Subject:	**Capacity Analysis for Preliminary Tunnel Diameter Selection (Final Draft)**
Project feature:	Projectwide
Prepared for:	California Department of Water Resources (DWR) / Delta Conveyance Office (DCO)
Prepared by:	Delta Conveyance Design and Construction Authority (DCA)
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1. Purpose

The purpose of this technical memorandum (TM) is to perform a preliminary analysis for tunnel diameter selection for the Delta Conveyance Project (Project). Under the proposed project, the new north Delta facilities would be sized to convey up to 6,000 cubic feet per second (cfs) of water from the Sacramento River to the State Water Project (SWP) facilities in the south Delta. DWR is also considering different alternatives with capacities that range from 3,000 to 7,500 cfs. Within this established range of Project design flow capacities, a hydraulics and capacity analysis of tunnel diameter options was performed for three specific design flow capacity options of 4,500, 6,000, and 7,500 cfs at the direction of the DCA. Based on the results of this evaluation, this TM recommends the proposed maximum tunnel flow velocity criteria and corresponding minimum tunnel inside diameter (ID) at each maximum design flow capacity of 4,500, 6,000, and 7,500 cfs.

1.1 Background

Potential tunnel diameters were previously identified, considered, and evaluated in support of the WaterFix project. Those tunnels would connect the Sacramento River intakes (intakes) to an intermediate forebay (IF) and extend from the IF to the pumping plant. Note, the original twin tunnels of WaterFix from the IF to the pumping plant have been withdrawn in favor of a single tunnel and subsequent analysis to this TM has resulted in elimination of the IF.

This evaluation considered separate maximum Project diversion flows of 4,500 and 6,000 -cfs with up to two intakes, and 7,500 cfs with up to three intakes. Tunnel flow velocities and corresponding hydraulic head loss and transient-surge conditions were examined within the Project diversion flow range to establish a range of technically feasible candidate tunnel diameters , including their effects on other Project feature configurations (that is, size and operating depths of hydraulic structures).

Hydraulic performance as well as preliminary construction and operating cost analyses were conducted for each candidate tunnel diameter considered and compared to a 44-foot-diameter tunnel. The 44-foot-diameter tunnel was an early and preliminary baseline for a single tunnel analysis based on the original WaterFix hydraulic design criteria.

Figure 1 provides a schematic of the Project configuration considered in this analysis. This evaluation was conducted between the intakes and the new Pumping Plant wet well within Figure 1’s dashed boundary. Evaluation of the southern tunnels connecting the Southern Forebay to the existing approach channels of the existing export Harvey O. Banks Pumping Plant (Banks) and potentially C.W. “Bill” Jones Pumping Plant

(Jones) would be conducted as part of a separate hydraulic analysis of the South Delta Conveyance Facilities (SDCF) System.

Project Schematic

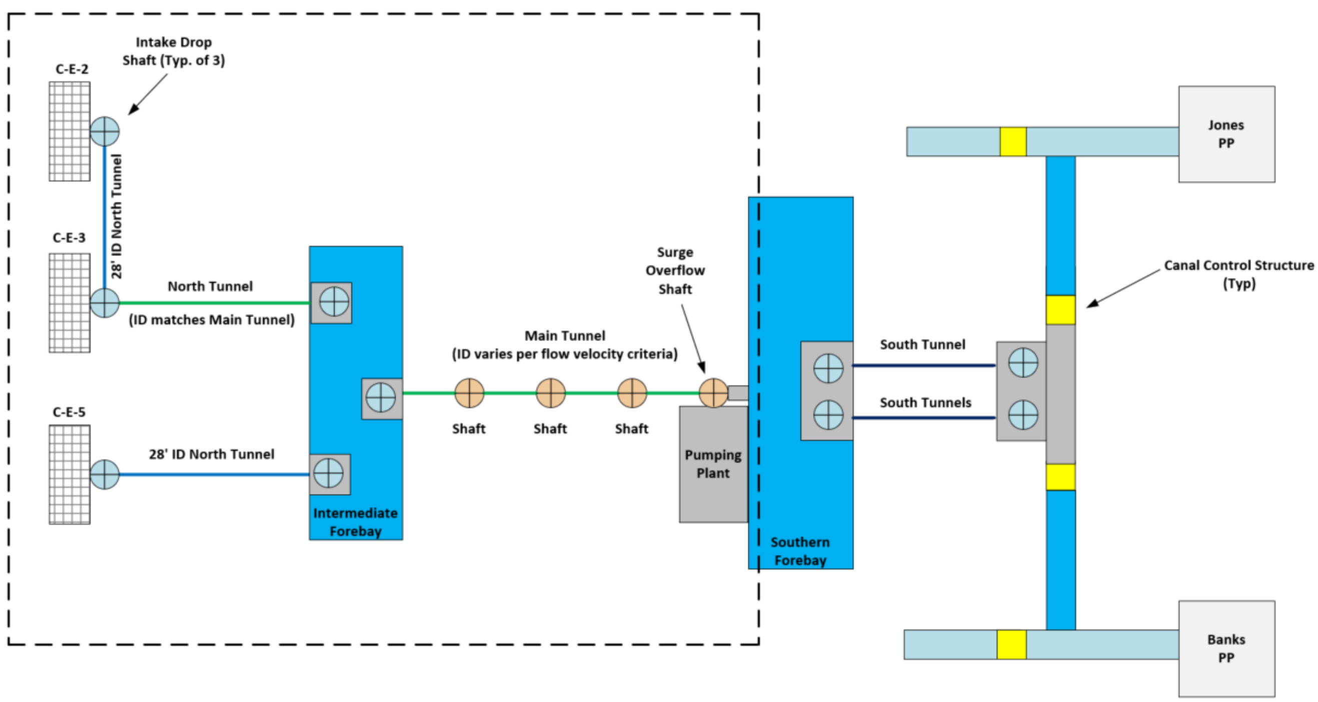

Figure 1. Delta Conveyance System Project Schematic

The following Project configuration was considered for this evaluation:

- Main tunnel alignment
- Three intake locations on the left bank of the Sacramento River, as follows:
  - Intake C-E-2, located downstream of Scribner's Bend between Clarksburg and Hood at approximate River Mile (RM) 41.1
  - Intake C-E-3, located upstream of Hood at approximate RM 39.4
  - Intake C-E-5, located immediately upstream of the confluence with Snodgrass Slough at approximate RM 36.8

For purposes of this early tunnel sizing exercise, an IF was included in the analysis. While subsequent hydraulic evaluations resulted in elmination of the IF, this design change had no impact on the results of this tunnel sizing analysis. The IF is located north of Twin Cities Road and east of Snodgrass Slough and includes:

- Tunnels connecting the intakes and IF per Figure 1
- A single tunnel connecting the IF and the pumping plant wet well
- The pumping plant, which conveys water to the Sourthern Forebay located on the northwestern side of the existing Clifton Court Forebay

1.2 Summary of Results

Based on the results of the hydraulic and capacity analysis, and construction cost comparison as described in this TM, it is recommended that the maximum tunnel flow velocity be limited to 6 feet per second (fps) for the maximum design flow capacities of 4,500, 6,000, and 7,500 cfs. This velocity criteria results in the recommended tunnel diameters of:

- 31-foot diameter at the maximum design flow capacity of 4,500 cfs
- 36-foot diameter at the maximum design flow capacity of 6,000 cfs
- 40-foot diameter at the maximum design flow capacity of 7,500 cfs

These tunnel diameters are recommended for further development as part of the Project's conceptual design within the maximum design flow range of 4,500 to 7,500 cfs.

2. Methodology

Tunnel diameter options were evaluated using the following process:

1) Selected the range of candidate tunnel diameters to be evaluated for the Project that achieve the assigned flow velocity criteria range.
2) Conducted system end-to-end hydraulic head loss analysis using candidate tunnel diameters, and reviewed results against corresponding system operating water surface elevations (WSELs) within the tunnel, IF, and pumping plant wet well, and hydraulic horsepower (hp) required by the pumping plant. Compared these results with the characteristics of the baseline 44-foot tunnel diameter.
3) Conducted hydraulic transient-surge analyses using candidate tunnel diameters, and developed the envelope of maximum and minimum hydraulic grade lines (HGLs) for each transient-surge condition. Conducted analysis with three IF footprints of 800 by 1,500 feet (width by length), 800 by 1,000 feet, and 500 by 500 feet, and with no IF included in the Project.
4) Conducted a construction cost comparison of each tunnel diameter option considered against the 44-foot baseline tunnel diameter.
5) Conducted an annual power cost comparison for each tunnel diameter head loss compared against the head loss of the 44-foot baseline tunnel diameter. The present value cost associated with the annual power consumption was included with the construction cost comparison for each tunnel diameter evaluated.
6) Evaluated the candidate tunnel diameter options according to relative suitability of flow velocity, transient-surge response, impacts to operating depths of Project elements (IF and pumping plant wet well invert elevation requirements), and construction and operating costs.
7) Verify recommended velocities are suitable for tunnel lining system.

2.1 Data and Information Sources

Average monthly flows were taken from the DSM2 model results presented in Appendix 5A, Section C, of the *Final Bay Delta Conservation Plan/California WaterFix Environmental Impact Report/Environmental Impact Statement* (Final EIR/EIS) (DWR and Reclamation 2016).

2.2 Tunnel Sizing and Evaluation Criteria

Criteria were developed from experience with sizing tunnels and water transmission systems to guide the identification and evaluation of candidate tunnel sizes. Criteria included:

- Project Design Flow Range:
  - Hydraulic evaluation was conducted over the range of flows between 0 cfs and the maximum design flow capacities of 4,500, 6,000, and 7,500 cfs.
- Tunnel Flow Velocity Range:
  - At full capacity, the minimum tunnel flow velocity was limited to 3.5 fps. The minimum tunnel flow velocity was considered to maintain a minimum scour velocity for cleaning sediment within the tunnel.
  - At full capacity, the maximum tunnel flow velocity was limited to 8.0 fps. The maximum flow velocity is consistent with the upper flow velocity range for typical water transmission mains and usually provides a balance between pipe size, power cost, and acceptable transient-surge conditions. This upper limit flow velocity is considered suitable for tunnels with segmental concrete lining systems.
- Candidate Tunnel Diameter Sizing:
  - The minimum candidate tunnel diameters for each maximum design flow capacity (4,500, 6,000, and 7,500 cfs) were calculated based on the maximum flow velocity of 8 fps.
  - The maximum candidate tunnel diameter for each maximum design flow capacity (4,500, 6,000, and 7,500 cfs) were calculated based on the minimum flow velocity of 3.5 fps.
- Standard Roughness Coefficients:
  - The maximum and minimum interior equivalent roughness coefficients (Manning’s N) of 0.016 and 0.014, respectively, have been assigned for the interior roughness of the segmentally lined tunnels and provide a sufficiently conservative analysis range suitable for the conceptual-level tunnel head loss analysis.
  - Manning’s N of 0.016 was used as the standard roughness coefficient for the hydraulic head loss analysis to evaluate a more conservative case of resultant tunnel head loss due to the highest tunnel interior friction condition.
  - Manning’s N of 0.014 was used as the standard roughness coefficient for the hydraulic transient-surge analysis to evaluate a more conservative case of resultant maximum and minimum surge pressures within the tunnel hydraulic grade line (HGL) due to lower tunnel interior friction condition.
- Steady-State Hydraulic Head Loss Analysis:
  - The source of diversion flows from the Sacramento River was simulated from intakes C-E-2 and C-E-3 for design flow capacities up to 6,000 cfs.
  - The source of diversion flows from the Sacramento River was simulated from intakes C-E-2, C-E-3, and C-E-5 for design flow capacities between 6,000 to 7,500 cfs.
- Transient-Surge Analysis:
  - Hydraulic transient-surge analysis was conducted between the C-E-2 drop shaft to the pumping plant wet well overflow facility at the design flow capacities of 6,000 and 7,500 cfs.

