipe	Yes	265	feet	144	inches	Unspecified	130	C Hazen-Williams	0			216, 217	Cylindrical Pipe	User Specified			None
157	Pipe	Yes	18233	feet	144	inches	Unspecified	130	C Hazen-Williams	2.610465			84, 214	Cylindrical Pipe	User Specified			None
158	Pipe	Yes	334	feet	144	inches	Unspecified	130	C Hazen-Williams	0.1537168			223, 224	Cylindrical Pipe	User Specified			None
159	Pipe	Yes	781	feet	144	inches	Unspecified	130	C Hazen-Williams	0.1752876			224, 225	Cylindrical Pipe	User Specified			None
160	Pipe	Yes	1884	feet	144	inches	Unspecified	130	C Hazen-Williams	0.4827213			225, 259	Cylindrical Pipe	User Specified			None
161	Pipe	Yes	771	feet	144	inches	Unspecified	130	C Hazen-Williams	0.1752876			38, 221	Cylindrical Pipe	User Specified			None
162	Pipe	Yes	732	feet	144	inches	Unspecified	130	C Hazen-Williams	0.3505752			221, 222	Cylindrical Pipe	User Specified			None
163	Pipe	Yes	265	feet	144	inches	Unspecified	130	C Hazen-Williams	0			222, 223	Cylindrical Pipe	User Specified			None
164	Pipe	Yes	9	feet	60	inches	Unspecified	130	C Hazen-Williams	0			40, 227	Cylindrical Pipe	User Specified			None
165	Pipe	Yes	6.5	feet	60	inches	Unspecified	130	C Hazen-Williams	0			227, 228	Cylindrical Pipe	User Specified			None
166	Pipe	Yes	9	feet	60	inches	Unspecified	130	C Hazen-Williams	0			42, 229	Cylindrical Pipe	User Specified			None
167	Pipe	Yes	6.5	feet	60	inches	Unspecified	130	C Hazen-Williams	0			229, 230	Cylindrical Pipe	User Specified			None
168	Pipe	Yes	9	feet	60	inches	Unspecified	130	C Hazen-Williams	0			44, 231	Cylindrical Pipe	User Specified			None
169	Pipe	Yes	6.5	feet	60	inches	Unspecified	130	C Hazen-Williams	0			231, 232	Cylindrical Pipe	User Specified			None
170	Pipe	Yes	9	feet	60	inches	Unspecified	130	C Hazen-Williams	0			46, 233	Cylindrical Pipe	User Specified			None
171	Pipe	Yes	6.5	feet	60	inches	Unspecified	130	C Hazen-Williams	0			233, 234	Cylindrical Pipe	User Specified			None
172	Pipe	Yes	9	feet	60	inches	Unspecified	130	C Hazen-Williams	0			48, 235	Cylindrical Pipe	User Specified			None
173	Pipe	Yes	6.5	feet	60	inches	Unspecified	130	C Hazen-Williams	0			235, 236	Cylindrical Pipe	User Specified			None
174	Pipe	Yes	9	feet	60	inches	Unspecified	130	C Hazen-Williams	0			56, 237	Cylindrical Pipe	User Specified			None
175	Pipe	Yes	6.5	feet	60	inches	Unspecified	130	C Hazen-Williams	0			237, 238	Cylindrical Pipe	User Specified			None
176	Pipe	Yes	9	feet	60	inches	Unspecified	130	C Hazen-Williams	0			58, 239	Cylindrical Pipe	User Specified			None
177	Pipe	Yes	6.5	feet	60	inches	Unspecified	130	C Hazen-Williams	0			239, 240	Cylindrical Pipe	User Specified			None
178	Pipe	Yes	9	feet	60	inches	Unspecified	130	C Hazen-Williams	0			60, 241	Cylindrical Pipe	User Specified			None
179	Pipe	Yes	6.5	feet	60	inches	Unspecified	130	C Hazen-Williams	0			241, 242	Cylindrical Pipe	User Specified			None
180	Pipe	Yes	9	feet	60	inches	Unspecified	130	C Hazen-Williams	0			62, 243	Cylindrical Pipe	User Specified			None
181	Pipe	Yes	6.5	feet	60	inches	Unspecified	130	C Hazen-Williams	0			243, 244	Cylindrical Pipe	User Specified			None
182	Pipe	Yes	9	feet	60	inches	Unspecified	130	C Hazen-Williams	0			64, 245	Cylindrical Pipe	User Specified			None
183	Pipe	Yes	6.5	feet	60	inches	Unspecified	130	C Hazen-Williams	0			245, 246	Cylindrical Pipe	User Specified			None
184	Pipe	Yes	9	feet	60	inches	Unspecified	130	C Hazen-Williams	0			66, 247	Cylindrical Pipe	User Specified			None
185	Pipe	Yes	6.5	feet	60	inches	Unspecified	130	C Hazen-Williams	0			247, 248	Cylindrical Pipe	User Specified			None
186	Pipe	Yes	9	feet	60	inches	Unspecified	130	C Hazen-Williams	0			68, 249	Cylindrical Pipe	User Specified			None
187	Pipe	Yes	6.5	feet	60	inches	Unspecified	130	C Hazen-Williams	0			249, 250	Cylindrical Pipe	User Specified			None
188	Pipe	Yes	6	feet	60	inches	Unspecified	130	C Hazen-Williams	0.24			254, 251	Cylindrical Pipe	User Specified			Closed
189	Pipe	Yes	9.5	feet	60	inches	Unspecified	130	C Hazen-Williams	0.3			251, 252	Cylindrical Pipe	User Specified			Closed
190	Pipe	Yes	9.5	feet	60	inches	Unspecified	130	C Hazen-Williams	0			251, 253	Cylindrical Pipe	User Specified			Closed
191	Pipe	Yes	8	feet	60	inches	Unspecified	130	C Hazen-Williams	0			50, 254	Cylindrical Pipe	User Specified			None
192	Pipe	Yes	2	feet	60	inches	Unspecified	130	C Hazen-Williams	0			252, 255	Cylindrical Pipe	User Specified			Closed
193	Pipe	Yes	2	feet	60	inches	Unspecified	130	C Hazen-Williams	0			253, 256	Cylindrical Pipe	User Specified			Closed
194	Pipe	Yes	50	feet	144	inches	Unspecified	130	C Hazen-Williams	0.3			259, 261	Cylindrical Pipe	User Specified			None
195	Pipe	Yes	50	feet	144	inches	Unspecified	130	C Hazen-Williams	0.3			257, 261	Cylindrical Pipe	User Specified			None
196	Pipe	Yes	3400	feet	23	feet	Unspecified	130	C Hazen-Williams	0			261, 267	Cylindrical Pipe	User Specified			None
197	Pipe	Yes	50	feet	144	inches	Unspecified	130	C Hazen-Williams	0.3			260, 262	Cylindrical Pipe	User Specified			None
198	Pipe	Yes	50	feet	144	inches	Unspecified	130	C Hazen-Williams	0.3			258, 262	Cylindrical Pipe	User Specified			None
199	Pipe	Yes	3400	feet	23	feet	Unspecified	130	C Hazen-Williams	0			262, 266	Cylindrical Pipe	User Specified			None

Pipe	Name	Pipe Defined	Length	Length Units	Hydraulic Diameter	Hydraulic Diam. Units	Friction Data Set	Roughness	Roughness Units	Losses (K)	Initial Flow	Initial Flow Units	Junctions (Up,Down)	Geometry	Material	Size	Type	Special Condition
200	Pipe	Yes	0.1	feet	23	feet	Unspecified	130	C Hazen-Williams	1			267, 268	Cylindrical Pipe	User Specified			None
201	Pipe	Yes	0.1	feet	23	feet	Unspecified	130	C Hazen-Williams	1			266, 268	Cylindrical Pipe	User Specified			None
202	Pipe	Yes	251.5	feet	40	feet	Unspecified	130	C Hazen-Williams	1			268, 265	Cylindrical Pipe	User Specified			None

