Diversion during Construction Technical Memorandum

Sites Program Management Team

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Date:	August 20, 2020
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Acronyms and Abbreviations

ac-ft	acre feet
ATS	automatic transfer switch
cfs	cubic feet per second
DWR	California Department of Water Resources, Division of Engineering
EIR/EIS	Environmental Impact Report/Environmental Impact Statement
ft/s	feet per second
HMI	Human/Machine Interface
HPU	hydraulic power unit
I/O	Inlet/Outlet
MAF	million acre-feet
PLC	Programmable Logic Controller
SCADA	Supervisory Control and Data Acquisition
TM	technical memorandum
USBR	U.S. Bureau of Reclamation
WSE	water surface elevation

1.0 Introduction

1.1 Project Overview

The Sites Project Authority (Authority) is preparing a feasibility-level evaluation for a 1.5-million-acre-foot (MAF) reservoir as a preferred option for the Sites Reservoir Project. This reservoir would be in the same location as the reservoir studied previously by the California Department of Water Resources, Division of Engineering (DWR), and the U. S. Bureau of Reclamation (USBR).

Golden Gate Dam and Sites Dam would be constructed on Funks Creek and Stone Corral Creek, respectively. During construction, natural creek flows would need to be diverted downstream. The diversion at Sites Dam would ultimately be used as a permanent outlet for stream maintenance and emergency reservoir drawdown releases.

1.2 Purpose and Scope

This memorandum presents information related to the feasibility design of creek diversions during construction, and the permanent outlet at Sites Dam. It has been prepared to support the project description for Alternatives 1 and 2 for consideration in the Environmental Impact Report/Environmental Impact Statement (EIR/EIS).

1.3 Limitations

The scope of work for this technical memorandum (TM) was restricted to the development of feasibility designs for the Funks Creek and Stone Corral Creek diversions during construction, and the permanent outlet at Sites Dam diversion features. It did not include consideration of other Sites facilities beyond those specifically listed.

The feasibility designs presented in this TM were based on topographic contours that originated from DWR for their 2003 studies (DWR, 2003a; DWR, 2003b). Updated site-specific topographic maps would be prepared for use in preliminary and final phases of design.

AECOM represents that our services were conducted in a manner consistent with the standard of care ordinarily applied as the state of practice in the profession, within the limits prescribed by our client.

This TM is intended for the sole use of the Sites Project Authority. The scope of services performed may not be appropriate to satisfy the needs of other users; and any use or re-use of this document, or of the findings, conclusions, or recommendations presented herein is at the sole risk of said user.

2.0 Hydrologic and Hydraulic Assumptions

2.1 Hydrologic Assumptions

Hydrographs were prepared for Funks Creek and Stone Corral Creek using the USBR Hydrology Manual (see Figure 2-1). The peak flows for a 100-year return period for Funks Creek and Stone Corral Creek were determined to be 8,500 cubic feet per second (cfs) and 11,800 cfs, respectively. This analysis would need to be refined at later stages of design with a Hydrologic Modeling System Hydrology Study.

2.2 Hydraulic Assumptions

The assumed distributions of flows to permanent outlets in the reservoir are described in the Funks and Stone Corral Creeks Reservoir Operating Elevations and Emergency Release Management TM (AECOM, 2020a). The assumed ultimate maximum release through the Sites outlet is 2,500 cfs for a 1.5-MAF reservoir.

Emergency drawdown flows would be decreased for a 1.3-MAF reservoir. However, these flows have not been determined at this stage. For the purposes of providing information for the EIR/EIS, this TM focuses on a 1.5-MAF reservoir.

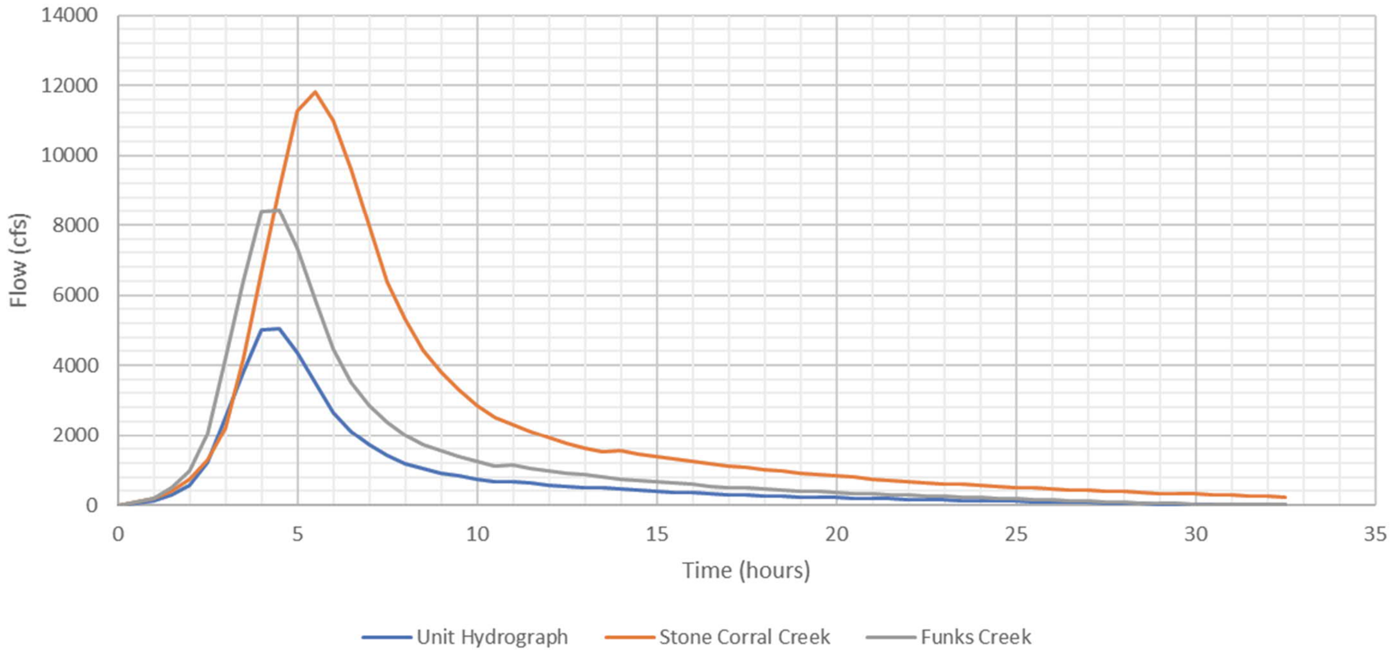

Figure 2-1. 100-Year Flood Event on Funks and Stone Corral Creeks at Sites Reservoir

Additional outlets for emergency drawdown releases would be situated around the reservoir. The main inlet/outlet (I/O) works provides the most hydraulic capacity, as detailed in the I/O Tower & Tunnels TM (AECOM, 2020b). Two additional potential high-level outlets are described in the Funks and Stone Corral Creeks Reservoir Operating Elevations and Emergency Release Management TM (AECOM, 2020a).

It is assumed that 10 to 50 cfs would be required for normal stream maintenance releases, with occasional requirements for releases of up to 200 cfs during storm events.

3.0 Golden Gate Dam Bypass

3.1 Construction Flows to be Diverted

Storage capacity at the Golden Gate Dam site was analyzed based on the preliminary design for the Golden Gate Dam (AECOM 2020c). It was determined that a cofferdam sited upstream of the dam could provide sufficient storage to contain the 100-year storm flood flows, which could then be released downstream of the dam via a diversion in a controlled fashion.

Typically, county flood control detention storage guidelines require designs to consider 24-hour storms. Glen and Colusa Counties do not have such guidelines; however, nearby Yolo County guidelines require consideration of 24-hour storms.

A hydrologic analysis was performed using the unit hydrograph from Table 4-15 of the Flood Hydrology Manual (USBR, 1989). Flood flows were considered for 33 hours after the start of the storm to account for lag time. With no outflow, the anticipated volume that would need to be stored over 33 hours for a 100-year storm event on Funks Creek is approximately 5,900 acre-feet (ac-ft) (see Appendix A).

If water could be stored to water surface elevation (WSE) 305 feet, there would be about 6,700 ac-ft of storage capacity upstream of the dam. A cofferdam can be constructed to elevation 310 feet (about 60 feet high) in one season to protect against the 100-year storm event. This would allow water to be stored to elevation 305 feet while retaining 5 feet of freeboard.

3.2 General Arrangement

The Basis of Design Main Dams and Saddle Dams TM (AECOM, 2020c) considers different options for Golden Gate Dam. Option 2 is a hardfill dam that would include the I/O works in the dam itself. If this option is advanced, the I/O works themselves could be used for diversion during construction. This memorandum describes the Golden Gate Dam diversion that would be constructed if an earthfill dam option is advanced.

A diversion pipe on the order of 2,000 feet long would be trenched in the bedrock under the foundation of the dam using an alignment that minimizes excavation and interference with the dam foundation. The steel pipe would be encased in reinforced concrete. The pipe would be backfilled with concrete grout when decommissioned at the end of construction. It is assumed that a 48-inch-inside diameter diversion pipe would be reasonable to allow for access and inspection, while minimizing decommissioning requirements. A 48-inch pipe could release flows of up to 200 cfs with a velocity of about 16 feet per second (ft/s).

There would be a short riser (5 to 10 feet tall) with a trashrack on the upstream end of the pipe. On the downstream end, a flow control valve would be installed along with riprap to provide energy dissipation to diverted flows.