   - The upstream flow conditions were controlled by simulating closure of radial gates at the entrance to the drop shaft at each intake structure. Downstream flow conditions were simulated by fixed weir openings within the pumping plant's overflow shaft.
   - Simultaneous shutdown of the main raw water pumps in the pumping plant followed by the closure of sediment basin outlet gates at each intake in operation was simulated for each transient-surge analysis. Outlet gates remained open in their last setpoint position for 5 minutes following pump shutdown and were then simultaneously closed within 1 minute. Closure of the sediment basin outlet gates was simulated to prevent reverse flow into the Sacramento River from the intakes during the transient-surge event.
   - Transient-surge analysis was conducted with IF footprints of: 800 by 1,500 feet (width by length); 800 by 1,000 feet; 500 by 500 feet; and with no IF included in the Project.
   - The source of diversion flows from the Sacramento River for the steady-state head loss analysis was simulated from intakes C-E-2 and C-E-3 for the design flow capacity of 6,000 cfs; and from C-E-2, C-E-3, and C-E-5 for the design flow capacity of 7,500 cfs.
   - Hydraulic transient-surge analysis was conducted with both the minimum and maximum Sacramento River elevations at C-E-2, C-E-3, and C-E-5 at design flow capacities of both 6,000 and 7,500 cfs.
- Cost Analysis:
   - Comparative construction costs were derived from six separate estimates prepared for a single tunnel system, each at different diameters and lengths, which include launch and retrieval shafts and associated tunnel structures. Shaft diameters were adjusted to reflect the change in tunnel diameter. Direct cost estimate values were based on 2019 pricing and factored by a 1.76 multiplier to account for programmatic costs, such as risk, contingency, and other soft costs. Using separate estimates, a cost equation was established between the tunnel diameter, tunnel length, and cost to develop comparative costs for the tunnel diameters considered in this analysis.
   - Power cost comparisons for calculated head loss were conducted for each tunnel diameter for two Project operating conditions to establish the upper and lower range of calculated costs. The operating design flow conditions evaluated were:
      - (1) Each design rated capacity of 4,500, 6,000, and 7,500 cfs assumed for 24 hours per day and 365 days per year for the highest cost
      - (2) Average monthly flows from the DSM2 model runs presented in Appendix 5A, Section C, of the 2016 Final EIR/EIS (DWR and Reclamation 2016) for the lower cost.
   - The net present value (NPV) of the highest and lowest power costs were computed over a 100-year period with a discount rate of 3.0 percent at an electrical power cost of $0.07 per kilowatt-hour (kWh). A combined pump and motor efficiency of 80 percent was used.

2.3 Assumptions and Boundary Conditions

Basic assumptions and boundary conditions that apply to identifying and evaluating the Project include the following:

- Alignment of tunnels and locations of the intakes, shafts, IF, pumping plant, and Southern Forebay are as described in the California WaterFix Draft Supplemental EIR/EIS (DWR and Reclamation 2018)
- Tunnel ID between the C-E-2 drop shaft and the C-E-3 drop of 28 feet
- Tunnel ID between the C-E-5 drop shaft and the IF of 28 feet

- These design flow sequences of operation at intakes develop the maximum system head loss between the C-E-2 drop shaft and the pumping plant wet well over the full design flow range evaluated:
  – 0 to 2,250 cfs; C-E-2 only
  – 2,250 to 4,500 cfs; C-E-2 maintains the diversion capacity of 2,250 cfs, and C-E-3 begins diverting flows up to 2,250 cfs
  – 4,500 to 6,000 cfs; diversion capacities of C-E-2 and C-E-3 are increased at equal capacities up to the maximum diversion rate of 3,000 cfs each
  – 6,000 to 7,500 cfs; diversion capacities of C-E-2 and C-E-3 are maintained at 3,000 cfs each, then C-E-5 begins diverting flows up to 1,500 cfs
- Sacramento River elevations (North American Vertical Datum of 1988 [NAVD88]) assumed for the hydraulic head loss and transient-surge analysis:
  – Minimum of 1.9 and maximum of 31.4 feet at C-E-2
  – Minimum of 1.6 and maximum of 30.4 feet at C-E-3
  – Minimum of 0.7 and maximum of 28.4 feet at C-E-5
- Overflow weir crest at the pumping plant wet well inlet shaft was set at 19 feet
- Moment of inertia value for the pumping plant's large pumps was calculated using Bentley Systems' (Bentley's) HAMMER software as 825,500 pounds per square foot (lb-ft^2); this value was calculated based on the candidate pump's break horsepower (BHP) at the rated design point condition
- Moment of inertia value for the pumping plant's smaller pumps was calculated using HAMMER as 187,000 lb-ft^2; this value was calculated based on the candidate pump's BHP at the rated design point condition
- Wave speed for the main tunnel between C-E-3 and the pumping plant wet well was calculated as 1,589 fps using the Wave Speed Calculator function in HAMMER
- Wave speed for the 28-foot-diameter tunnel between C-E-2 and C-E-3, and C-E-5 and the IF was calculated as 1,589 fps using the Wave Speed Calculator function in HAMMER

2.4 Tools

2.4.1 Conveyance System Hydraulic Model

A hydraulic model was constructed for the Project between the intakes and the pumping plant wet well using Innovyze's InfoWorks Integrated Catchment Modeling (ICM) software. The model configuration consisted of the following components:

- Sacramento River intakes C-E-2, C-E-3, and C-E-5, including inlet structures, screens, control gates, sediment basins, and tunnel drop shafts
- IF, including separate tunnel inlet and tunnel outlet structures
- For IF removed, the forebay was removed and the two shafts were connected by a tunnel segment
- Pumping plant wet well and gravity flow and surge overflow shaft
- Tunnels connecting the intakes to the IF and the IF to the pumping plant wet well and gravity flow and surge overflow shaft

2.4.2 Transient-Surge Analysis

Bentley's HAMMER software was used to perform the transient-surge analysis. In addition to the steady-state pipe and hydraulic parameters, the HAMMER program uses the method of characteristics described by Wylie and Streeter (1993) to solve the pressure transients in the system. This method consists of deriving basic equations from physical principles (the continuity equation and conservation of energy and momentum). The equations are then solved along characteristic lines whose slope is dependent upon the acoustic wave speed.