Reservoir Table

Reservoir	Name	Object Defined	Inlet Elevation	Elevation Units	Initial Pressure	Initial Pressure Units	Database Source	Liquid Elev.	Liquid Elev. Units	Surface Pressure	Surface Pressure Units	Balance Energy	Balance Concentration	(Pipe #1) K In, K Out	(Pipe #2) K In, K Out	(Pipe #3) K In, K Out	(Pipe #4) K In, K Out	(Pipe #5) K In, K Out
1	Reservoir	Yes						205	feet	0	psig	No	No	(P1) 0, 0	(P3) 0, 0	(P5) 0, 0	(P7) 0, 0	(P9) 0, 0
39	Reservoir	Yes						119	feet	0	psig	No	No	(P50) 0, 0	(P52) 0, 0	(P54) 0, 0	(P56) 0, 0	(P58) 0, 0
265	INLET/OUTLET TOWER	Yes						340	feet	0	psig	No	No	(P202) 0, 0				

Reservoir	(Pipe #6) K In, K Out	(Pipe #7) K In, K Out	(Pipe #8) K In, K Out	(Pipe #9) K In, K Out	(Pipe #10) K In, K Out	(Pipe #11) K In, K Out	(Pipe #12) K In, K Out	(Pipe #13) K In, K Out	(Pipe #14) K In, K Out	(Pipe #15) K In, K Out	(Pipe #16) K In, K Out	(Pipe #17) K In, K Out	(Pipe #18) K In, K Out	(Pipe #19) K In, K Out	(Pipe #20) K In, K Out	(Pipe #21) K In, K Out	(Pipe #22) K In, K Out	(Pipe #23) K In, K Out	(Pipe #24) K In, K Out	(Pipe #25) K In, K Out
1	(P11) 0, 0	(P17) 0, 0	(P19) 0, 0	(P21) 0, 0	(P23) 0, 0	(P25) 0, 0	(P27) 0, 0	(P29) 0, 0												
39	(P60) 0, 0	(P66) 0, 0	(P68) 0, 0	(P70) 0, 0	(P72) 0, 0	(P74) 0, 0	(P76) 0, 0	(P78) 0, 0												
265																				

Reservoir	(Pipe #1) Depth	(Pipe #2) Depth	(Pipe #3) Depth	(Pipe #4) Depth	(Pipe #5) Depth	(Pipe #6) Depth	(Pipe #7) Depth	(Pipe #8) Depth	(Pipe #9) Depth	(Pipe #10) Depth	(Pipe #11) Depth	(Pipe #12) Depth	(Pipe #13) Depth	(Pipe #14) Depth	(Pipe #15) Depth	(Pipe #16) Depth	(Pipe #17) Depth	(Pipe #18) Depth	(Pipe #19) Depth	(Pipe #20) Depth
1	(P1) 179.6	(P3) 179.6	(P5) 179.6	(P7) 179.6	(P9) 179.6	(P11) 179.6	(P17) 179.6	(P19) 179.6	(P21) 179.6	(P23) 179.6	(P25) 179.6	(P27) 179.6	(P29) 179.6							
39	(P50) 89.9	(P52) 89.9	(P54) 89.9	(P56) 89.9	(P58) 89.9	(P60) 89.9	(P66) 89.9	(P68) 89.9	(P70) 89.9	(P72) 89.9	(P74) 89.9	(P76) 89.9	(P78) 89.9							
265	(P202) 300																			

Reservoir	(Pipe #21) Depth	(Pipe #22) Depth	(Pipe #23) Depth	(Pipe #24) Depth	(Pipe #25) Depth	Pipe Depth Units
1						feet
39						feet
265						feet

Appendix D
Harrington Pipeline Alignment Analysis

Harrington Pipeline Alignment Analysis
Technical Memorandum (Final)

To:	Henry Luu/HDR
CC:	
Date:	August 28, 2020
From:	Jeff Smith/Jacobs
Quality Review by:	Brad Memeo/Jacobs
Authority Agent Review by:	TBD
Subject:	Analysis of Harrington Pipeline Route

1.0 Background

The Sites Project Authority (Authority) adopted the recommended project (VP7) as provided in the *Sites Project Value Planning Alternatives Appraisal Report,* dated April 2020, to reduce the program cost from $5.2 billion to $3.0 billion. One of the new conveyance components of VP7, uses the Tehama-Colusa Canal (TCC) to convey water from Funks Reservoir, approximately 40 miles south, to near the end of the TCC. At this point, a new discharge outlet and pipeline would convey water for discharge to either the Colusa Basin Drain (CBD) or the Sacramento River. Since the discharge point is near the end of the TCC, close to Dunnigan, this pipeline has been referred to as the "Dunnigan pipeline." The Dunnigan pipeline is a 4-mile-long, 9-foot-diameter pipe to the CBD, or a 10-mile-long, 10.5-foot-diameter pipeline if it flows to the Sacramento River.

2.0 Purpose

Recently, the Authority asked the Conveyance Team to investigate the possibility of using an alternative alignment to the Dunnigan pipeline alignment. This alternative alignment, called the Harrington alignment, is parallel and approximately 9 miles north of the Dunnigan alignment. The Harrington alignment is associated with an existing main pipeline used by Colusa County Water District (CCWD). This potential alignment would either use the existing CCWD pipeline's unused capacity and/or construct a parallel pipe to convey the 1,000 cubic feet per second (cfs) from the TCC to the CBD using, to the extent possible, CCWD's existing right-of-way. If the Harrington alignment has merit, then further analysis would be completed to take the pipeline to the Sacramento River.

3.0 Analysis

Information regarding the existing pipeline was obtained from CCWD's General Manager, Shelly Murphy, and other sources. This information included the following:

- Parcel lines
- Existing pipeline as-built drawings
- Pipeline flow of 125 cfs peak design capacity

- Water surface elevations:
  - TCC – 180 feet
  - CBD – 40 feet

3.1 Alignment and Sizing

The as-built drawing for the main pipeline (Lateral 2A) shows it begins at the TCC and goes directly east for 5 miles to West Road, where it discharges to a canal that flows north. Lateral 2A is aligned along the southern side of White Road in an east-west direction, but ends about 1.25 miles short of the CBD, where it crosses under White Road and then discharges into the canal.

As it leaves the TCC, Lateral 2A consists of 1 mile of 60-inch-diameter pipe, followed by 3 miles of 54-inch-diameter pipe, and a final last mile of 48-inch-diameter pipe. The pipe was installed in 1965 and consist of reinforced concrete pipe (60-inch diameter) and concrete cylinder pipe (54- and 48-inch diameter).

Figure 1 shows the approximate location of Lateral 2A in relation to the TCC and CBD.

3.2 Flow Calculations

Calculations were completed to determine: (1) if there was unused capacity in Lateral 2A; and (2) new pipeline diameter required to convey 1,000 cfs to the 6.25 miles from the TCC to CBD.

3.2.1 CCWD Lateral 2A

Based on information provided by Jeff Sutton (TCC Authority General Manager), the turnout on the TCC to CCWD Lateral 2A is designed for a maximum capacity of 125 cfs. Actual design flows of the lateral was not available, but a hydraulic analysis of Lateral 2A in the initial mile of 60-inch-diameter pipe indicates it can accommodate a maximum flow of about 210 cfs under gravity flow condition with approximate known head conditions. There is not enough information to determine the available capacity of the downstream 54- and 48-inch-diameter pipes because lateral demand flows are unknown. Regardless, of the capacity of the downstream smaller pipes, the roughly 210 cfs calculated for the 60-inch pipe would be the maximum this lateral could convey. It could be less if the downstream pipes have further constraints. This analysis shows that there may be some additional capacity in lateral 2A of about 85 cfs (210 cfs calculated – 125 cfs current turnout limitation) in the 60-inch-diameter pipe, but this is only a fraction of the 1,000 cfs needed to convey the Sites Project water to the CBD. Therefore, it was determined that using the existing CCWD Lateral 2A was not practical because a new, large-diameter pipe is required regardless.