3.3 Mechanical Components

The Golden Gate Dam bypass piping system would be composed of a trashrack at the inlet, an outlet guard valve, an access port, and a flow control valve. No upstream control of the 48-inch pipe has been assumed at this time.

The inlet bar trashrack would be installed to protect the pipe and appurtenances from damage and clogging due to debris. The sizing and the spacing of the bar rack and its slots would be developed as design progresses. It is expected that aquatic life impacts would not be a factor when assessing flow conditions through the inlet bar rack during construction. This would require consultation and verification with the project environmental team.

At the bypass piping outlet, the system assembly would include a guard valve, a reducer fitting, and a flow control valve. The guard valve would be used to isolate the upstream pipe from the flow control valve to facilitate any maintenance work. An American Water Works Association Standards C-504 butterfly valve would be a sufficient guard valve. It is expected that both valves would be operated solely via manual actuation, requiring operators to access the outlet site for operation. A temporary flow meter would be installed to measure flow rates for a range of flow control valve settings.

The flow control valve would be the final component in the bypass piping assembly, allowing for controlled flow releases to Funks Creek. It is expected that this valve would be a smaller diameter than that of the bypass pipe, and would therefore require an upstream reducer fitting. The flow control valve could be a throttling knife gate valve, such as that manufactured by Hilton, or a multiple-orifice valve, such as that manufactured by Ross. The valve type and size would be determined as design progresses.

4.0 Sites Dam Diversion and Permanent Outlet

4.1 Construction and Permanent Flows

Storage capacity at the Sites Dam site was analyzed based on the preliminary design for Sites Dam (AECOM, 2020c). It was determined that although a cofferdam upstream of the dam could contain a portion of the 100-year storm flood flows, the diversion would need to be opened to release a significant portion of storm inflow downstream of the dam.

A hydrologic analysis was performed using the unit hydrograph from Table 4-15 of the Flood Hydrology Manual (USBR, 1989). Flood flows were considered for 33 hours after the start of the storm to account for lag time.

With no outflow, the anticipated volume that would need to be stored for a 33-hour 100 year storm event on Stone Corral Creek is 3,500 ac-ft.

Assuming a cofferdam up to elevation 310 feet with 5 feet of freeboard (see Section 3.1 above), water could be stored up to a WSE of elevation 305 feet, providing storage capacity of about 1,940 ac-ft. If a larger cofferdam can be constructed in the first dry season during construction, there would be greater storage capacity. Should construction of a cofferdam that can store the 100-year storm be infeasible in one season, the diversion would need to operate during the storm to keep water levels from overtopping the cofferdam.

Maximum flows for the permanent outlet at this location are 2,500 cfs (see Section 2.2). Preliminary analysis shows that release capabilities of up to 2,500 cfs, in combination with 1,940 ac-ft of storage, should be sufficient to prevent the cofferdam from overtopping (see Appendix B). A 12-foot-diameter tunnel with two 84-inch-diameter fixed-cone valves, as described in the sections below, would accommodate these releases.

4.2 General Arrangement

The diversion at Sites would ultimately function as a permanent outlet; unlike the Golden Gate Dam bypass, it would not be decommissioned after construction.

Sites Dam would either be an embankment dam (Options 1A and 1B) or a hardfill dam (Option 2). If a hardfill dam is constructed, then the diversion/outlet can be placed through the dam. If an embankment dam is constructed, then an outlet tunnel would be constructed through the dam abutment to avoid creating a potential seepage path through the embankment. This TM and its associated drawings focus on the embankment dam options where the diversion/outlet would go through the abutment.

On the upstream end of the diversion/outlet pipe, there would be an intake with a trashrack similar to the low-level intake for the I/O works (AECOM, 2020b). On the downstream end, a flow control valve (such as a fixed-cone valve) would be installed, along with an energy dissipation chamber and riprap to provide energy dissipation to diverted flows.

The energy dissipation chamber would be designed to accommodate emergency drawdown flows, as well as stream maintenance releases dictated by the permitting agencies.

4.3 Mechanical Components

4.3.1 Intake Appurtenances

The intake system for the Sites Dam piping system is expected to include a bar trashrack, slide gate, a separate fish screen and inlet valve to support Stone Corral creek release flows, a stoplog bulkhead, and permanent air vent assembly.

The inlet bar rack would be installed to protect the tunnel from damage and to prevent clogging due to debris. The sizing and the spacing of the bar rack and its slots would be developed as design progresses. The use of an independent inlet valve for stream maintenance flows would limit the inlet bar rack's use to supporting drawdown scenarios, and thereby minimize requirements for aquatic life protection at the bar rack.

A slide gate, 14 feet by 14 feet, would be installed downstream of the inlet bar rack. This gate would normally be closed; it would operate to support emergency drawdown events. The gate would be hydraulically actuated using stainless-steel actuators and hydraulic oil supply tubing to resist corrosion, induced by continuously submerged conditions. The hydraulic actuation system would be on-off, and would include limit switches for open and close relays.

Teeing off the tunnel piping downstream of the slide gate, a fish screen and valve assembly would be installed and operated to support stream maintenance flows. The fish screen would be designed and sized to meet the requirements for aquatic life protection. The fish screen would be constructed of copper-nickel for antibiofouling considerations; an air burst system is not currently included. Depending on the structural requirements and design for supporting this assembly, the valve would be either a slide gate or metal-seated butterfly valve. The current approximation for maximum stream maintenance flow is 200 cfs. A 42-inch valve would be sufficient in size to support this flow. An actuation system similar to the 14-foot by 14-foot slide gate would be used.

The hydraulic actuation system would be powered by a hydraulic power unit (HPU) to be installed in a control building on the Sites Dam crest. Food-grade hydraulic oil would be routed from the HPU via hydraulic-grade stainless-steel tubing to the valve actuators submerged in the reservoir. Backup power to pressurize the HPU would be provided to operate the inlet valves for two open-close cycles each.

Downstream of both valves, branch piping would manifold to a common air vent pipe. This vent pipe would be required to vent the tunnel during dewatering and re-filling. The required vent would be routed along the heel of the dam and up to the dam crest to discharge above the reservoir's maximum operating water level.

A stoplog bulkhead could be installed upstream of both inlet valves to allow for an additional water-tight barrier when dewatering the tunnel for inspection. The bulkhead, composed of multiple pieces, would be fitted into a permanent slot. The bulkhead must withstand the maximum reservoir hydrostatic pressures applied on the upstream side. Installation and removal would be accomplished by boat and barge-mounted handling equipment from the surface, with loading on the pieces equalized upstream and downstream.

4.3.2 Outlet Works

The outlet works system at the tunnel outlet would include guard valves, combination air release and vacuum valves, flow control valves for drawdown, and one flow control valve for creek release. Various fittings (e.g., tees, wye-branches, and reducers) would be required.

An outlet structure would be constructed to house the appurtenances and their control systems. It is expected that all valves would be operated using electric actuators with the ability for local, manual override. An ultrasonic flowmeter would be installed on the outlet piping. The data would be used to control and position all flow control valves to achieve the desired discharge rate. All controls and sensor data would interface with a Programmable Logic Controller (PLC) and Supervisory Control and Data Acquisition (SCADA) network.

The guard valve(s) would be used to isolate the upstream tunnel from the rest of the outlet works appurtenances. The number and placement of the guard valve(s) would change as design progresses and the number of drawdown flow control valves is determined. If individual guard valves are used for each flow control valve, knife gate–type valves would be used for the drawdown flow control valves, and a 36-inch metal seated butterfly valve would be used for the stream maintenance flow control valve.

Downstream of each guard valve, a combination air release and vacuum-breaking valve would be installed so that a full vacuum would not form downstream, should operators close a guard valve prior to closing a flow control valve.

To facilitate maintenance work and tunnel inspection, a large tee-fitting (with a blind flange for access on one leg) would be provided upstream of the flow control valves piping assembly.

The stream maintenance flow control valve could be a 42-inch vertical sleeve valve, such as a Bailey Valve model B11. The vertical sleeve valve would discharge into its own energy dissipating stilling basin, which would drain to the creek via an overflow weir installed near the basin's top. The flow control valve's electric actuator would throttle the valve to an open position, based on the target release flowrate and the outlet structure's flowmeter readout. The size and type for this valve may change as design progresses.

The number, size, and style of drawdown control valves would be determined as design progresses. A potential arrangement could include two 84-inch-diameter fixed-cone valves. Each fixed-cone valve would branch off the outlet piping via reduced wye-fittings. In this arrangement, each flow control valve would have its own guard valve. Both fixed-cone valves would discharge into an energy-dissipating chamber, with outflow to Stone Corral Creek. Assuming a drawdown requirement of 2,500 cfs, the velocity through each 84-inch valve would be approximately 33 ft/s. Similar to the stream maintenance flow control valve, each of these valves would be throttled via electric actuation to meet the desired drawdown flow rate.

4.4 Instrumentation and Electrical Considerations

4.4.1 Instrumentation

At the dam crest control building, a PLC would interface to the HPU, valve controls, and monitoring sensors (e.g., reservoir level, gate position indication). If the HPU is a separate stand-alone PLC node, the two PLCs would be linked via ethernet, or a peer-to-peer communications network. The panel-mounted Human/Machine Interface (HMI) would be a minimum of 10 inches in size, with graphical user interface, touchscreen capability and Ethernet and / or serial (RS232 / 485) ports. The HMI would be a stand-alone device used for local monitoring and controls. Critical controls, including shutdowns, would be operated and monitored via hardwired pushbuttons and lights on the panel front. All lights for panel-mounted interfaces should be pushed to test.

Valve controls would encompass alarming for position switch failure (both open and closed positions active); and at a minimum, alarmed if watchdog timers for travel are exceeded.