3. Analysis and Evaluation

3.1 Steady-State Hydraulic Head Loss Analysis

In accordance with the methodology and criteria described, a steady-state, hydraulic head loss analysis was performed for a range of tunnel diameters between the C-E-3 drop shaft and the IF, and from the IF to the pumping plant wet well that resulted in flow velocities between 3.5 to 8.0 fps for the maximum design flows of 4,500, 6,000, and 7,500 cfs.

Table 1 summarizes candidate tunnel diameters meeting the flow velocity criteria. For each design flow capacity of 4,500, 6,000, and 7,500 cfs, tunnel diameters are shown in 1-foot increments within the tunnel flow velocity range of 3.5 to 8.0 fps.

A steady-state hydraulic analysis was conducted for each tunnel diameter shown in Table 1 for separate flow ranges between 750 to 4,500, 6,000, and 7,500 cfs. To determine the minimum resultant WSELs in the IF and the pumping plant wet well, tunnel head losses were separately calculated between the intakes to the IF, and from the IF to the Pumping plant wet well for each design flow range for each candidate tunnel diameter.

Table 1. Summary of Candidate Tunnel Diameters Relative to Design Flow and Flow Velocity Criteria

Design Flow Capacity = 4,500 cfs		Design Flow Capacity = 6,000 cfs		Design Flow Capacity = 7,500 cfs	
Tunnel Dia (feet)	Tunnel Flow Velocity (fps)	Tunnel Dia (feet)	Tunnel Flow Velocity (fps)	Tunnel Dia (feet)	Tunnel Flow Velocity (fps)
40	3.6	47	3.5	52	3.5
39	3.8	46	3.6	51	3.7
38	4.0	45	3.8	50	3.8
37	4.2	44	3.9	49	4.0
36	4.4	43	4.1	48	4.1
35	4.7	42	4.3	47	4.3
34	5.0	41	4.5	46	4.5
33	5.3	40	4.8	45	4.7
32	5.6	39	5.0	44	4.9
31	6.0	38	5.3	43	5.2

Table 1. Summary of Candidate Tunnel Diameters Relative to Design Flow and Flow Velocity Criteria

Design Flow Capacity = 4,500 cfs		Design Flow Capacity = 6,000 cfs		Design Flow Capacity = 7,500 cfs	
Tunnel Dia (feet)	Tunnel Flow Velocity (fps)	Tunnel Dia (feet)	Tunnel Flow Velocity (fps)	Tunnel Dia (feet)	Tunnel Flow Velocity (fps)
30	6.4	37	5.6	42	5.4
29	6.8	36	6.0	41	5.7
28	7.3	35	6.2	40	6.0
27	7.9	34	6.6	39	6.3
-	-	33	7.0	38	6.6
-	-	32	7.5	37	7.0
-	-	31	7.9	36	7.4
-	-	-	-	35	8.0

Notes:

- = not applicable

Dia = diameter

The steady-state hydraulic analysis incorporated the highest friction factor (Manning's N of 0.016) and the lowest Sacramento River elevations of 1.9, 1.6, and 0.70 feet at C-E-2, C-E-3, and C-E-5, respectively, for each candidate tunnel diameter to establish the lowest HGL (maximum head loss) throughout the Project (intakes to pumping plant wet well) and the resulting minimum operating WSELs at each hydraulic facility (IF and pumping plant wet well). This analysis included head losses through the fish screens at each intake. Fish screens were assumed to be in a clean condition.

Figures 2 through 4 plot the tunnel head loss results that were developed between C-E-2 and the IF over the full envelope of design flow conditions. On Figures 2 through 4, the three maximum design flow capacities of 4,500, 6,000, and 7,500 cfs are shown in each graph as vertical dashed lines (blue, black, and brown). The head loss curves are plotted from a minimum flow of 750 cfs up to each design flow maximum capacity. Tunnel diameter plots are generally shown in 2-foot increments on each figure. Head loss corresponding with tunnel diameters not shown can be obtained through visual inspection for any flow condition.

Head loss curves shown on Figures 2 through 4 have an inflection point at 2,250 cfs. This is due to flows entering the tunnel from C-E-2 only for all tunnel flows up to 2,250 cfs. For all tunnel flows greater than 2,250 cfs but less than 6000 cfs, flows from C-E-2 are combined with flows from C-E-3. As such, the maximum tunnel flow between the C-E-2 and C-E-3 drop shafts is limited to 3,000 cfs (maximum diversion flow from C-E-2), and the maximum tunnel flow between the C-E-3 drop shaft and the IF is limited to 6,000 cfs (maximum combined diversion flows from C-E-2 and C-E-3). Flows entering the IF from C-E-5 are conveyed through a separate, parallel tunnel. Tables 2 through 4 summarize the head loss and corresponding IF WSEL for each tunnel diameter (at 1-foot increments) at each maximum design flow capacity of 4,500, 6,000, and 7,500 cfs.