3.2.2 New Pipeline

Hydraulic calculations were preformed to determine the pipeline size needed for a new pipeline for 6.25 miles, from the TCC to the CBD, using a parallel alignment to Lateral 2A. The location of this alternative pipeline alignment is shown on Figure 2. Following are the criteria used for to calculate the pipe diameter:

- The water surface elevation of the upper end at TCC is about 180 feet
- The downstream end of the proposed pipeline at the CBD is roughly 40 feet.
- Hazen-Williams C-value of 130

Results of this analysis indicates a roughly 9.5-foot-diameter pipeline would be required using gravity flow. This results in a velocity of about 14 feet per second, which is higher than the normal 7 feet per second. However, since this pipe is gravity flow, the approach to sizing the pipe was to make the pipe as small as possible while using all the available driving head differential. Since an energy dissipater would be required at the end of the pipe at the CBD, flowing at this velocity was not a concern.

3.3 Utilizing Existing Lateral 2A Right-of-Way for New Pipe Installation

One of the reasons for studying this potential alignment for a new discharge pipeline was to take advantage of using the existing right-of-way for Lateral 2A for a shared installation of the new pipeline. This analysis used the as-built drawing information to determine:

- The overall right-of-way width and location of the existing Lateral 2A within the right-of-way
- If there is enough space to install the new 9.5-foot-diameter pipeline
- The location of the right-of-way with respect to White Road and whether encroachments have occurred within this right-of-way since Lateral 2A was constructed 55 years ago

3.3.1 Right-of-Way Width

Analysis of the Lateral 2A as-built drawings showed the width of the right-of-way varies from 70 to 90 feet. The general location of the pipe within the right-of-way is 40 to 50 feet north of the southern line of right-of way. this would leave about 20 to 50 feet of room on each side of new pipeline alignment for installation. This is a very narrow corridor to install the 9.5-foot-diameter pipe, but the space is possible, assuming a vertical trench wall would be possible (at a higher cost than laying back) and an additional temporary construction easement of about 50 feet can be obtained.

3.3.2 Encroachments in Existing Right-of-Way

Parcel line information was obtained from the Real Estate Team and overlaid with Google Earth to assist in determining where the existing pipe may be located. The parcel information did not correlate well to roads and other features shown in Google Earth, especially the last 2 miles along White Road. The presumed White Road right-of-way lines were shown south of the road in the orchard and did not include any of the physical road.

The Google Earth image did seem to indicate a corridor and a few features that help to roughly locate the existing pipeline, but this was not clearly definitive. What the image did show is that orchards have encroached within the existing pipeline right-of-way, especially on the section between the TCC and Grieve Road (3 miles). In this segment, there is a farm access road where the existing pipeline is likely located, but the distance between the orchard and this road is only about 30 to 40 feet. In other words, there are mature trees currently located within the existing pipeline right-of-way, given the right-of-way is 80 to 90 feet wide in this segment.

In the other 2-mile segment, between Grieve Road and the end of Lateral 2A, the existing pipeline parallels White Road and is located about 40 to 45 feet south of the road centerline. This places the existing pipeline roughly in the farm road adjacent to the orchard. The space between the southern edge of road and the existing pipeline contains power poles and a buried communication cable that could interfere with using as a work area for construction of a new pipeline.

The Jacobs team also looked at placing the new pipeline in White Road, but determined this would also be challenging because of a narrow road width that is often bordered by ditches or other features on both sides. The work area within the road is approximately 50 feet at best. Additionally, there are numerous turnouts that cross the road that would result in a 16-18-foot-deep trench to avoid the lateral crossings.

4.0 Comparison

An analysis of the existing right-of-way and pipeline corridor indicate that there is insufficient space available to install the new pipeline without requiring removal of orchards. A rough approximation of the area of orchards to be removed to accommodate construction is 90 acres (assuming 150 feet of easements, which includes removing trees in the existing right-of-way, plus a temporary construction easement). The total width of work area required for construction is about 200 feet, assuming some layback area for the deep trench; which is roughly the same as anticipated for the Dunnigan pipeline. Use of a vertical trench may only require about 125 feet of work area, but maintaining a deep vertical trench in these wet soils (because of high groundwater) is expected to be almost impossible.

Another consideration associated with this alignment includes discharging to the CBD roughly 8 miles upstream of the proposed Dunnigan Pipeline discharge point , which may result in additional losses resulting from seepage and other possible water losses. In other words, more than 1,000 cfs of flow may be required to ensure 1,000 cfs ends up in the Sacramento River. This is also true for the Dunnigan Pipeline, but fewer losses are expected with the Dunnigan pipeline because the length of conveyance in the CBD is shorter by about 8 miles (10 miles versus 18 miles).

Installation of a pipeline to the CBD for this alignment requires 6.25 miles, versus about 4 miles for the Dunnigan pipeline, from the TCC to the CBD. This pipeline requires a 9.5-foot-diameter pipe, versus the 9-foot-diameter pipe anticipated for Dunnigan. Although the Dunnigan Pipeline is significantly shorter, there is less head differential available to convey the 1,000 cfs. Both the Harrington and Dunnigan pipelines require tunneling under I-5, Old Hwy 99, and Union Pacific Railroad tracks.

A Class 5 cost estimate was prepared for both the Harrington and Dunnigan pipelines. The expected accuracy ranges for this class estimate are –20 to –50 percent on the low side, and +30 to +100 percent on the high side. This estimate includes a contractor's overhead and profit, a 10 percent contingency, and 17 percent for soft costs (administrative, design, construction management). It does not include any costs for real estate acquisition. Estimate costs are as follows:

Construction Cost for Dunnigan Pipeline to Colusa Basin Drain = $64.5 million

Construction Cost for Harrington Pipeline to Colusa Basin Drain = $112.4 million

The comparison of construction costs shows the Harrington pipeline to be almost twice the cost of the Dunnigan pipeline. This is explained given the Dunnigan Pipeline is much shorter and a slightly smaller diameter pipeline. Although land acquisition costs are not included in this construction cost, the Harrington pipeline will likely require removal of approximately 90 acres of orchards, while the Dunnigan pipeline is anticipated to require removal of roughly 40 acres of orchards and vineyards. Therefore, the cost differential is expected to increase further if land acquisition costs are included in the comparison.

5.0 Recommendation

Based on the analysis presented in this technical memorandum, we recommend using the Dunnigan pipeline alignment to convey water from the TCC to the CBD. The Harrington alignment does not warrant further study.

Figures

FIGURE 1
Existing CCWD Pipeline

FIGURE 2
Harrington Alternate

Appendix E
Emergency Drawdown Facilities Evaluation

Emergency Drawdown Facilities Evaluation Technical Memorandum (Final)

To:	Henry Luu
CC:	Michael Forest/AECOM, Jeff Herrin/AECOM
Date:	August 28, 2020
From:	Jeff Smith/Jacobs
Quality Review by:	Peter Rude/Jacobs
Authority Agent Review by:	TBD
Subject:	Site Reservoir Emergency Drawdown Facilities Requirements and Alternatives

1.0 Background and Purpose

The Sites Joint Powers Authority (Authority) has embarked on the implementation of a 1.5-million-acre-foot reservoir, known as the Sites Reservoir. Other major facilities include two pump generating plants, two smaller regulating reservoirs, and several miles of 12-foot-diameter pipelines used to pump water from the Tehama Colusa Canal (TCC) and Glenn-Colusa Irrigation District (GCID) Canal to and from the Sites Reservoir. The Funks Reservoir is located on the TCC and serves currently as a regulating reservoir, and the Terminal Regulating Reservoir (TRR) will be a new regulating reservoir on the GCID Canal.

The Sites Reservoir will require a procedure to provide emergency drawdown, as described in more detail in a technical memorandum (TM) entitled “Funks and Stone Corral Creeks - Reservoir Operating Elevations and Emergency Release Management,” prepared by AECOM and dated May 27, 2020. Results provided in this TM show that about 16,000 cubic feet per second (cfs) of flow will need to be discharge through the inlet/outlet tunnels that ultimately are connected to Funks and TRR reservoirs, as well as Funks Creek. How this 16,000 cfs flow will be distributed is not provided, but the flow is assumed to be able to be conveyed to Funks Reservoir, the TRR, and ultimately Funks Creek. Some flow may also be sent into the TCC and GCID Canal for dispersion away from the project site.

The purpose of this TM is to provide calculations showing how much flow the proposed pipelines connected to the Funks Reservoir and TRR can accommodate during an emergency drawdown condition.