All control system input-output would be designed to be failsafe in case of loss of power. All analog instrumentation would be monitored for high and low out-of-range signals to provide trouble alarms.

SCADA interfaces to external control houses would be achieved via radio. Studies should be performed as needed to detail radio requirements (unique frequencies versus spread spectrum versus Yagi element antennae).

4.4.2 Electrical

The source of electrical power would be coordinated with the balance of the project powerhouse and transmission features.

Typically, a single medium-voltage circuit would be brought to a small outdoor electrical area near the control building. A three-phase pad-mounted transformer filled with food-grade insulating liquid with primary protection and a secondary NEMA 3R outdoor circuit breaker would be used to derive utilization voltage. A secondary voltage of 480/277 volts is probably the most reasonable choice for the sizes and types of loads.

For redundancy, a permanently installed propane standby generator would be installed in the same area. Both the generator and the transformer outputs would feed an automatic transfer switch (ATS) in the control building.

In the control building, the output of the ATS would feed a Motor Control Center, which would have a combination of circuit breaker and motor controllers. Motor controller selection would be appropriate for the mechanical load. A small unitized 480: 120/240-volt single-phase power center would be provided to derive 120 volts for controls and convenience power.

Studies performed as part of future dam design phases would determine how power would be routed to both the dam crest control building and outlet control structure at the toe of the dam.

Backup power for the dam crest control building could also potentially be achieved using battery banks, depending on the load requirements from the HPU and its backup design (pre-charged nitrogen bottles or battery backup). Sizing and design of the primary and backup electrical systems would be developed as the design progresses.

4.5 Diversion Tunnel

4.5.1 Alignment

The Sites Dam Diversion Tunnel is in the left abutment of the dam to avoid conflict with the S-2 fault, located in the right abutment (See Figure 4-1). The feasibility level alignment was approximately 1,600 feet long, with a radius of 1,250 feet, and has a maximum cover of approximately 320 feet. The upstream tunnel invert elevation is approximately 275 feet, and slopes downwards until it terminates at the downstream portal at an elevation of approximately 265 feet.

Additional geotechnical investigations and analysis would be required to optimize the alignment.

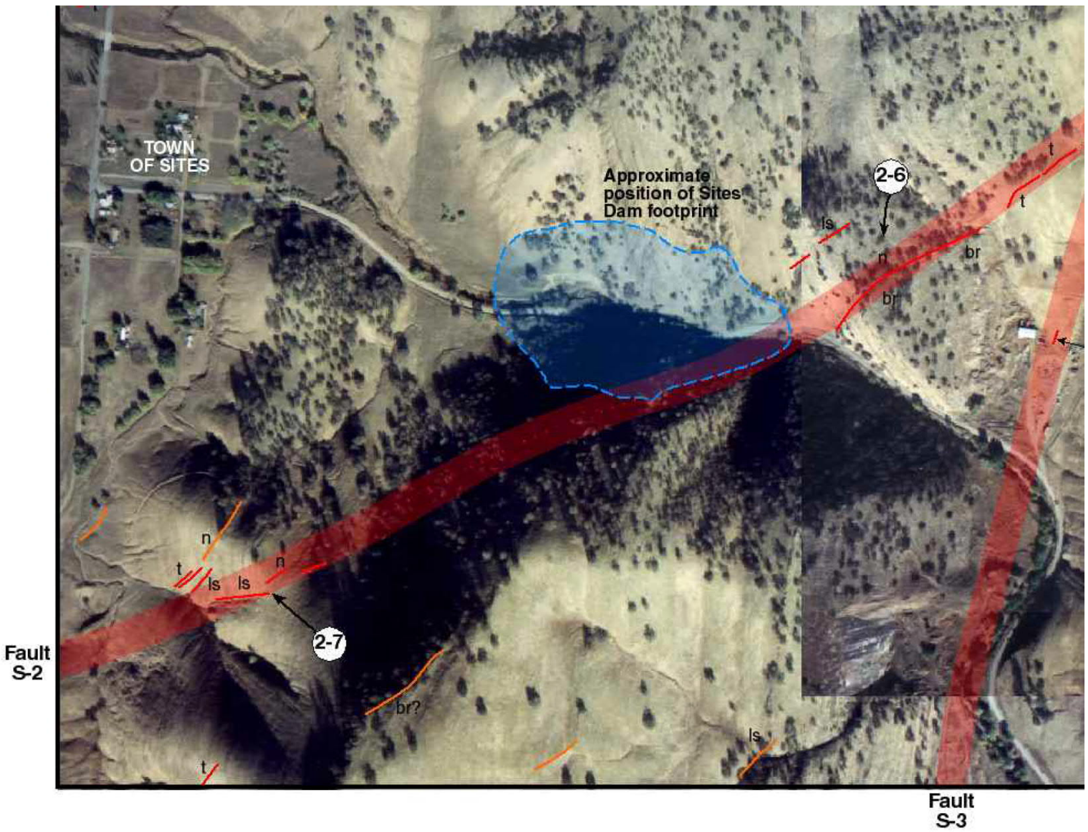

Figure 4-1. Faulting Map of Sites Diversion Tunnel Area (William Lettis, 2002)

4.5.2 Interpreted Geotechnical and Geological Conditions

The bedrock underlying the Sites Dam Diversion Tunnel is part of the Great Valley sequence consisting of the Boxer and Cortina Formations. The Boxer Formation is a Cretaceous age thinly bedded mudstone and siltstone with scattered thin to medium sandstone interbeds and/or conglomerate lenses. The Cortina Formation is a Cretaceous age sandstone with moderate to thick mudstone and siltstone interlayers. Strike of the bedding is roughly north-south, nearly normal to the tunnel alignment, with an average dip of about 50 degrees east. A prominent joint set, Set A, trends about east-west, with near-vertical dips.

A previously performed geotechnical investigation to characterize the conditions at the project site was focused on Sites Dam; geotechnical borings were not performed along the proposed tunnel alignment. Only boring DH-302 was in the left abutment; the remainder were along the channel. Based on this limited information, it has been assumed so far that 50 percent of the tunnel would be in the Boxer Formation, and 50 percent would be in the Cortina Formation. Colluvial deposits were encountered in the borings, but were not present in any appreciable amount. Two landslides were in the approximate location of the proposed left abutment, but it is assumed that they would be excavated as part of the dam construction.

Pressure tests were performed in the boring in vicinity of the proposed left abutment, and it did not show any take at lower depths (84 to 95 feet; a hydraulic conductivity of 0.23 ft/day). Groundwater above the tunnel grade was encountered in the boring in the left abutment (elevation 343 feet), indicating that groundwater would be encountered during tunnel excavation.

4.5.3 Tunnel Construction Approach

A combination of drill-and-blast and roadheader excavation is assumed to be the construction method for the Diversion Tunnel. Drill-and-blast would be used in areas where the rock strength is higher, and the use of a roadheader is inefficient. Roadheader excavation would be used in soft to moderately strong rocks with an unconfined compressive strength less than 15,000 pounds per square inch. In both cases, the road header would be used to muck up the excavated material into mining cars.

4.5.4 Sizing

Sizing the Sites outlet tunnel is dependent on the maximum flows and velocity limitations (see Design Criteria TM [AECOM 2020b]). If lined with reinforced concrete, it would have a finished inside diameter of 12 feet. The tunnel internal diameter could be reduced to 9.5 feet if the final lining was designed so that maximum permittable velocities could be 35 ft/s.

4.5.5 Tunnel Support Systems

The Diversion Tunnel is assumed to be constructed using a two-pass lining system. The first pass (initial support) would be installed as part of the excavation process to support the ground and provide a safe work environment. The second pass or final lining would be installed once the tunnel excavation is complete, and would accommodate all designs loads while serving as the water conveyance line.

4.5.5.1 Tunnel Excavation and Initial Support System

The excavation is assumed to be a 16-foot-diameter horseshoe-shaped tunnel.

After excavation is completed for each mining cycle, initial support is installed immediately to stabilize the ground and provide a safe working environment for the subsequent construction activities. Based on the geotechnical conditions, steel sets and/or rock bolts are the assumed initial support system for the tunnels. Between the steel sets and/or rock bolts, timber or shotcrete lagging is assumed to be used to control raveling, and as a structural member to share ground loads with the steel supports. It is assumed that 12 inches of steel-fiber-reinforced shotcrete would be required to support the ground, in addition to the steel sets and/or rock bolts.

4.5.5.2 Final Lining

Once excavation is completed for the Diversion Tunnel, the final lining would be installed. The final lining is required to prevent rock fallout and erosion, and minimize seepage into the surrounding rock. Concrete lining also provides a smooth interior surface that reduces head loss. However, where the confining weight of rock cover over the tunnel is less than the internal pressure, a steel liner is incorporated into the concrete lining to provide tunnel stability and prevent leakage.

The final lining for the Diversion Tunnel is assumed to be 12-inch-thick reinforced cast-in-place concrete with an internal diameter of 12 feet for the majority of the tunnel. Where insufficient cover is present, the lining is assumed to ¾-inch-thick steel carrier pipe. The steel liner would extend to a point where the overburden is sufficiently thick to provide the needed confinement. The annulus between the steel carrier pipe and initial support would be backfilled with concrete. Contact grouting would be required afterwards to ensure all voids have been filled. The steel carrier pipe would be constructed using butt-welded joints or modified lap-welded joints that are slightly flared, established from within the tunnel. Waterproofing with a membrane between the initial lining and final lining, or by employing post-construction cut-off grouting, is also assumed to be required.