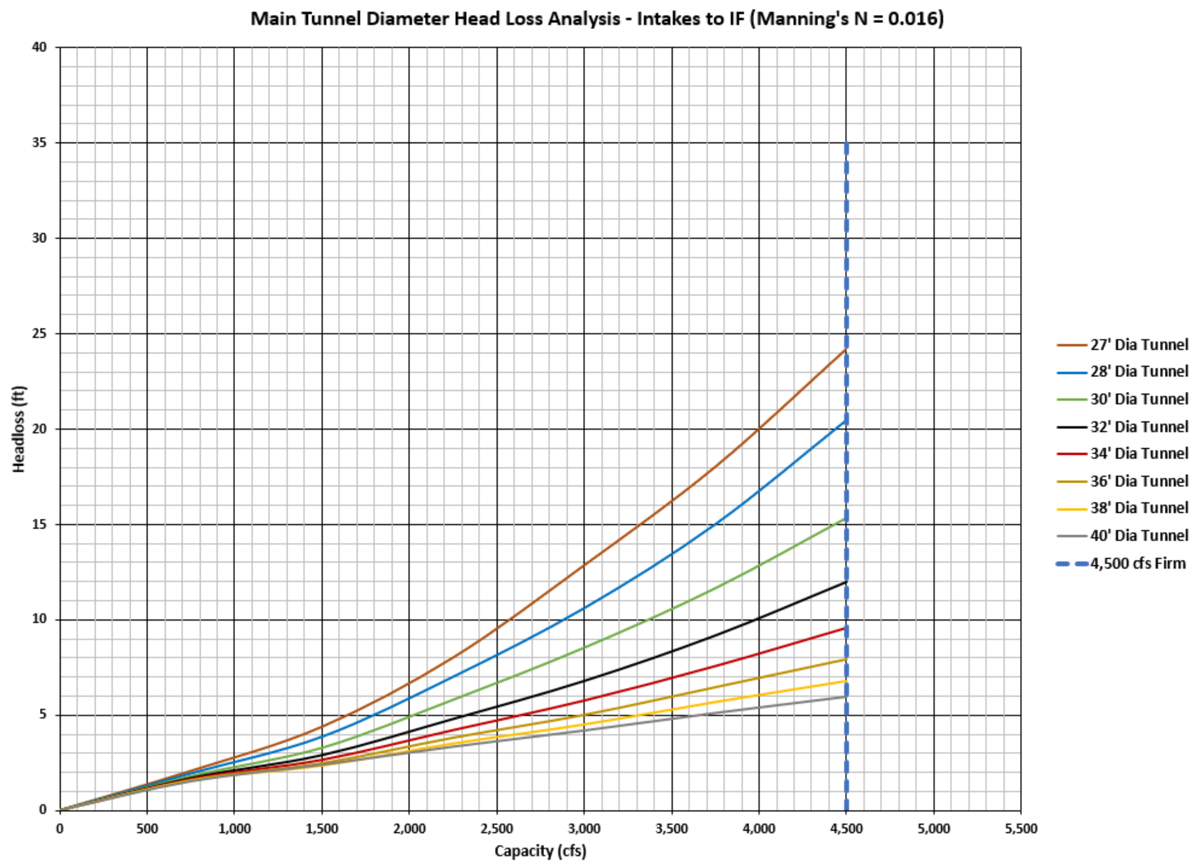

Figure 2. Flow Capacity versus Head Loss for Candidate Tunnel Diameters up to Maximum Design Flow 4,500 cfs

Table 2. Tunnel Head Loss and Intermediate Forebay Water Surface Elevation, Design Flow Capacity 4,500 cfs

Tunnel Diameter (feet)	Flow Velocity (fps)	Head Loss from Intakes to IF (feet)	IF WSEL (feet)
40	3.6	6.0	-4.1
39	3.8	6.3	-4.4
38	4.0	6.8	-4.9
37	4.2	7.3	-5.4
36	4.4	7.9	-6.0
35	4.7	8.7	-6.8
34	5.0	9.6	-7.7
33	5.3	10.9	-9.0
32	5.6	12.0	-10.1
31	6.0	13.7	-11.8
30	6.4	15.3	-13.4
29	6.8	17.9	-16.0
28	7.3	20.5	-18.6
27	7.9	24.2	-22.3

As can be seen on Figure 2 and in Table 2, at the maximum design flow capacity of 4,500 cfs, the minimum tunnel diameter of 27 feet (7.9-fps flow velocity at 4,500 cfs) develops a steady-state head loss of 24.2 feet and results in a WSEL in the IF of -22.3 feet (steady-state head loss of 24 feet subtracted from the Sacramento River WSEL of 1.9 feet). At the same design flow capacity, the maximum tunnel diameter of 40 feet (3.6-fps flow velocity at 4,500 cfs) develops a steady-state head loss of 6 feet and results in a WSEL in the IF of -4.1 feet. The head loss comparison between the minimum and maximum tunnel diameters (27 and 40 feet) at the design flow capacity of 4,500 cfs is 24 versus 6 feet.