2.0 Flow Calculations

The Site Reservoir inlet/outlet (I/O) tunnel consists of two 23-foot diameter penstocks that end at the foot of Sites Reservoir. It is proposed to connect the I/O tunnel to both the Funks Reservoir and the TRR. These connections are made using two 12-foot-diameter pipelines for each reservoir. At each reservoir, the pipelines are connected to a pumping/generating plant (PGP) that pumps water from the regulating reservoir to Sites Reservoir, as well as turbines that will generate power when flow is released from Sites Reservoir. There will also be energy-dissipation equipment, such as a fixed cone valve(s), adjacent to each PGP to throttle the flow of water into each regulating reservoir when the turbines are not being used.

For the emergency drawdown condition, calculations were performed to determine the maximum flow that can be conveyed through the 12-foot-diameter pipes to each regulating reservoir using the fixed-cone valves. Flow through these pipes will be based on gravity flow.

2.1 Funks Regulating Reservoir

Following are design criteria used to perform flow calculations:

- Sites Reservoir Levels
  - Maximum = 498 feet
  - 10% drawdown level = 478 feet
- Pipeline
  - Two 12-foot internal diameter
  - Length = 6,000 feet
  - Hazen-Williams C-factor = 120
  - Maximum velocity = 40 feet per second
- Energy-dissipation Valve Elevation = 215 feet

Based on this information, calculations show that there is more than enough water surface elevation differential to provide a high volume of flow during the drawdown condition. Specifically, there is enough head to achieve a flow of 12,500 cfs through the two pipes, but velocities in these pipes would be around 55 feet per second. At the upper limit of 40 feet per second, the flow would be about 9,000 cfs, or roughly about 56% of the total drawdown flow.

We understand the U.S. Bureau of Reclamation (Reclamation) restricts the maximum allowable velocity in a pipeline to 20 feet per second. If this criterion was used, then the maximum flow through the two pipelines would be 4,500 cfs, or roughly 23% of the total drawdown flow.

2.2 Terminal Regulating Reservoir

Following are design criteria used to perform flow calculations:

- Sites Reservoir Levels
  - Maximum = 498 feet
  - 10% drawdown level = 478 feet
- Pipeline
  - Two 12-foot internal diameter
  - Length = 25,000 feet
  - Hazen-Williams C-factor = 120
  - Maximum velocity = 40 feet per second
  - Energy-dissipation Valve Elevation = 130 feet

Based on this information, calculations show more than enough water surface elevation differential to provide a high volume of flow during the drawdown condition. Because of the higher friction losses associated with the longer pipes, this system could achieve a flow of about 7,000 cfs through the two pipes, resulting in a velocity of about 30 feet per second. The flow of 7,000 cfs is roughly about 44% of the total drawdown flow.

Using Reclamation's design criteria of 20 feet per second, the maximum flow through the two pipelines would be 4,500 cfs, or roughly 23% of the total drawdown flow.

3.0 Discussion of Results

Based on the calculations performed as part of this analysis, using the proposed pipelines to carry flow during an emergency drawdown condition could achieve the entire flow of 16,000 cfs, with 9,000 being discharged to Funks Reservoir and 7,000 cfs to the new TRR. This is all predicated on allowing a maximum velocity of 40 feet per second in the pipelines and both reservoirs accommodating these flows.

Funks Reservoir has a spillway that can accommodate a flow of 22,000 cfs or more than the total emergency drawdown flow of 16,000 cfs. The TRR is lower in the system and is not anticipated to have a spillway that could accommodate the 7,000 cfs emergency drawdown flow the system is capable of conveying. Although a high-capacity spillway could be added at the TRR, there is concern that excessive flow from the TRR could pose a flooding threat to residents downstream. In the event the TRR is found to not be able to accommodate the emergency drawdown flows, one option is to install additional energy-dissipation valves at Funks Reservoir and connect to the TRR pipelines, which would increase the flow into Funks where the flow could possibly be accommodated.

If the Authority adhered to Reclamation’s criteria of a maximum of 20 feet per second in the pipelines, then the maximum drawdown flow that could be sent through the pipelines would be 4,500 cfs for each system, or a total of 9,000 cfs. The additional 7,000 cfs would need to be discharged by other facilities, such as: 1) an energy-dissipation structure at the tunnel outlet that discharges to Funks Creek; or 2) the addition of more pipelines from the outlet to Funks Reservoir with additional energy dissipation to Funks Reservoir.

4.0 Recommendations

This analysis has shown that the proposed Sites Project facilities at Funks and the TRR could convey the entire emergency drawdown flow of 16,000 cfs. However, before this would be allowed, there are several recommended actions:

1. Determine what the Authority will allow for a maximum velocity in the pipes during the very rare operating condition of an emergency drawdown. A maximum velocity of 40 feet per second is allowed in similar situations, but Reclamation only allows 20 feet per second under all conditions.
2. Complete a flood analysis of this general area to determine the impacts of a 7-day discharge of 16,000 cfs in the area of the Funks Reservoir and TRR. This analysis should provide results that would indicate the maximum allowable flow to both regulating reservoirs, as well as a general summary of flooding conditions and impacts in the area.

The Jacobs design team is continuing with design of facilities to accommodate the normal operation of the Site Project and will not include additional facilities, such as additional energy-dissipating valves, which would be required for an emergency drawdown condition. However, once a flood analysis is performed as requested above in item 2, the design team can modify the facilities per direction from the Authority.

Appendix F
Funks Creek Environmental Water Source Analysis

Funks Creek Environmental Water Source Analysis Technical Memorandum (Final)

To:	Henry Luu
CC:	Michael Forest/AECOM, Jeff Herrin/AECOM
Date:	August 28, 2020
From:	Jeff Smith/Jacobs
Quality Review by:	Peter Rude/Jacobs
Authority Agent Review by:	TBD
Subject:	Site Reservoir – Funks Creek Environmental Water Source Analysis

1.0 Purpose

The Sites Reservoir Project may require providing supplemental environmental water to Funks Creek at the base of Golden Gate Dam. The reason for this possibility is that construction of this dam will isolate flow into the creek, rendering Funks Creek dry during for most of the year. To mitigate this change, a concept to introduce 10 cubic feet per second (cfs) to Funks Creek at the base of Golden Gate Dam has been suggested by the Environmental Team.

The purpose of this technical memorandum is to provide hydraulic calculations and a simple economic analysis to evaluate two different systems to deliver the 10 cfs to Funks Creek. If a change occurs in the flow rate, then this memorandum will need to be revised.

2.0 Description of Systems

Two alternative systems have been identified to deliver 10 cfs to the head of Funks Creek at the base of Golden Gate Dam. The first alternative is to provide a dedicated pumped system that includes a pump at the Funks Pumping Generating Plant (PGP), a small pipeline from Funks PGP to Funks Creek, and an outlet into Funks Creek. The second alternative system is to provide a gravity system that includes a connection at the Sites inlet/outlet (I/O) tunnels manifold (where Funks and Terminal Regulating Reservoir (TRR) 12-foot-diameter pipelines connect to the I/O tunnels), a small pipeline from this manifold to Funks Creek, and an energy-dissipation structure/outlet into Funks Creek. Figure 1 provides a basic overview of the locations of the two alternatives.

Alternative 1 will have, at Funks Reservoir, a pumping station that is dedicated to supplying water only to Funks Creek. This pump station will draw water from one of the PGP pump bays. The pipeline alignment from Funks PGP to Funks Creek will initially follow the proposed Funks and TRR 12-foot-diameter transmission pipes, but then diverge in a northwesterly direction, crossing Funks Creek, and skirting the edge of hills to keep the pipeline at a lower elevation than the Funks Creek discharge point. Keeping the pipeline lower reduces pumping head requirements. The total pipeline length is roughly 7,000 feet.

Alternative 2 begins at the I/O tunnel manifold and then travels about 2,800 feet north to the Funks Creek discharge point. This pipeline will have a higher pressure than Alternative 1. The pressure will be equal to the Sites Reservoir elevation; therefore, the pipeline will require a pressure-reducing valve to dissipate energy before the water is discharged into Funks Creek.