4.5.6 Groundwater Control and Pre-Excavation Grouting

Based on the boring performed, groundwater is expected to infiltrate into the tunnel during construction. The water would drain onto the invert, and if tunnel excavation is proceeding upstream, would flow towards the tunnel portal. If the excavation is occurring downstream, pumping would be required to drain the tunnel.

Probe-hole drilling and pre-excavation grouting are assumed to be necessary throughout tunnel construction to help control the groundwater and stabilize the ground. Probe hole drilling involves drilling a 4- to 8-inch-diameter hole ahead of the tunnel heading to intersect water-bearing features. Subsequently, a packer is installed into the hole, and cement grout with admixtures is injected into the ground, typically at a pressure that locally fractures the rock, to push back the water and seal up the water-bearing joints or discontinuities.

Weep holes would be required to reduce hydrostatic head behind the shotcrete initial support.

4.5.7 Portals

There would be two portals, which are assumed to be temporary features used to commence and terminate excavation of the Diversion Tunnel. The downstream portal would likely be used as a launch portal for the tunnels and would be the point of access/egress during construction. The upstream portal is assumed to be the daylighting portal. It is assumed that the portals would be supported using rock anchors and steel-fiber-reinforced shotcrete. Canopy support using canopy tubes or spilling would be required at the break-in and break-out locations.

5.0 Construction Considerations

Construction of the Golden Gate Dam diversion and the Sites Dam diversion/outlet would disturb on the order of 5 acres in the Reservoir inundation area, and a similar area outside of it at the downstream tunnel portal. The construction disturbance would consist of the footprints of the intake structure; energy-dissipation measures on the downstream side; the tunnel portals at Sites; the materials, spoils, and equipment staging areas; and access roads. Excavation for the diversion pipe at Golden Gate dam should be covered by the dam's footprint.

Construction activities would consist of:

- Dewatering of the construction site and an on-site treatment facility
- Trenching for the diversion pipeline at Golden Gate Dam
- Hillside excavation for the downstream and upstream tunnel portals for Sites Dam
- Tunneling and hauling tunnel muck to a disposal area
- Excavation for the intake structures and downstream energy-dissipation structures
- Building the structures
- Finished grading and site clean-up.

6.0 Recommended Additional Geotechnical Investigation

Further geotechnical investigation is recommended to characterize the subsurface conditions, geotechnical properties, and hydrogeological conditions along the Diversion Tunnel. The investigations would include geotechnical borings; downhole televiewer logging (for orientation of rock discontinuities); and hydraulic conductivity testing (packer testing), trenching, seismic refraction surveys, and groundwater monitoring wells. The investigation footprint for the Diversion Tunnel would encompass the area around each portal and along each tunnel alignment. It is assumed that a boring would be required every 500 feet along the Diversion Tunnel Alignment (to be confirmed), with each boring extending two tunnel diameters below the tunnel invert.

A geotechnical investigation would be required to map the fault(s) adjacent to Sites Dam, and to confirm that the Diversion Tunnel alignment minimizes fault crossings. The footprint of the investigation would be along each mapped fault, as well as the area between the faults and tunnel. The investigations would include geotechnical borings, trenching, seismic refraction surveys, and groundwater monitoring wells.

7.0 Plan Sheets

Table 7-1 lists the diversion facility drawings that are presented under separate submittal.

Table 7-1. List of Diversion Facility Drawings

Drawing No.	Main Title	Subtitle
STS-365-C-2601	Sites Reservoir Diversion Facility	Golden Gate Dam Bypass Pipe Plan
STS-366-C-2601	Sites Reservoir Diversion Facility	Sites Dam Creek Diversion Tunnel Plan
STS-366-C-3601	Sites Reservoir Diversion Facility	Sites Dam Creek Diversion Tunnel Profile
STS-366-C-3602	Sites Reservoir Diversion Facility	Sites Dam Diversion Tunnel Initial Support and Final Lining

8.0 Estimated Quantities and Disturbance Areas

The estimated diversion construction quantities and disturbance areas due to construction are summarized in Appendix C. These quantities are likely to change as the work is advanced.

9.0 References

AECOM (2020a). Funks and Stone Corral Creeks Reservoir Operating Elevations and Emergency Release Management, Draft TM. May.

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AECOM (2020c). Basis of Design – Main Dams and Saddle Dams, Task Order No.1, Task HR2.92, Draft TM. July.

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Appendix A

100-Year Flood Flows – Funks Creek

Storage needed for Funks Creek
33 hour 100 year flood

Time [h]	Flow In [cfs]	Volume In [cf]	Flow Out [cfs]	Volume Out [cf]	Storage Required [cf]	Storage Required [ac-ft]
0	0	0	0.00	0	0	
0.5	105	188485.6	0.00	0	188485.6	
1	197	354149.8	0.00	0	354149.8	8.1
1.5	396	712840.4	0.00	0	1066990	24.5
2	736	1325192	0.00	0	2392183	54.9
2.5	1278	2300776	0.00	0	4692958	107.7
3	2208	3974920	0.00	0	8667878	199.0
3.5	4201	7562107	0.00	0	16229985	372.6
4	6667	12001123	0.00	0	28231109	648.1
4.5	9036	16264363	0.00	0	44495471	1021.5
5	11283	20308550	0.00	0	64804022	1487.7
5.5	11813	21262712	0.00	0	86066733	1975.8
6	10994	19788652	0.00	0	1.06E+08	2430.1
6.5	9608	17294569	0.00	0	1.23E+08	2827.1
7	7991	14383900	0.00	0	1.38E+08	3157.3
7.5	6387	11496879	0.00	0	1.49E+08	3421.3
8	5329	9593086	0.00	0	1.59E+08	3641.5
8.5	4429	7971573	0.00	0	1.67E+08	3824.5
9	3816	6868415	0.00	0	1.73E+08	3982.2
9.5	3290	5921769	0.00	0	1.79E+08	4118.1
10	2856	5140792	0.00	0	1.85E+08	4236.1
10.5	2522	4539097	0.00	0	1.89E+08	4340.3
11	2297	4135158	0.00	0	1.93E+08	4435.3
11.5	2097	3775194	0.00	0	1.97E+08	4521.9
12	1919	3453609	0.00	0	2E+08	4601.2
12.5	1762	3171656	0.00	0	2.04E+08	4674.0
13	1627	2928839	0.00	0	2.07E+08	4741.3
13.5	1536	2765597	0.00	0	2.09E+08	4804.8
14	1562	2811863	0.00	0	2.12E+08	4869.3
14.5	1469	2644679	0.00	0	2.15E+08	4930.0
15	1383	2489738	0.00	0	2.17E+08	4987.2
15.5	1308	2353891	0.00	0	2.2E+08	5041.2
16	1246	2243219	0.00	0	2.22E+08	5092.7
16.5	1185	2133431	0.00	0	2.24E+08	5141.7
17	1127	2029311	0.00	0	2.26E+08	5188.3
17.5	1070	1926074	0.00	0	2.28E+08	5232.5
18	1011	1819747	0.00	0	2.3E+08	5274.3
18.5	960	1728658	0.00	0	2.31E+08	5314.0
19	915	1647016	0.00	0	2.33E+08	5351.8
19.5	870	1566651	0.00	0	2.35E+08	5387.7
20	831	1496247	0.00	0	2.36E+08	5422.1
20.5	792	1426212	0.00	0	2.38E+08	5454.8
21	749	1348742	0.00	0	2.39E+08	5485.8
21.5	715	1286168	0.00	0	2.4E+08	5515.3

Storage needed for Funks Creek
33 hour 100 year flood

Time [h]	Flow In [cfs]	Volume In [cf]	Flow Out [cfs]	Volume Out [cf]	Storage Required [cf]	Storage Required [ac-ft]
22	680	1223422	0.00	0	2.41E+08	5543.4
22.5	649	1168086	0.00	0	2.43E+08	5570.2
23	618	1112750	0.00	0	2.44E+08	5595.8
23.5	587	1057414	0.00	0	2.45E+08	5620.0
24	563	1012827	0.00	0	2.46E+08	5643.3
24.5	538	968558.2	0.00	0	2.47E+08	5665.5
25	513	924289.5	0.00	0	2.48E+08	5686.7
25.5	489	880020.8	0.00	0	2.49E+08	5706.9
26	466	838722	0.00	0	2.49E+08	5726.2
26.5	440	792316.9	0.00	0	2.5E+08	5744.4
27	421	757448.2	0.00	0	2.51E+08	5761.8
27.5	399	717816.6	0.00	0	2.52E+08	5778.3
28	380	683609.8	0.00	0	2.52E+08	5793.9
28.5	361	650408.2	0.00	0	2.53E+08	5808.9
29	344	619661.4	0.00	0	2.54E+08	5823.1
29.5	332	597527.1	0.00	0	2.54E+08	5836.8
30	316	567957.3	0.00	0	2.55E+08	5849.9
30.5	300	539908.2	0.00	0	2.55E+08	5862.3
31	282	506706.6	0.00	0	2.56E+08	5873.9
31.5	263	473505.1	0.00	0	2.56E+08	5884.8
32	249	447738.9	0.00	0	2.57E+08	5895.0
32.5	230	414537.4	0.00	0	2.57E+08	5904.6