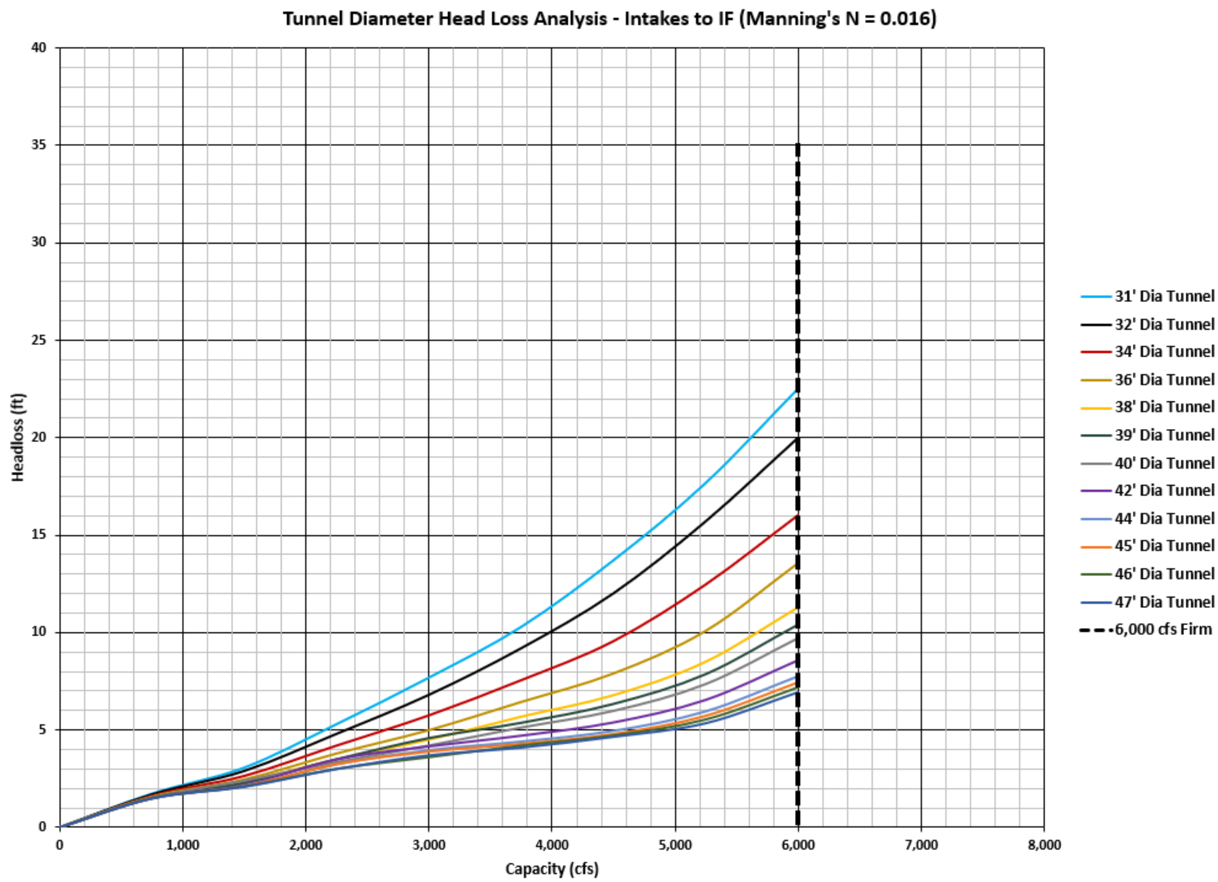

Figure 3. Flow Capacity versus Head Loss for Candidate Tunnel Diameters up to Maximum Design Flow 6,000 cfs

Table 3. Tunnel Head Loss and Intermediate Forebay Water Surface Elevation, Design Flow Capacity 6,000 cfs

Tunnel Diameter (feet)	Flow Velocity (fps)	Head Loss from Intakes to IF (feet)	IF WSEL (feet)
47	3.5	6.9	-5.0
46	3.6	7.2	-5.3
45	3.8	7.4	-5.5
44	3.9	7.8	-5.9
43	4.1	8.2	-6.3
42	4.3	8.6	-6.7

Table 3. Tunnel Head Loss and Intermediate Forebay Water Surface Elevation, Design Flow Capacity 6,000 cfs

Tunnel Diameter (feet)	Flow Velocity (fps)	Head Loss from Intakes to IF (feet)	IF WSEL (feet)
41	4.5	9.1	-7.2
40	4.8	9.7	-7.8
39	5.0	10.4	-8.6
38	5.3	11.3	-9.4
37	5.6	12.3	-10.4
36	6.0	13.6	-11.7
35	6.2	15.2	-13.3
34	6.6	16.0	-14.1
33	7.0	18.4	-16.5
32	7.5	20.0	-18.1
31	7.9	22.5	-20.6

Referring to Figure 3 and Table 3, at the maximum design flow capacity of 6,000 cfs, the minimum tunnel diameter of 31 feet (7.9-fps flow velocity at 6,000 cfs) develops a steady-state head loss of 22.5 feet and results in a WSEL in the IF of -20.6 feet. At the same design flow capacity, the maximum tunnel diameter of 47 feet (3.5-fps flow velocity at 6,000 cfs) develops a steady-state head loss of 6.9 feet and results in a WSEL in the IF of -5.0 feet. The head loss comparison between the minimum and maximum tunnel diameters (31 and 47 feet) at the design flow capacity of 6,000 cfs is 22.5 versus 6.9 feet.