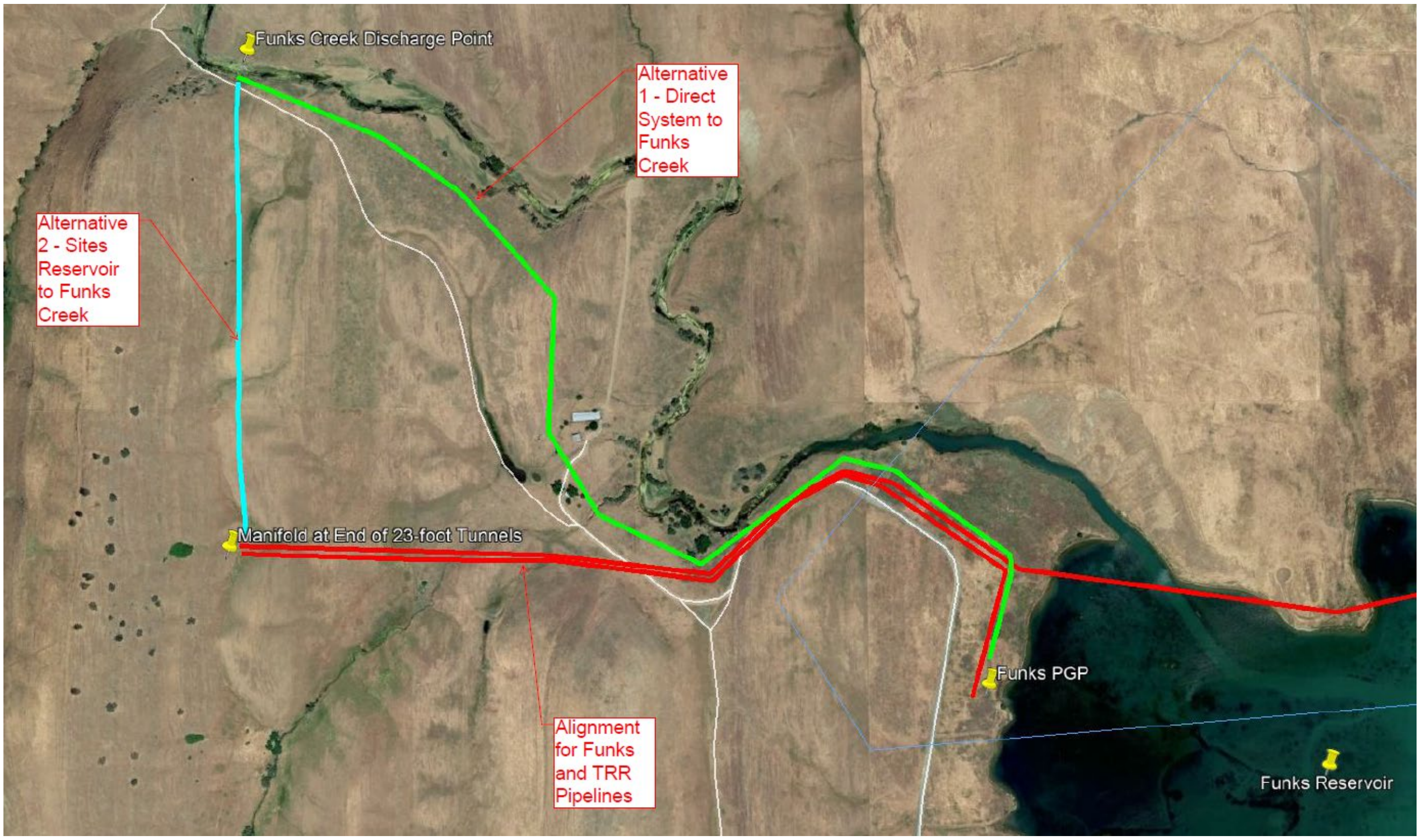


FIGURE 1: FUNKS CREEK ALTERNATIVES

3.0 System Sizing Calculations

Calculations were performed to determine pipeline and pump station (Alternative 1 only) sizes. Following are design criteria used to perform the hydraulic calculations:

- Sites Reservoir water surface elevation = 450 feet
- Funks Reservoir water surface elevation = 200 feet
- Funks Creek discharge point elevation = 260 feet
- Use of PVC pipe that can handle the pressure requirements of this system
- Hazen-Williams C-factor = 135
- Maximum velocity = 15 feet per second

Alternative 1 will require a roughly 150 horsepower pump, along with an 18-inch-diameter pipe. The pipe will be flowing at roughly 5 feet per second.

Alternative 2 will require a 12-inch-diameter pipe that will be flowing at about 13 feet per second. Because of the higher pressure in this pipeline, a pressure-reducing valve station will be required where it discharges to Funks Creek.

4.0 Economic Analysis

An economic analysis was performed, looking at a 20-year life cycle cost that included capital and operational costs for both alternatives. Following are the assumptions used for this economic analysis:

- Pumping days = 200 per year for 24 hours per day
- Pipe unit cost = $20/diameter-inch/liner foot
- Pump station = $100,000
- Pressure control valve station = $30,000
- Electricity cost = $0.12 per kilowatt-hour
- Annual electricity cost escalation = 4%
- Discount rate = 2.5%

Even though the Funks PGP will pump the 10 cfs up to Sites Reservoir, this incremental pumping above the normal 2,100 cfs for which Funks is designed to pump still has a cost associated with it. In other words, the requirement to pump an additional 10 cfs adds operational power costs that would not be included if water did not need to be discharged to Funks Creek. Therefore, this analysis includes power costs to pump to Sites Reservoir at the 10 cfs rate for 200 days each year.

A 20-year time period was selected because this is both common for this type of analysis and coincides with a general life of a pump before replacement is required. No additional cost for the Funks PGP pumping unit was assumed because adding 10 cfs of capacity to the design capacity of 2,100 cfs is very small and essentially minor.

Table 1 summarizes the economic calculations.

TABLE 1: RESULTS OF ECONOMIC ANALYSIS

Cost	Alternative 1 Pumped to Funks Creek	Alternative 2 Gravity From Sites Reservoir to Funks Creek
Capital	$2,620,000	$702,000
Operation	$1,178,000	$4,180,000
Net Present Value	$3,880,000	$5,383,000

The results of this analysis show Alternative 1 has a significantly higher capital cost; however, over a 20-year period, the total cost of Alternative 1 is much less. The higher operational cost for Alternative 2 results from the increase cost to pump to Sites Reservoir and then dissipate this extra energy as it flows back to Funks Creek.

Appendix G
Hydroelectric Energy Recovery Valuation

Hydroelectric Energy Recovery Valuation
Technical Memorandum (Final)

Sites Program Management Team

To:	Henry Luu/HDR
CC:	
Date:	August 28, 2020
From:	Wayne Dyok/H2O EcoPower
Quality Review by:	Peter Rude/Jacobs
Authority Agent Review by:	TBD
Subject:	Hydroelectric Energy Recovery Valuation

1.0 Background

The Sites Project Authority (Authority) adopted the recommended project (VP7) as provided in the *Sites Project Value Planning Alternatives Appraisal Report*," dated April 2020, to reduce the program cost from $5.2 billion to $3 billion. One of the features of this new project is to size the Sites Reservoir at 1.5 million acre-feet (MAF), as opposed to the previously analyzed 1.8 MAF reservoir. Much of the information obtained through past studies remains pertinent to the smaller Sites Reservoir. However, there are some notable differences.

In the previous studies for the 1.8 MAF Sites Reservoir, it was presumed that there would be both energy-recovery facilities at Funks and Terminal Regulating Reservoir (TRR), and additional pumped storage capability at Funks, or an alternative reservoir named Fletcher. The generation capacity was estimated to be on the order of 120 megawatts (MW). However, studies conducted for the U.S. Bureau of Reclamation (Reclamation) and the Authority indicate that the pumped storage component is marginal.[1] Because of concerns about permitting the pumped storage component and how that could affect the project schedule, and the uncertainty of future revenue streams from pumped storage, the pumped storage component is no longer part of the project.

Previous studies assumed a maximum pumping rate of 5,900 cubic feet per second (cfs) and a maximum generation rate of 5,100 cfs. For the 1.5 MAF Sites Reservoir, the maximum pumping rate is set at 3,900 cfs (2,100 cfs from Funks Reservoir and 1,800 cfs from the TRR). The maximum reservoir elevation is 497.6 feet (mean sea level) and corresponds to 1.5 MAF of total storage. The minimum reservoir level is at elevation 340 feet, corresponding to a capacity of about 120,000 acre-feet (ac-ft).