Appendix B

100-Year Flood Flows – Stone Corral Creek

Stone Corral Creek
33 hour 100 year flood

Time [h]	Flow In [cfs]	Volume In [cf]	Flow Out [cfs]	Volume Out [cf]	Storage Required [cf]	Storage Required [ac-ft]
0	0	0	0	0	0	
0.5	92	165699.8	100	180000	0	
1	206	370201.9	100	180000	190201.9	4.4
1.5	486	874249.3	100	180000	884451.2	20.3
2	970	1745511	100	180000	2449963	56.2
2.5	2048	3687234	500	900000	5237196	120.2
3	4200	7559606	500	900000	11896802	273.1
3.5	6417	11550998	500	900000	22547800	517.6
4	8398	15115764	500	900000	36763564	844.0
4.5	8441	15193800	1000	1800000	50157364	1151.5
5	7340	13211392	1000	1800000	61568755	1413.4
5.5	5855	10538283	1000	1800000	70307039	1614.0
6	4448	8006543	1000	1800000	76513582	1756.5
6.5	3522	6338815	1000	1800000	81052397	1860.7
7	2872	5170415	2000	3600000	82622811	1896.8
7.5	2377	4278301	2000	3600000	83301112	1912.3
8	1997	3594216	2000	3600000	83295328	1912.2
8.5	1739	3130918	2000	3600000	82826246	1901.4
9	1547	2784699	2000	3600000	82010945	1882.7
9.5	1382	2487151	2000	3600000	80898096	1857.2
10	1246	2242483	2000	3600000	79540579	1826.0
10.5	1125	2024157	1500	2700000	78864736	1810.5
11	1142	2055953	1500	2700000	78220690	1795.7
11.5	1057	1902743	1500	2700000	77423432	1777.4
12	980	1763421	1500	2700000	76486853	1755.9
12.5	919	1654243	1500	2700000	75441096	1731.9
13	863	1552663	1500	2700000	74293759	1705.6
13.5	808	1455037	1000	1800000	73948796	1697.6
14	754	1357721	1000	1800000	73506517	1687.5
14.5	705	1269719	1000	1800000	72976236	1675.3
15	663	1193175	1000	1800000	72369411	1661.4
15.5	624	1123298	1000	1800000	71692709	1645.8
16	587	1056407	100	180000	72569117	1666.0
16.5	549	987367.2	100	180000	73376484	1684.5
17	516	928045.3	100	180000	74124529	1701.7
17.5	485	872958.6	100	180000	74817488	1717.6
18	457	821732.1	100	180000	75459220	1732.3
18.5	428	770505.6	100	180000	76049725	1745.9
19	405	729307.8	100	180000	76599033	1758.5
19.5	382	688326.5	100	180000	77107360	1770.1
20	360	647345.3	100	180000	77574705	1780.9
20.5	338	609199.5	100	180000	78003904	1790.7
21	315	567626.6	100	180000	78391531	1799.6
21.5	297	535238.5	100	180000	78746770	1807.8

Stone Corral Creek
33 hour 100 year flood

Time [h]	Flow In [cfs]	Volume In [cf]	Flow Out [cfs]	Volume Out [cf]	Storage Required [cf]	Storage Required [ac-ft]
22	278	500736.3	100	180000	79067506	1815.1
22.5	261	470000.4	100	180000	79357506	1821.8
23	247	444560.1	100	180000	79622066	1827.9
23.5	233	419365.4	100	180000	79861432	1833.4
24	219	394048	100	180000	80075480	1838.3
24.5	202	363312.1	100	180000	80258792	1842.5
25	186	335505.3	100	180000	80414297	1846.1
25.5	171	307258.7	100	180000	80541556	1849.0
26	154	276522.8	100	180000	80638079	1851.2
26.5	137	245786.8	100	180000	80703865	1852.7
27	120	215613.5	100	180000	80739479	1853.5
27.5	102	184344	100	180000	80743823	1853.6
28	85	153579.1	100	180000	80717402	1853.0
28.5	68	122843.1	100	180000	80660245	1851.7
29	51	92107.2	100	180000	80572352	1849.7
29.5	40	72835.64	100	180000	80465188	1847.2
30	35	62590.33	100	180000	80347778	1844.5
30.5	29	52345.03	100	180000	80220123	1841.6
31	23	42099.72	100	180000	80082223	1838.4
31.5	18	31854.41	100	180000	79934077	1835.0
32	12	21609.1	100	180000	79775687	1831.4
32.5	6	11363.79	100	180000	79607050	1827.5

Appendix C

Estimated Quantities and Disturbance Areas

FINAL, 8/20/2020

SITES RESERVOIR PROJECT
SITES DAM DIVERSION/OUTLET
QUANTITY ESTIMATES FOR EARTHWORK, CONCRETE AND RIPRAP

ITEM	QUANTITY	UNIT	FACTOR OF SAFETY/ BULKING	ROUNDED QUANTITY	NOTES
EXCAVATION VOLUMES					
Upstream portal excavation volume	15,723	yd^3	1.7	26,800	
Tunnel excavation volume	15,193	yd^3	1.3	19,800	Tunnel is 1,590 ft long
Downstream portal excavation volume	12,416	yd^3	1.7	21,200	
TOTAL		yd^3		69,390	Bulking factor for excavated materials is 1.7
FACILITY FOOTPRINTS					
Upstream portal excavation footprint	0.5	acres	2	1	Inside the reservoir footprint. Intake structure included in footprint
Downstream portal excavation footprint	0.6	acres	2	2	Downstream of the reservoir. Outlet structure included in footprint
Disposal area for excavated materials	2	acres	2	4	Materials disposed in creek, inside the reservoir footprint.
TOTAL		acres		7	5 acres in the reservoir footprint, 2 acres downstream
CONCRETE VOLUMES					
Intake structure concrete aggregate	900	yd^3	1.2	1,100	
Tunnel lining (shotcrete and concrete) aggregate	6751	yd^3	1.2	8,200	
Outlet structure concrete aggregate	900	yd^3	1.2	1,100	
TOTAL		yd^3		10,400	Assuming 90% aggregate in the concrete
RIPRAP VOLUME	185	yd^3	1.2	230	Assuming 1000 sq. ft with 5 ft thick riprap at outlet structure

Assumptions/Notes:

1) All excavated materials would be disposed of in the upstream thalweg (below El. 300).
2) Tunnel spoils and excavated material from the outlet structure would be hauled to the upstream thalweg via the tunnel.
3) Access roads during construction included in the Sites Dam estimates.
4) Stockpile and staging areas included in Sites dam footprint.

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.

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

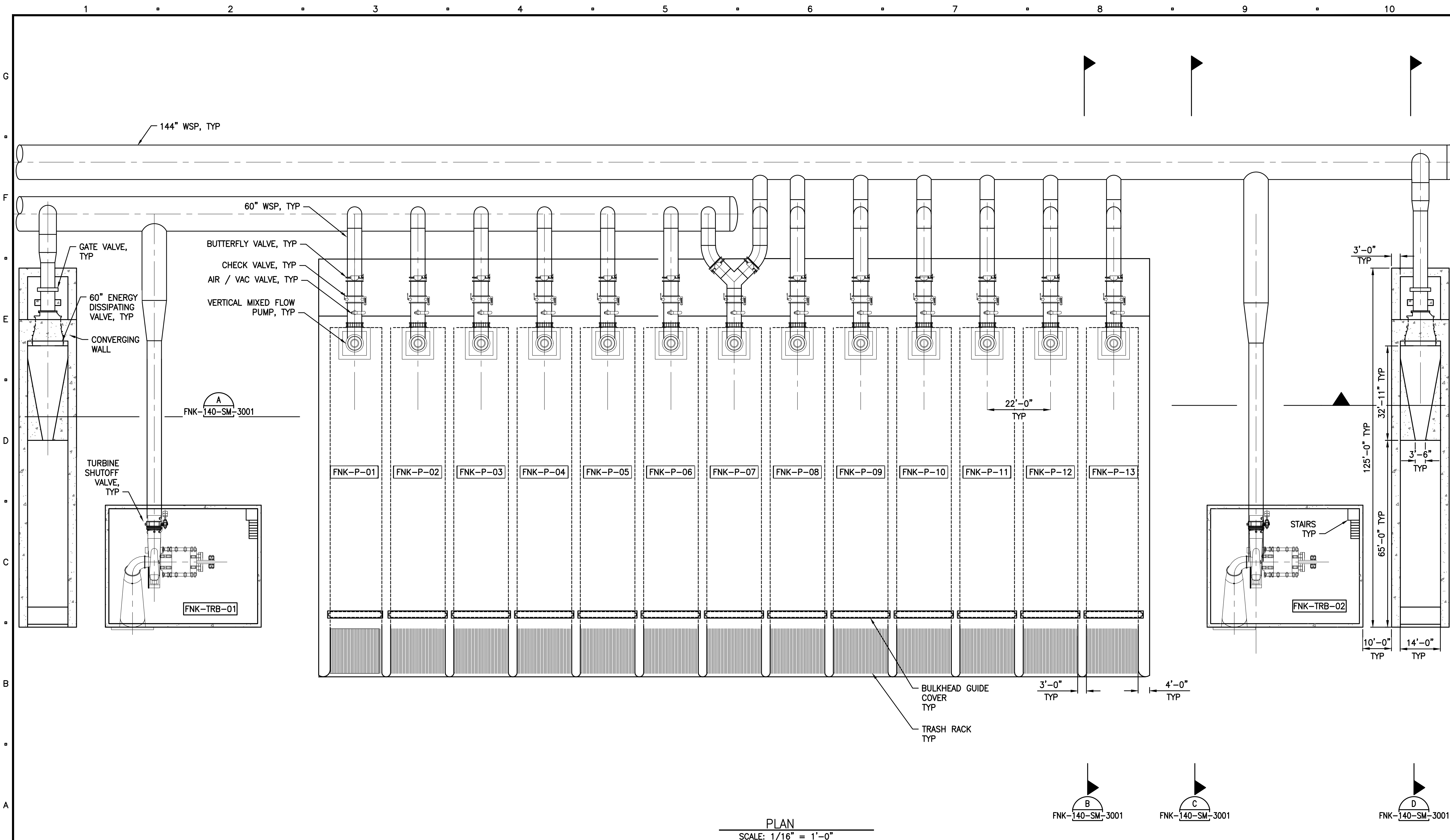
The pump-system curve (Figure 6) shows representative pumps operating in parallel and at a common pump speed (all pumps on ASDs and all pumps driven at the same speed), with flow coverage within the POR across a flow rate of 100 to 1,800 cfs.