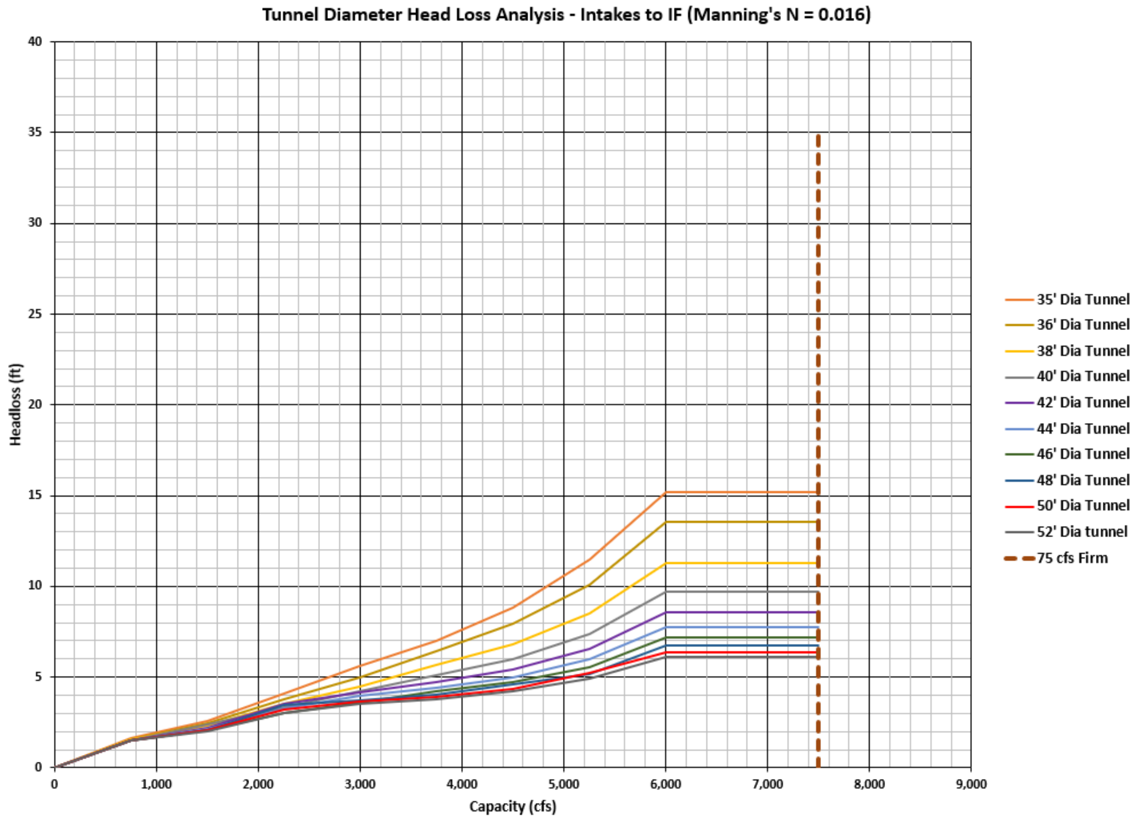

Figure 4. Flow Capacity versus Head Loss for Candidate Tunnel Diameters up to Maximum Design Flow 7,500 cfs

Table 4. Tunnel Head Loss and Intermediate Forebay Water Surface Elevation, Design Flow Capacity 7,500 cfs

Tunnel Diameter (feet)	Flow Velocity at 7,500 cfs (fps)	Flow Velocity at 6,000 cfs (fps)	Head Loss from Intakes to IF (feet)	IF WSEL (feet)
52	3.5	2.8	6.1	-4.2
51	3.7	2.9	6.3	-4.4
50	3.8	3.1	6.4	-4.5
49	4.0	3.2	6.6	-4.7
48	4.1	3.3	6.7	-4.8
47	4.3	3.5	6.9	-5.0
46	4.5	3.6	7.2	-5.3
45	4.7	3.8	7.4	-5.5
44	4.9	3.9	7.7	-5.8
43	5.2	4.1	8.2	-6.3
42	5.4	4.3	8.6	-6.7

Table 4. Tunnel Head Loss and Intermediate Forebay Water Surface Elevation, Design Flow Capacity 7,500 cfs

Tunnel Diameter (feet)	Flow Velocity at 7,500 cfs (fps)	Flow Velocity at 6,000 cfs (fps)	Head Loss from Intakes to IF (feet)	IF WSEL (feet)
41	5.7	4.5	9.1	-7.2
40	6.0	4.8	9.7	-7.8
39	6.3	5.0	10.4	-8.5
38	6.6	5.3	11.3	-9.4
37	7.0	5.6	12.3	-10.4
36	7.4	5.9	13.6	-11.7
35	8.0	6.2	15.2	-13.3

Figure 4 shows the head loss developed in the tunnel up to the maximum design flow capacity of 7,500 cfs. The figure shows there is no increase in tunnel head loss between 6,000 and 7,500 cfs. This is because the tunnel connecting E-C-5 to the IF is a separate tunnel that runs parallel with the tunnel connecting E-C-2 and E-C-3, and the diversion flows from E-C-5 are added to the tunnel after E-C-2 and E-C-3 are combined at the IF. The tunnel connecting the C-E-3 drop shaft to the IF was sized identical to the main tunnel (that is, the tunnel connecting the IF to the pumping plant wet well) to reduce the number of tunnel boring machines (TBMs) required for the Project. Therefore, the tunnel section between the C-E-3 drop shaft and the IF is based on the minimum and maximum tunnel flow velocity of 3.5 to 8.0 fps, respectively, at the maximum design flow capacity of 7,500 cfs; however, the head loss was calculated up to a maximum of 6,000 cfs, which is the maximum possible flow based on the diversion capacities at C-E-2 and C-E-3 of 3,000 cfs each. Flow velocities corresponding with each tunnel diameter are shown in Table 4 for both the maximum design flow capacities of 6,000 and 7,500 cfs.

Referring to Figure 4 and Table 4, at the maximum design flow capacity of 6,000 cfs, the minimum tunnel diameter of 35 feet develops a steady-state head loss of 15.2 feet and results in a WSEL in the IF of -13.3 feet. At the same design flow capacity, the maximum tunnel diameter of 52 feet develops a steady-state head loss of 6.1 feet and results in a WSEL in the IF of -4.2 feet. The head loss comparison between the minimum and maximum tunnel diameters (35 and 52 feet) at the design flow capacity of 6,000 cfs is 15.2 versus 6.1 feet.

Figures 5, 6, and 7 plot the tunnel head loss results developed between the IF and the pumping plant wet well over the full envelope of design flow conditions. The steady-state WSELs associated with the IF were established by the steady-state head loss analysis between the intakes and IF at the same design flow conditions and candidate tunnel diameter. The three maximum design flow capacities of 4,500, 6,000, and 7,500 cfs are shown in each graph as vertical dashed lines (blue, black, and brown). For clarity, not all tunnel diameters summarized in Table 1 are shown on Figures 5, 6, and 7. Head loss associated with tunnel diameters not shown can be obtained through visual inspection. A legend is provided on each figure that defines the color code for each tunnel diameter head loss curve shown.

Tables 5, 6, and 7 summarize the head loss and corresponding pumping plant wet well WSEL for each tunnel diameter (at 1-foot increments) at each maximum design flow capacity of 4,500, 6000, and 7,500 cfs.