[1] The results of the pumped storage study were based on the current capacity valuation requirements adopted by California Independent System Operator (CAISO). However, there is considerable literature suggesting the CAISO will modify their capacity valuation requirements and capacity values will increase substantially in the future as discussed further in this report.

Maximum release flows are established at 2,000 cfs to Funks Reservoir and 1,000 to the TRR. Two 12-foot-diameter pipes connect Sites Reservoir with Funks, and two additional 12-foot-diameter pipes connect Sites Reservoir with the TRR.

Funks Reservoir has a usable capacity of 1,170 ac-ft between elevations 199.5 and 205.2 feet, and a dead storage of 1,080 ac-ft below elevation 199.5 feet.[2] The TRR has a maximum water level of 124 feet. Typically, it is operated between elevation 123.0 and 123.2 feet in the summer and 121.8 feet in the winter. It is assumed to have a storage capacity of 446 ac-ft.

Each of the two 12-foot-diameter pipes connecting Sites to Funks will have a maximum release flow of 1,000 cfs. Jacobs has calculated the frictional head loss at 17.7 feet during periods of maximum head and maximum release flow. During maximum release flow and minimum head, the head loss is expected to be 18.7 feet. Similarly, the head loss between Sites Reservoir and the TRR is estimated to be 18.1 feet at maximum head and maximum release flow and 19.1 feet at minimum head and maximum release flow.

Release flows from Funks and the TRR will be discharged to the Tehama-Colusa Canal and the Glenn-Colusa Irrigation District (GCID) Main Canal.

2.0 Purpose

The Sites project is a water supply project using the 1.5 MAF Sites Reservoir for off-stream storage. The project requires water to be pumped from the Sacramento River during periods of high flow to two smaller reservoirs (Funks and the TRR) via two canals. From Funks and the TRR, water will be pumped into the Sites Reservoir. The Authority desires to recapture the pumping energy during periods of water supply release. The objectives of the hydropower task are

- Size and cost hydroelectric turbines at Funks and the TRR.
- Identify permitting approaches to meet the project schedule.
- Determine the value of recovered energy consistent with the operating objectives of providing release flows.
- Provide recommendations for moving forward.

3.0 Turbine Sizing and Cost

Based on the maximum head differential at Funks and TRR, maximum pipeline flows and associated head loss, two 21.4-MW turbines (total 42.8 MW) were preliminarily sized for Funks energy recovery based on a 90 percent turbine efficiency.[3] At the TRR, two 13.8 MW turbines (total 27.6 MW) could be installed for an approximate total capacity of about 70 MW.[4] This information was provided to the project team for the electrical connection assessment.

Three turbine suppliers (Mavel, General Electric, and Voith) provided technical assistance on turbine design details and cost information. All three were cooperative and willing to supply information at their cost to assist in the turbine sizing and selection. A fourth supplier, Andritz, will be asked to provide technical assistance during the next phase of the project. Each company was provided the same basic information, as illustrated in Table 1.

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[2] There is uncertainty regarding the actual storage at Funks and it is anticipated that a bathymetric study will be undertaken in later phases of design.

[3] Modern turbines have efficiencies greater than 90 percent, but this sizing was to preliminarily identify the approximate turbine sizes. Turbine sizes may be slightly adjusted as the design proceeds.

[4] The turbine design head is normally set at the head at which the project most frequently operates and provides the best operational efficiency. However, the turbine design also includes the maximum and minimum operating parameters.

Table 1: Project Operation Data

	Funks Reservoir	Terminal Regulating Reservoir
Maximum. Sites Water Elev. (feet [ft])	497.6	497.6
Minimum. Sites Water Elev. (ft)	340	340
Maximum Res. Elev. (ft)	205	124
Minimum Res. Elev. (ft)	199.5	119
Maximum Head (ft)	298.1	378.6
Minimum Head (ft)	135	216
No. Inflow Pipelines (12 ft diameter)	2	2
Total Generation Flow (cfs)	2000	1000
Flow per Turbine (cfs)	1000	500
Pipe and Minor Head Loss (ft)	18.5	18
Turbine efficiency (percent)	90	90
Maximum Generation (MW)	42.8	27.6
Maximum Generation at Min Head (MW)	17.7	15.0
Design Head (ft)	210	290

At Funks, it was determined early in the analysis that one turbine cannot operate over the full range of Sites Reservoir water surface elevation fluctuations. Therefore, either two different turbines would be needed at Funks, with some expected overlap in operation or two identical turbines could be selected and energy recovery unavailable at Sites Reservoir levels below the turbine operating range. For the former case, if releases are at the maximum release level and above the maximum hydraulic capacity of the unit (i.e., 1,000 cfs) and outside the overlap band, some energy may not be recaptured. A final decision on the second Funks turbine selection cannot be made until project operations modeling is completed in Fall 2020. For this analysis, it was assumed that all release energy could be recaptured to provide an upper level estimate of energy recovery potential. (At the TRR, the head is large enough that one turbine can operate throughout the entire Sites Reservoir operating range.)

The suppliers provided turbine design data for both Funks and TRR, based on operation at the maximum head and maximum generation flow (i.e., identical units at Funks and identical units at TRR). Two additional cases were also examined: (1) a turbine that would operate at a lower head over which the units would operate most often[5], and (2) a turbine that would operate down to the lowest Sites Reservoir level of 340 feet. The latter design indicated how much overlap there would be between units. These later options were identified to recover as much of the pumping energy as possible without adding a third turbine. Suppliers also provided information on generators, controls, electrical interconnections, turbine submergence for civil design, and cost.

Once the 1.5 MAF Sites Reservoir modeling is completed, it will be possible to optimize the second turbine design in Funks Reservoir based on an updated water level duration curve. Similarly, once the storage and operational characteristics of Funks and TRR are firmed, energy recovery calculations can be refined. For a given head, the turbine suppliers also will need to specify the minimum flows at which each turbine can

[5] Based on review of the modeling results for the 1.8 MAF Sites Reservoir alternative, the water level in Sites Reservoir appeared to be well above the minimum water level for most years, most of the time. Hence the period when the energy would not be recoverable may be small.

operate. This will be done in a subsequent task. For the present task, it was assumed that all potential energy from project releases could be recovered.

As the Sites Project design progresses, the Jacobs team will continue to work with suppliers to refine turbine designs and undertake value engineering to reduce costs (for example, using pump/turbines versus independent pumps and turbines).

For TRR operations, two 12 MW units are recommended. At Funks, assuming only two turbines are installed, the turbines would have nominal capacities of 20 MW based on design head, or one could be 20 MW and the second about 14.5 MW, if the facility was designed to operate down to the minimum reservoir level of 340 feet. (Note that these units can produce more than the design capacity at maximum head and maximum flow.) Hence, the total capacity for the 4 turbines could vary between 64 and 58.5 MW. For the 58.5 MW capacity option, energy could be recaptured over the entire head range; however, when release flows exceed 1,000 cfs at Funks the energy associated with the excess release might not be recoverable.

The design drawings provided by the suppliers are not in this technical memorandum. Generic turbine and generator data and supplier-provided design drawings are included in the design drawing package. Dimensions for these units should be reasonably close to the final design dimensions for the turbines and generators.

Available literature for turbines, generators, and controls suggests that costs for the electromechanical components can vary significantly. Price data provided by the suppliers included only costs for specific components. At this stage of design, Jacobs selected a wide price band, with an upper limit cost of $2000 per kilowatt for the turbines, generators, controls, civil works costs for the turbine/generator installation, and other related costs. The resulting upper-level capital cost would be on the order of $128 million. Based on our experience, the costs could be as low as $60 million (i.e., $1,000 per kilowatt). For the economic analysis, the higher cost was used to be conservative.

Suppliers generally agreed that a 2-year schedule for manufacture and delivery of the turbines and generators was reasonable. The procurement schedule will be refined as the design proceeds.