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"

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

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
0 1"

DRAWING NO.
FNK-140-SM-2001

DRAFT – PROJECT FEASABILITY STUDY – NOT FOR CONSTRUCTION

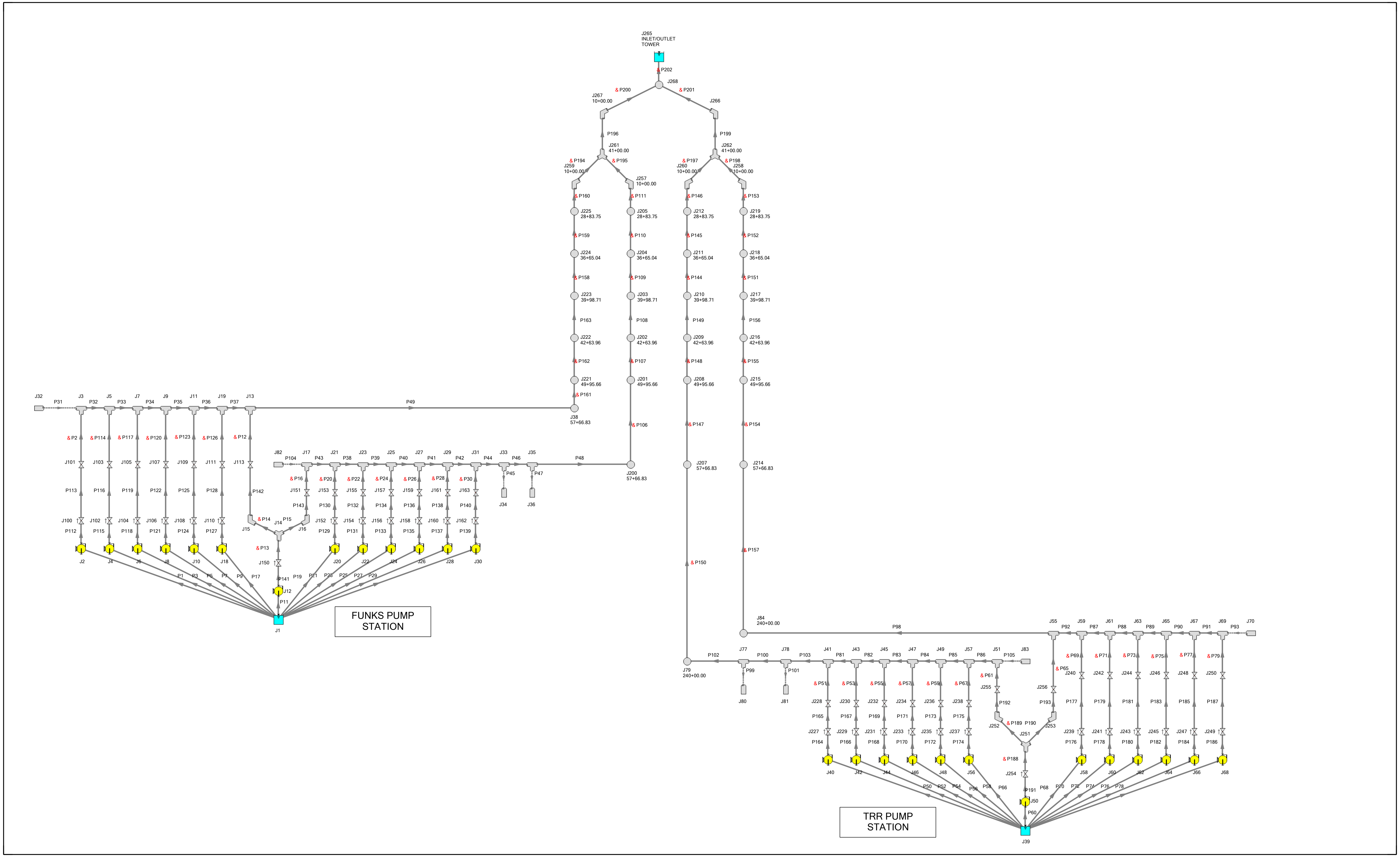


FIGURE 2: HYDRAULIC MODEL SCHEMATIC

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 #: RPM 505	E60367WMPF
		Model:	VCW
		Size:	67WMCC - 2.82