4.0 Permitting

Because of permitting and construction schedule concerns, the Authority has stated that the Federal Energy Regulatory Commission (FERC) licensing process poses unacceptable risks and the Authority does not intend to pursue a FERC project license at this time. Two other alternatives to FERC licensing are available: (1) obtain a FERC conduit exemption; or (2) seek legislative approval for the project to be exempt from FERC. (During informal discussions with FERC staff, FERC staff stated that they believe the Sites Project would be jurisdictional, most likely because of interstate commerce and possibly because of the project's dependence on Sacramento River flow for its existence [i.e., navigable water way].)

Because the project is an offstream water supply project, it should qualify for a 40 MW conduit exemption. The basic question is whether the project can be considered as one or two projects for purposes of the conduit exemption. Project information has been provided to FERC and an informal response on whether the project can be considered as two conduit exemptions is expected in late August. Should the Authority select the conduit exemption route, the Authority would need to consult with agencies, Native American tribes, and the public; however, because of the ongoing National Environmental Policy Act process, much of the consultation could be waived by FERC upon request. A simple exemption application would be required that relates only to the turbines, generators, and associated controls as that would be the limit of FERC jurisdiction. FERC approval for conduit exemptions is typically less than a year (note that permitting of San Diego County Water Authority's 20 MW Lake Hodges conduit exemption took less than 1 year from the time the application was filed).

In some respects, obtaining congressional approval to exempt the project from FERC entirely and be regulated by the State of California might be the simplest and most expeditious approach. Experience with obtaining such approvals generally requires that the project not be controversial and has support for the project from at least one U.S. Senator representing California. The Sites Project is going through both federal and state permitting. Although there may be some limited opposition to the project, at the federal, state, and local levels, general agreement on the need for the project appears to exist. Should the Authority elect to move forward with the legislative option, it would likely take between 6 months and 1 year for House, Senate, and Presidential approval.

5.0 Energy Recovery Valuation

5.1 Approach

Because operations modeling information will not be available until October, previous modeling results for the 1.8 MAF Sites Reservoir alternative were used and adjusted to meet the operational constraints of the 1.5 MAF Reservoir[6]. Rather than model the entire 82 years of record, only 3 years were selected for analysis, as follows: (1) a dry year represented by water year 1930; (2) average year represented by water year 1993; and (3) wet year represented by water year 1971. Each year modeled is characteristic of 1/3 of dry, average, and wet years, but are not the lowest or the highest flow years. Each of these three years had previously been modeled for the pumped storage valuation conducted by ZGlobal in 2019 for the Authority and Reclamation.

The daily pumping and release flows to and from Sites Reservoir were capped at the maximum pumping rate of 3,900 cfs and maximum release rate of 3,000 cfs. It was assumed that pumping would be available at the rate used for the 1.8 MAF reservoir assuming the pumping was at or less than the maximum pumping rate and the maximum reservoir level was not exceeded. Similarly, the release rates were assumed to be the same as in the 1.8 MAF case subject to the maximum release cap of 3,000 cfs. The initial reservoir level for each modeled year was assumed to be the same as for the 1.8 MAF case. Because this might have provided higher reservoir levels than would otherwise occur under a 1.5 MAF reservoir, a sensitivity analysis was undertaken with a lower starting reservoir level for the dry year.

Sites Reservoir levels were not allowed to exceed the maximum level of 497.6 feet or go below the minimum level of 340 feet. Pumping flows were curtailed if the reservoir level reached 497.6 feet. Similarly, the reservoir levels were checked to ensure that the Sites reservoir did not fall below elevation 340 feet.

Release flows were prorated between Funks and TRR with two-thirds of the release flow going to Funks and one third going to TRR. This was done based on the 2,000 cfs capacity into Funks and the 1,000 cfs capacity into TRR.

Daily energy generation was based on the daily flows to Funks and TRR, and the associated reservoir level, and an assumed plant efficiency of 90 percent.[7] A daily release flow volume was calculated, with the reservoir volumes adjusted daily. Using the Sites Reservoir elevation-storage capacity curve, the reservoir level was recalculated for each day and used in the generation calculation for that day. Daily adjustment for evaporation and leakage were made consistent with the 1.8 MAF modeling.

For the power and daily energy calculations, the gross head was based on the daily Sites Reservoir level and the average reservoir level for both Funks and TRR. These were established at 202 feet for Funks and 123 feet for TRR. (A level of 123 feet corresponds to the average summer water level for TRR.) Daily reservoir fluctuations in both Funks and TRR were ignored because the variations are only a few feet, compared to the total head, and the daily water level is expected to average or be slightly below the levels of 202 and 123 feet, respectively. Head loss for Funks was assumed at 18.5 feet at Funks and 18 feet at TRR based on operations at full capacity during peak demand hours.

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[6] The operating criteria for the 1.5 MAF reservoir will be different than for the 1.8 MAF reservoir alternative, but using the maximum and minimum constraints for reservoir levels and pumping and generation should yield reasonable results for valuing the recovery energy.

[7] Supplier-provided information indicated that the turbine efficiencies can be on the order of 94 percent at design conditions. However, turbine efficiency decreases during other operating conditions. Hence, a conservative efficiency of 90 percent was used to estimate recovered energy.

It was assumed that daily storage up to the maximum Funks and TRR storages of 1,170 ac-ft and 446 ac-ft, respectively, would be available for project operations. To take advantage of higher-value energy during the peak demand periods for each day, the turbines were assumed to operate at full capacity (2,000 and 1,000 cfs) for the number of hours of available flow based on the daily release volume. Both Funks and TRR were assumed to be at minimum reservoir levels at the start of the daily generation cycle. Once the daily flow balance was achieved, the turbines would be shut down and generation would cease. However, releases at the desired release rate would continue at both Funks and the TRR[8]. The cycle would repeat itself each day. Because peak demand hours have a significantly greater value that off-peak hours (for example, peak energy values can exceed $100 per megawatt-hours [MWh] in August and off-peak energy prices may be as low as $30 per MWh the same day), it is far more cost effective to operate during the peak hours, even though energy losses at maximum flow capacities are higher.

In release mode, the 1,170 ac-ft of storage in Funks should be enough to store any release flow (such as, at a release of 1,000 cfs and 2,000 cfs generation, storage requirement would be 991 ac-ft, corresponding to the maximum storage requirement). Therefore, at Funks, the units were assumed to operate at full capacity until the daily release volume was met. Of course, to conserve storage volume, if needed, one unit could also be shut off earlier and the second unit operate at full capacity for a longer period.

At the TRR, the maximum storage volume available is 600 ac-ft, which is above the maximum storage requirement of 496 ac-ft at a 500 cfs release and operation at 1,000 cfs. To facilitate the energy-value calculation, the spreadsheet was set up to take advantage of the maximum daily energy values during the peak demand period. This would slightly overestimate the value of the energy, during release flows that could not be fully stored. Because the value of the off-peak hours does not differ much, this approach would only be a minor underestimation of the value of the energy.

The daily energies were summed to provide an annual total of MWhs of generation. Because Sites Reservoir levels did not start and end at the same elevation for each of the 3 years analyzed, the annual energy values are presented for energy taken out of storage and for energy recovery of that year's pumping energy.

Forecast peak and off-peak energy prices for the year 2030 for each hour of the day, for each month, were obtained from ZGlobal. Two sets of prices were provided for the peak hours; Monday through Friday and weekends/holidays. Since it is uncertain if a peak hour will be a work day or weekend, the peak hourly rates were assumed to occur 5/7 of the time and the peak hour weekend rates were assumed to occur 2/7 of the time. This resulted in peak hour rates that are a combination of the two data sets provided by ZGlobal.

The number of hours of generation were determined by dividing the MWh of generation by the generation rate for that day assuming maximum generation. The average energy price during the hours of generation was then determined and multiplied by the MWh of generation [9]. Lastly, the daily energy values were summed for the year to derive the annual revenue. For comparison, the average daily energy values for each month were also used to determine an approximate value for the energy if flow was released at the specified release rate over the 24-hour period. The difference between the two annual energy values is illustrative of the increase in revenue from operating during peak demand periods to the extent possible.

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[8] For example, if the release for a given day was 500 cfs at Funks, then the project could either be operated at full capacity (2,000 cfs) for 6 hours or full capacity of one unit (1,000 cfs) for 12 hours. The units would then be shut off once the daily flow requirement from Sites was met. In the first case, assuming Funks was at a lower level at the start of generation, the project would need to store 1500 cfs for 6 hours, or a total of 744 ac-ft, which would be released during the remainder of the day at a constant rate of 500 cfs. In the latter case, the project would need to store 500 cfs for 12 hours, or a total of 496 ac-ft. Since both these storage requirements are less than the usable storage in Funks, usable storage would not be a controlling factor.

[9] For example, if the daily release was 450 cfs, that would equate to 5.4 hours of operation at 2,000 cfs. If the energy rates during those hours were $90, 100, 110, 100, 90, and 85, the average value during that 5.4-hour period would be $97.03. The MWh generated during that 5.4 hour-period would be multiplied by $97.03 to obtain the daily energy value.

Ancillary benefits were not accounted for in this analysis. This could be an additional source of revenue to the project. However, relative to capacity and energy benefits, based on previous modeling by ZGlobal these benefits would significantly less.

5.2 Results

Modeling results for the three years are summarized in Table 2. The table illustrates that releases in the wet and dry years were 701,000 ac-ft and 823,000 ac-ft respectively, but only 134,000 ac-ft in the average water year (1993). This is primarily because the Sites Reservoir elevation at the beginning of the year was low, with the operating priority to refill the reservoir. Far more water was diverted in the average-flow year compared to the low-flow year. However, in the wet year, the reservoir began at a high level. Flow was diverted to the Sites Reservoir until the reservoir was full, at which time, diversions were curtailed.

In both the wet and dry years, more water was taken out of storage than was diverted to Sites Reservoir. In the average year, 863,000 ac-ft of storage was added to Sites Reservoir. Net evaporation in each of the three years was assumed to be the same at about 25,000 ac-ft. Adding the diversion volume to the volume taken out of storage, and subtracting the net evaporation yields the same volume as the release volume in all three years (i.e., flows balance as shown in Table 2).

Table 2: Modeling Results

Water Year	1930 (dry)	1993 (average)	1971 (wet)
Sites Vol. Start Yr. (ac-ft)	847,000	197,000	1,138,000
Sites Vol. End Yr. (ac-ft)	580,000	1,060,000	953,000
Storage Released (ac-ft)	267,000	-863,000	185,000
Volume Diverted (ac-ft)	581,000	1,021,000	540,000
Evaporation (ac-ft)	25,000	25,000	25,000
Div. – Evap. + Rel (ac-ft)	823,000	134,000	701,000
Released Flow (ac-ft)	823,000	134,000	701,000
Average Sites Reservoir Elev. (ft)	447	421	479
Generation (MWh)	191,403	33,723	180.748
Generation from Reservoir Storage (MWh)	60,343	-174406	47,322
Annual Generation Value (2018$ @ $50/MWh)	$8.4 million	$2.2 million	$8.0 million

The water level in Sites varied from 414 feet to 480 feet (i.e., net head of about 260 feet to 194 feet) in the dry year; 359 to 474 feet in the average year, and 451 feet to 497.6 feet in the wet year. Two identical turbines associated with Funks and operating at the highest head would be able to operate over the entire head range in both the wet and dry years, and no energy would need to be wasted in these water years. However, in the average year (1993), when water levels are below about elevation 400 feet, the turbines may not be able to operate, unless one is set for a lower head. This needs to be investigated further during the next phase of work, once the operations modeling is completed.

Total generation was highest in the dry year at 191.403 MWh, whereas generation was 180,784 MWh in the wet year, but only 33,723 MWh in the average year. The average annual energy revenue for the three years is about $6.8 million.

For all years, there was little to no generation during the months of December, January, February, and March. During the dry year, there was generation for about 6 to hours per day in April and May, increasing to 24 hours per day in June and July, and then decreased generation to 17 and 10 hours in August and September. October and November averaged 12 and 7 hours respectively. During the average water year, there was almost no generation in April, May or June. During these months there was some filling. In July August and September, generation averaged about 6 hours per day. In October and November there was almost no generation. For the wet water year, generation averaged about 5 hours per day in April, but there was no generation in May. However, in June and July, generation was continuous at 24 hours per day. In August and September generation decreased to about 11 and 15 hours per day, respectively. In October and November, generation occurred an average of 12 and 7 hours respectively. The generation patterns suggest that the project might qualify for capacity credit and during part of the year, could be used for pump storage to increase revenue.

The sensitivity analysis indicates that if the starting reservoir level was 416 feet (i.e., 600,000 ac-ft) rather than elevation 442 feet (847,000 ac-ft), the energy generation would decrease to 171,952 MWh. This corresponds to a value of $7.5 million. Similarly, if the Sites Reservoir starting level was 404 feet (500,000 ac-ft of storage), energy would drop further to 162,967 MWh, or a value of $7.2 million. With the lower starting level, the Sites Reservoir would drop to a minimum level of 366 feet (storage of 233,000 ac-ft). The decreases are primarily because of the lower head levels in Sites Reservoir as the flows were not changed.

During the critical summer season, Funks and TRR could both provide capacity to the CAISO grid during dry, average or wet years. Approximately 50 MW of capacity might qualify based on the average available capacity during the critical summer period. At $200 per MW-day, that capacity could have a value of an additional $3.6 million[10]. Hence, in both a dry year, like 1930, and a wet year, like 1971, the project could have an annual revenue on the order of $12 million; but, in an average year, like 1993, the revenues would drop to $5.8 million. This is due to 1993 being used primarily to refill the Sites Reservoir. Assuming an annual operations and maintenance cost of about $500,000, net annual revenue would average about $9.8 million for the three years with 2030 energy and capacity prices. For the economic analysis, using a capital cost of $128 million, the equivalent annual cost over a 50-year life at 3 percent would be about $5 million resulting in a benefit-cost ratio of about 1.82. If the capital costs are lower, the benefit-cost ration would be higher and conversely if the energy cannot be fully recovered, the benefit-cost ration would decrease.

It should be kept in mind that the energy recovery will only be a percentage of the pumping energy used to fill Sites Reservoir. However, pumping will be undertaken during periods when power rates are lower and much of the energy generation will be accomplished during the peak energy price periods. Further, the pumping costs can be structured to avoid or minimize capacity costs. If the project is operated to provide capacity payments, then the value of the recovered energy could exceed pumping costs.

6.0 Recommendations

Once reservoir modeling is completed in October 2020, the energy recovery modeling analysis should be done using the entire 82-year record based on updated operating rules. This will provide a revenue stream for the 82 years simulated, which can be factored into a present-worth analysis. This will also negate the need to consider annual carry-over storage, since the carry-over storage will become insignificant over the 82-year period.

[10] The recent rolling outages caused by the heat wave suggests that CA ISO may rethink how capacity value is determined. The Sites Project presents a unique opportunity for obtaining capacity credits because generation coincides with the high electrical demand period.

The energy-recovery modeling analysis will provide critical water level and generation release duration curves that can be used to specify the Funks turbine designs. Without that data, the design of the second turbine at Funks cannot be optimized. The TRR design can be further optimized, but is better understood at this time as compared to Funks. Jacobs should continue to work with turbine suppliers to firm the Funks designs and improve the capital cost estimate.

Because the capacity value may be a significant component of the annual revenue stream, the Authority should monitor developments at CAISO as CAISO revises its capacity requirements over the next couple of years. It might also be beneficial for the Authority to participate in any capacity rule-makings. Perhaps more importantly, as the Authority enters discussions with entities (like Pacific Gas and Electric [PG&E] and Western Area Power Administration (WAPA) to purchase pumping power and design the electrical interconnections, the value of the project's energy generation and capacity to PG&E or WAPA should be a key component of the discussions, particularly because the capacity and energy values would be available to that entity at a time when they are critically needed. In that context, the Authority should consider how project operations can be adjusted to accommodate reservoir release requirements and maximize the value of recovery energy and project capacity.

In the longer term, the Authority should consider modeling future electricity prices for 2040 and beyond, because electricity and capacity prices may change as renewables become a larger percentage of the generation mix in California.

Depending upon FERC's guidance for a conduit exemption, the Authority should consider moving forward with the conduit exemption process or federal legislation to exempt the project from FERC's oversight.