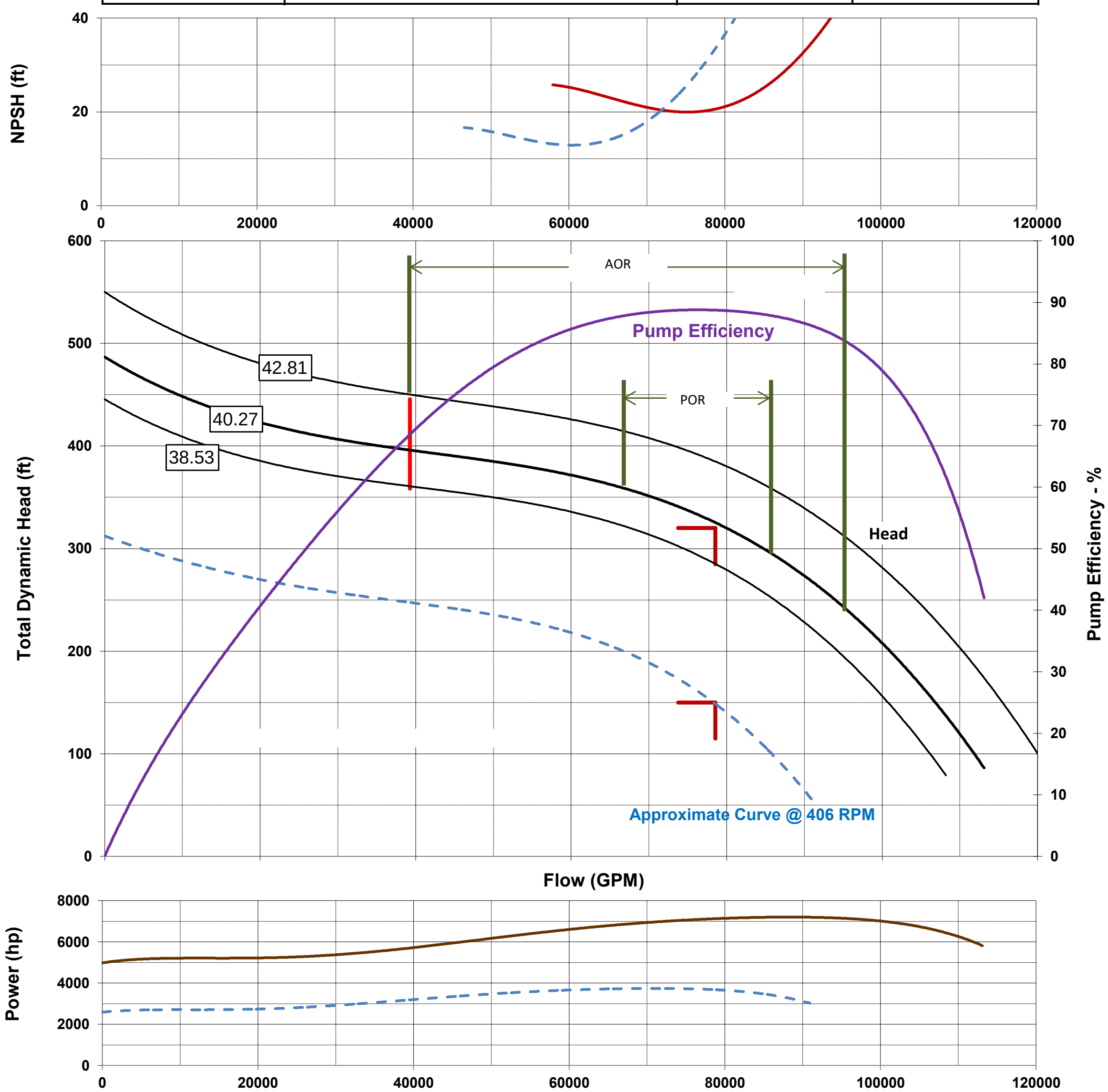


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FIGURE 3: FUNKS PUMP CURVE

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

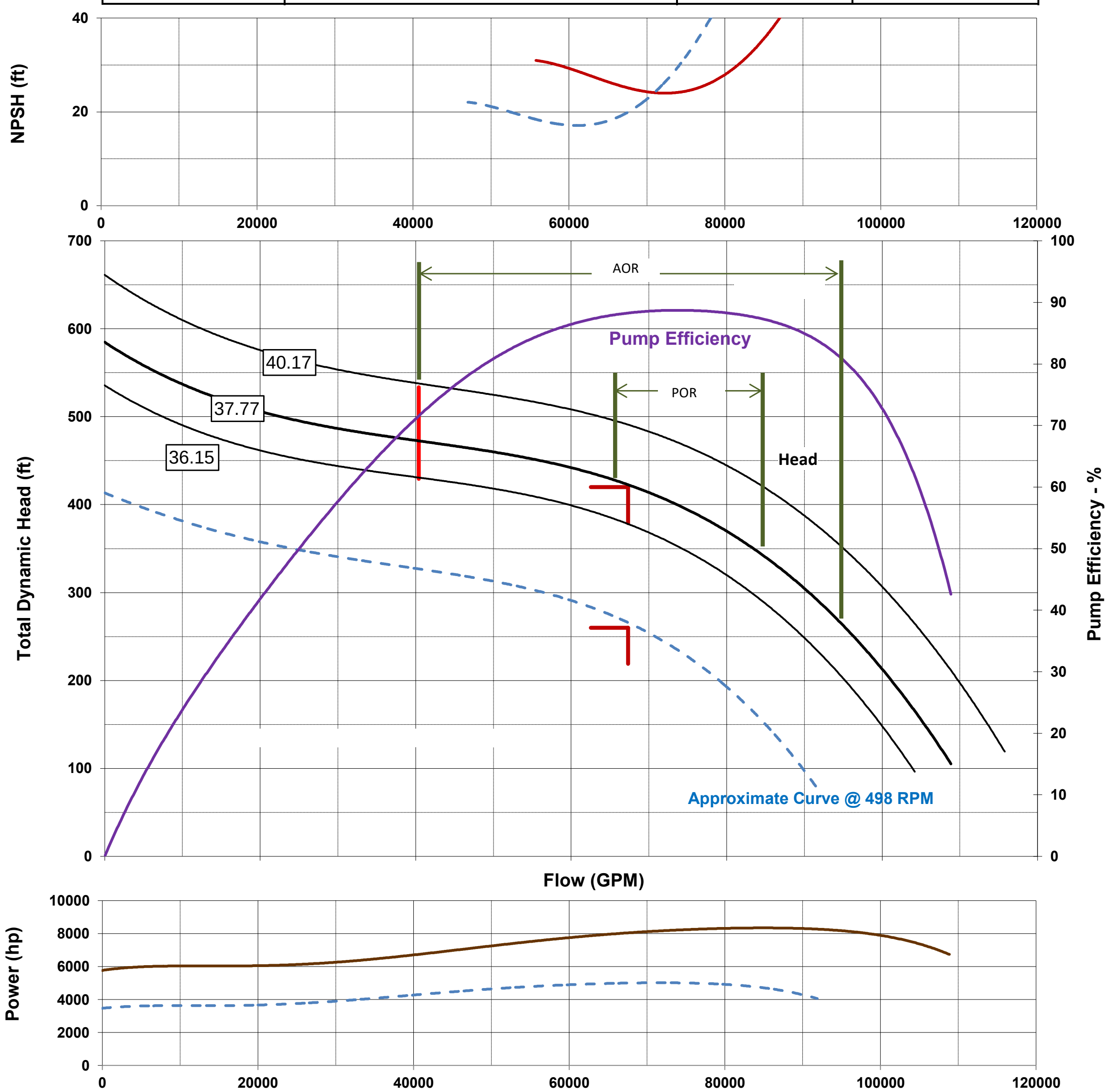

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FIGURE 4: TRR PUMP CURVE

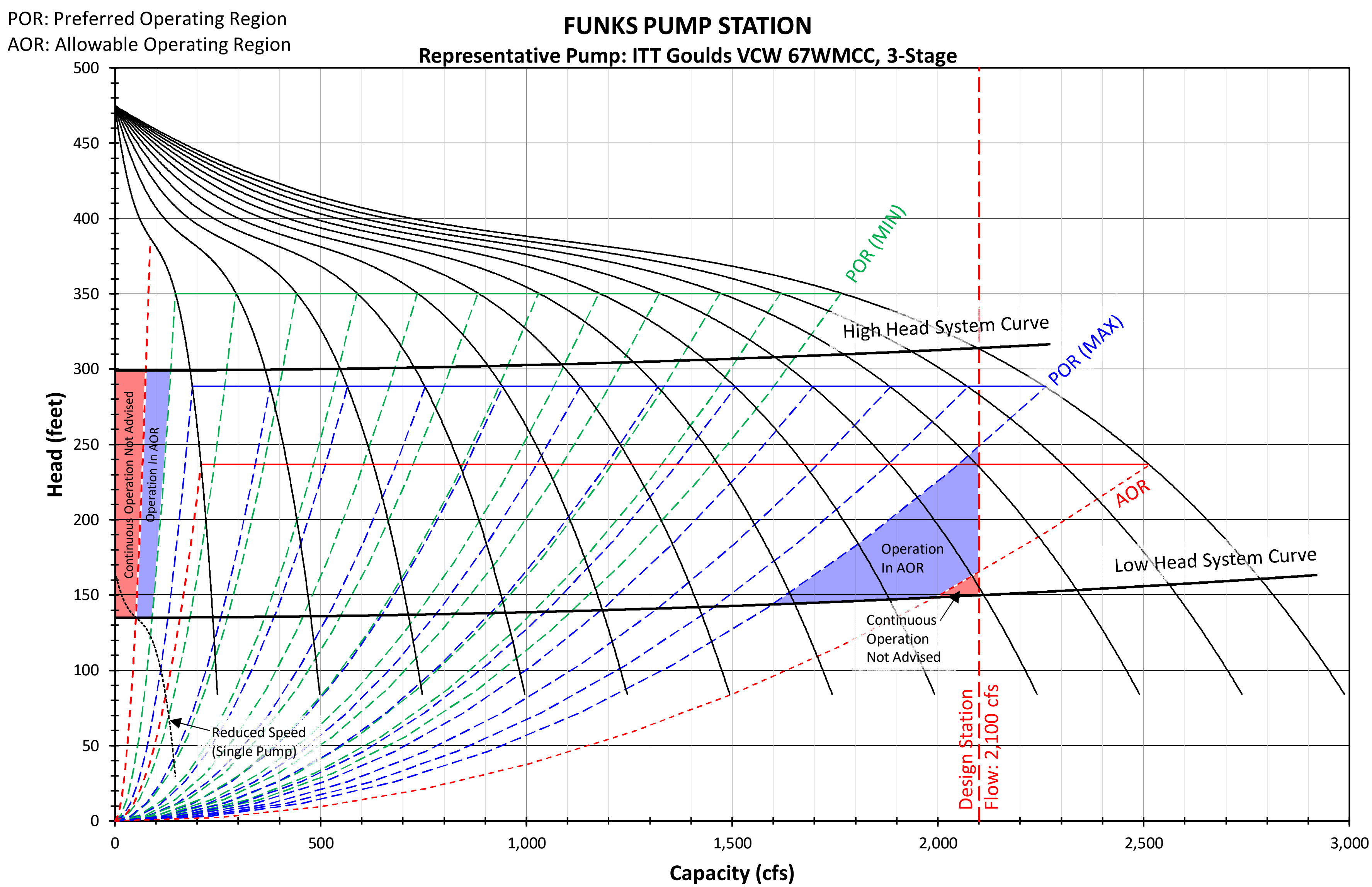


FIGURE 5: FUNKS PUMPING SYSTEM CURVES

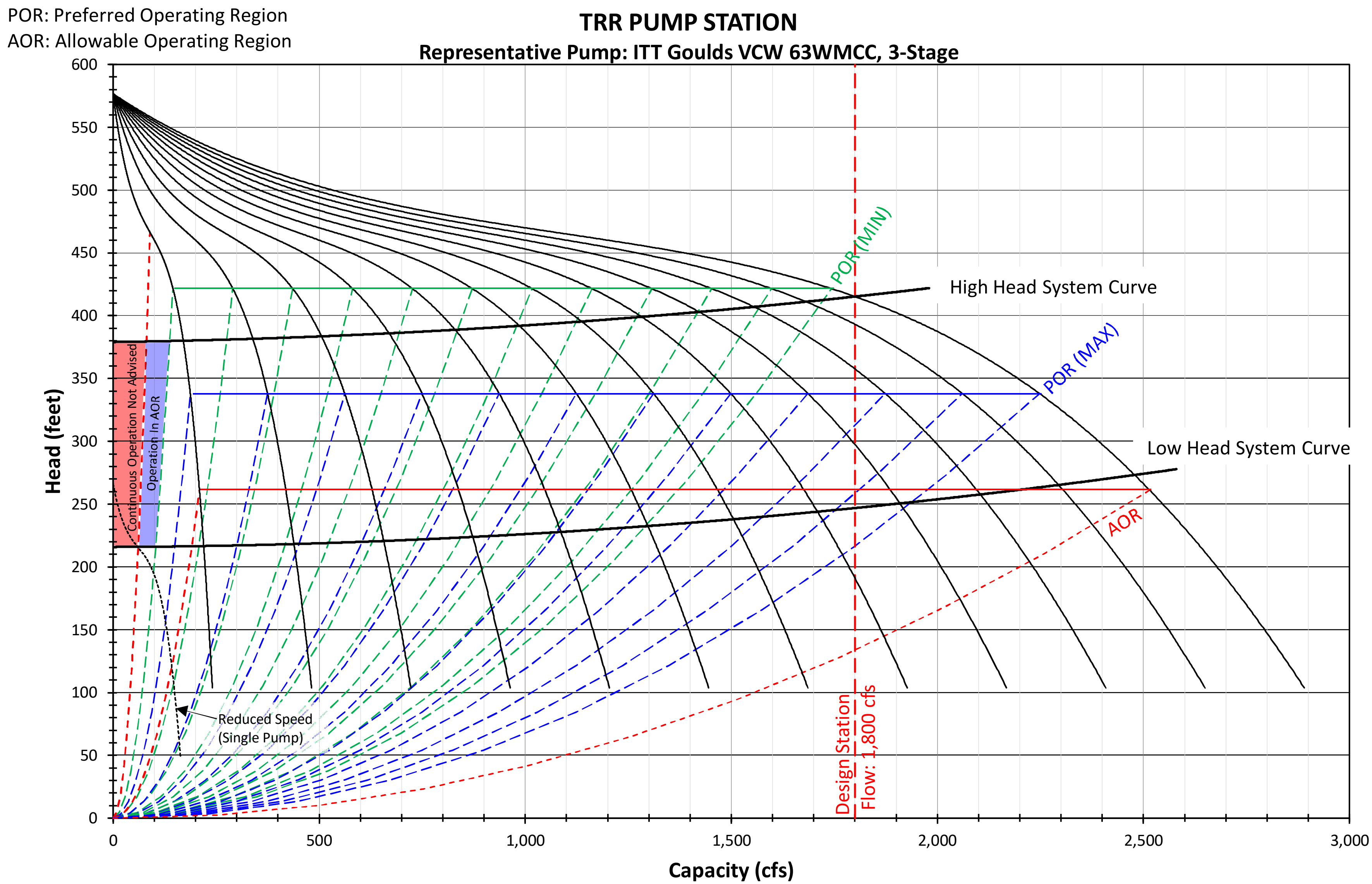
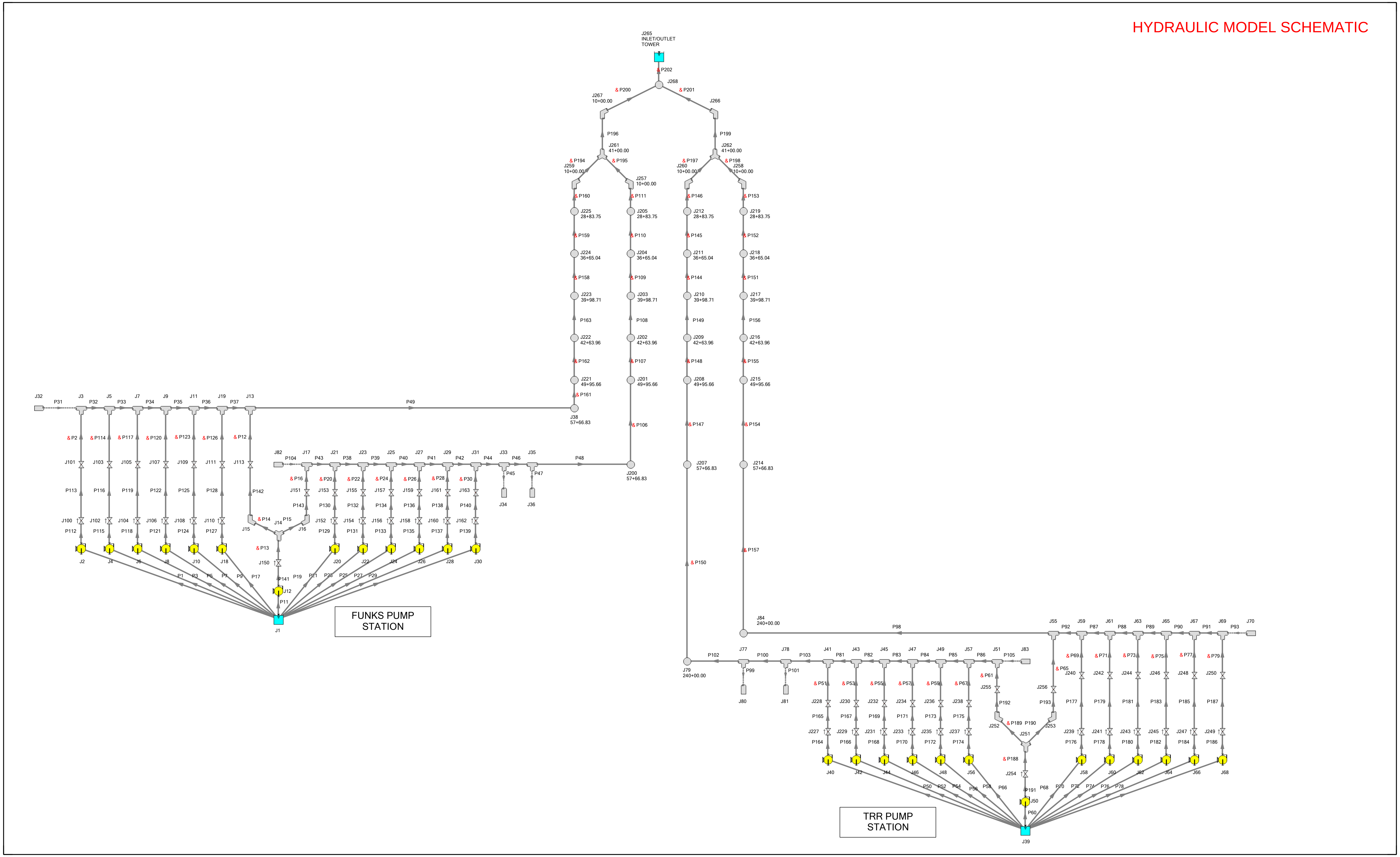


FIGURE 6: TRR PUMPING SYSTEM CURVES

Attachment A
Hydraulic Model Data


HYDRAULIC MODEL SCHEMATIC
J265
INLET/OUTLET
TOWER
FUNKS PUMP
STATION
TRR PUMP
STATION

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

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.

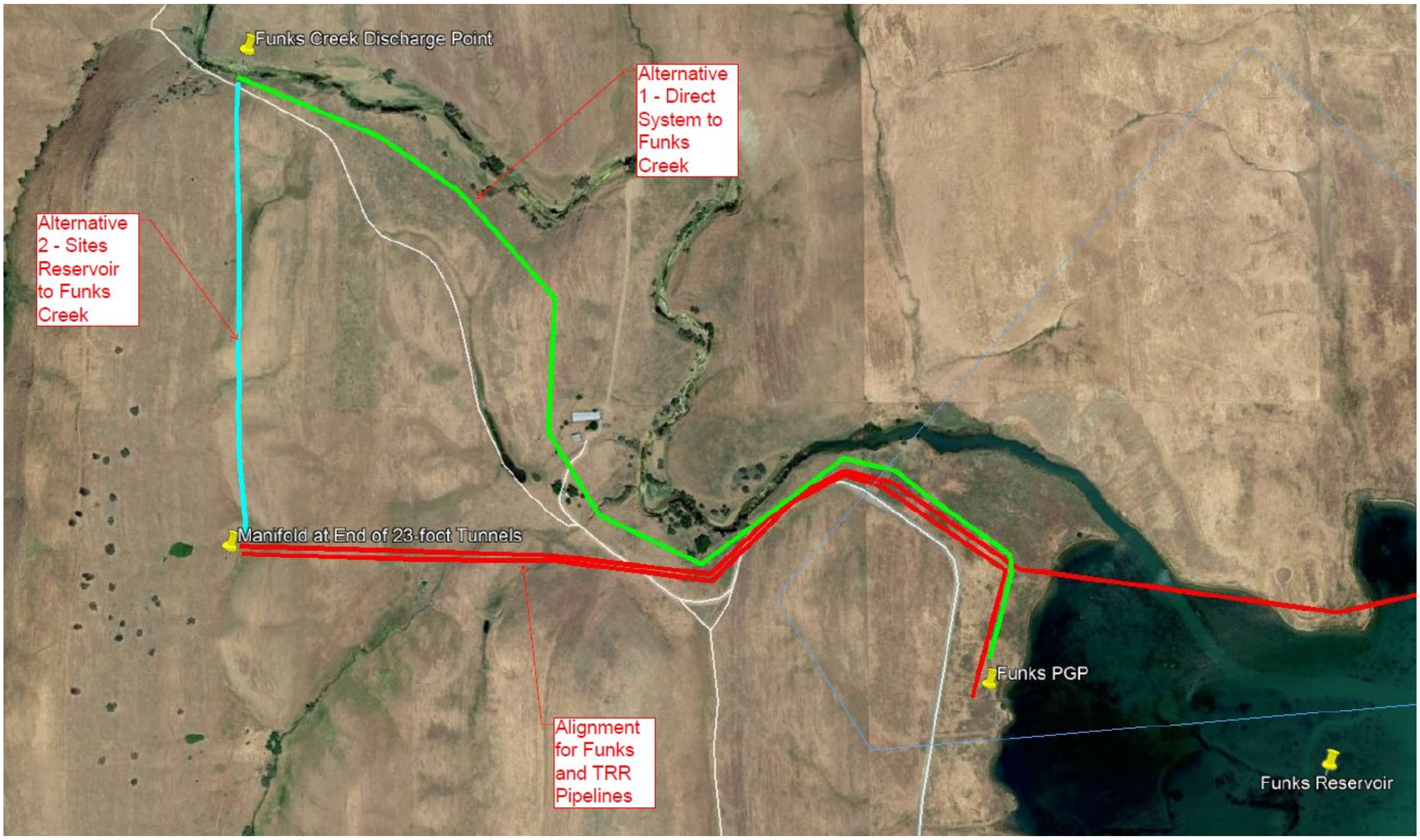


FIGURE 1: FUNKS CREEK ALTERNATIVES

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.

Gianelli PGP Comparison to Sites PGPs
Technical Memorandum (Final)

Sites Program Management Team

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.

Harrington Pipeline Alignment Analysis
Technical Memorandum (Final)

Sites Program Management Team

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

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.