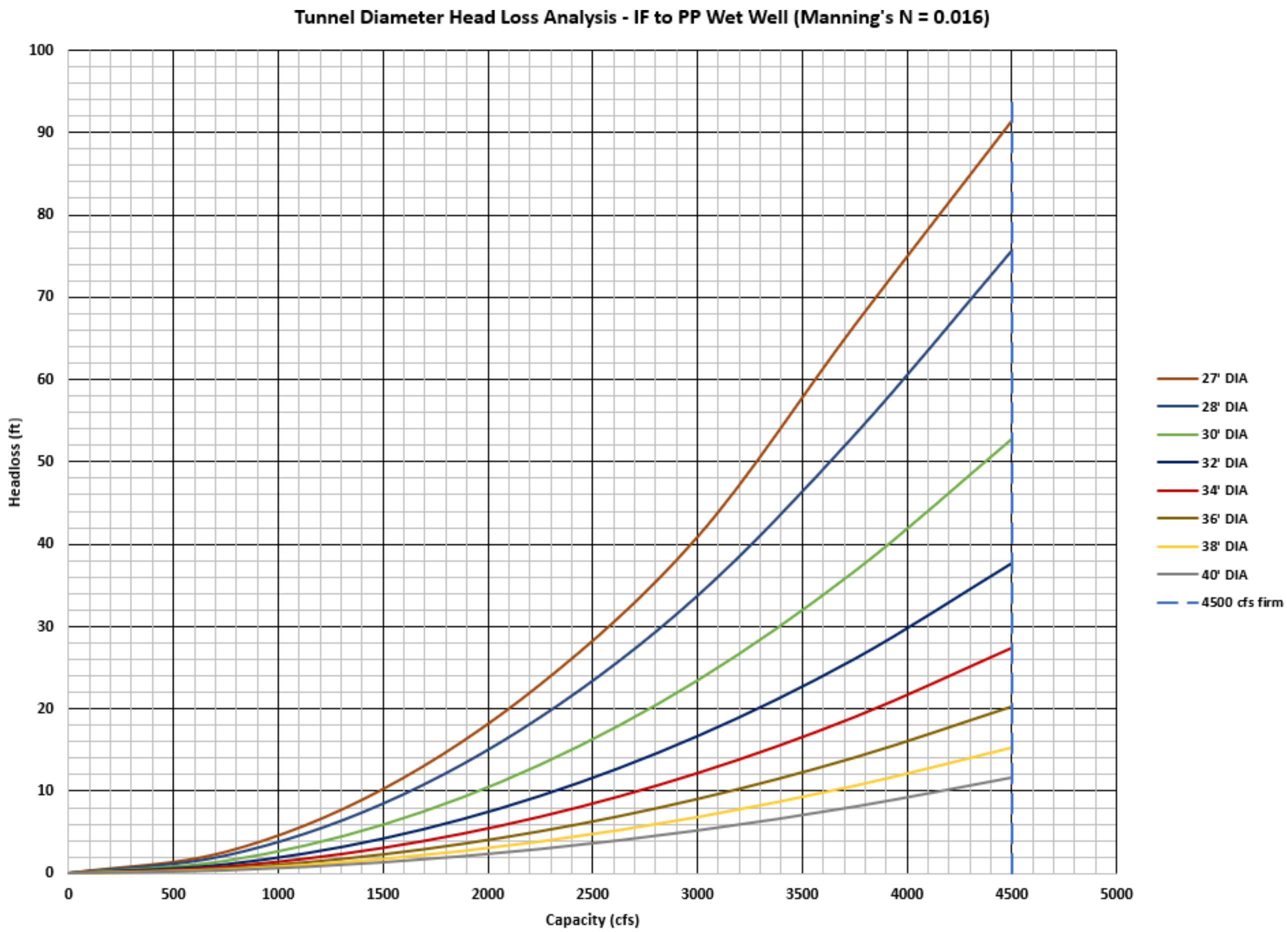

Figure 5. Flow Capacity versus Head Loss for Candidate Tunnel Diameters up to Maximum Design Flow 4,500 cfs

Table 5. Tunnel Head Loss and Pumping Plant Wet Well Water Surface Elevation at Design Flow Capacity 4,500 cfs

Tunnel Diameter (feet)	Flow Velocity (fps)	Head Loss from IF to Pumping Plant Wet Well (feet)	Pumping Plant Wet Well WSEL (feet)
40	3.6	11.7	-15.8
39	3.8	13.3	-17.7
38	4.0	15.3	-20.2
37	4.2	17.5	-22.9
36	4.4	20.2	-26.2
35	4.7	23.5	-30.3

Table 5. Tunnel Head Loss and Pumping Plant Wet Well Water Surface Elevation at Design Flow Capacity 4,500 cfs

Tunnel Diameter (feet)	Flow Velocity (fps)	Head Loss from IF to Pumping Plant Wet Well (feet)	Pumping Plant Wet Well WSEL (feet)
34	5.0	27.3	-35.0
33	5.3	44.2	-53.2
32	5.6	37.7	-47.8
31	6.0	44.8	-56.6
30	6.4	52.7	-66.1
29	6.8	63.4	-79.4
28	7.3	75.7	-94.3
27	7.9	91.5	-113.8

Referring to Figure 5 and Table 5, at the maximum design flow capacity of 4,500 cfs, the minimum tunnel diameter of 27 feet (7.9-fps flow velocity at 4,500 cfs) develops a steady-state head loss of 91.5 feet and results in a WSEL in the pumping plant wet well of -113.8 feet. At the same design flow capacity, the maximum tunnel diameter of 40 feet (3.6-fps flow velocity at 4,500 cfs) develops a steady-state head loss of 11.7 feet and results in a WSEL in the pumping plant Wet Well of -15.8 feet. The head loss comparison between the minimum and maximum tunnel diameters (27 and 40 feet) at the design flow capacity of 4,500 cfs is 91.5 versus 11.7 feet.

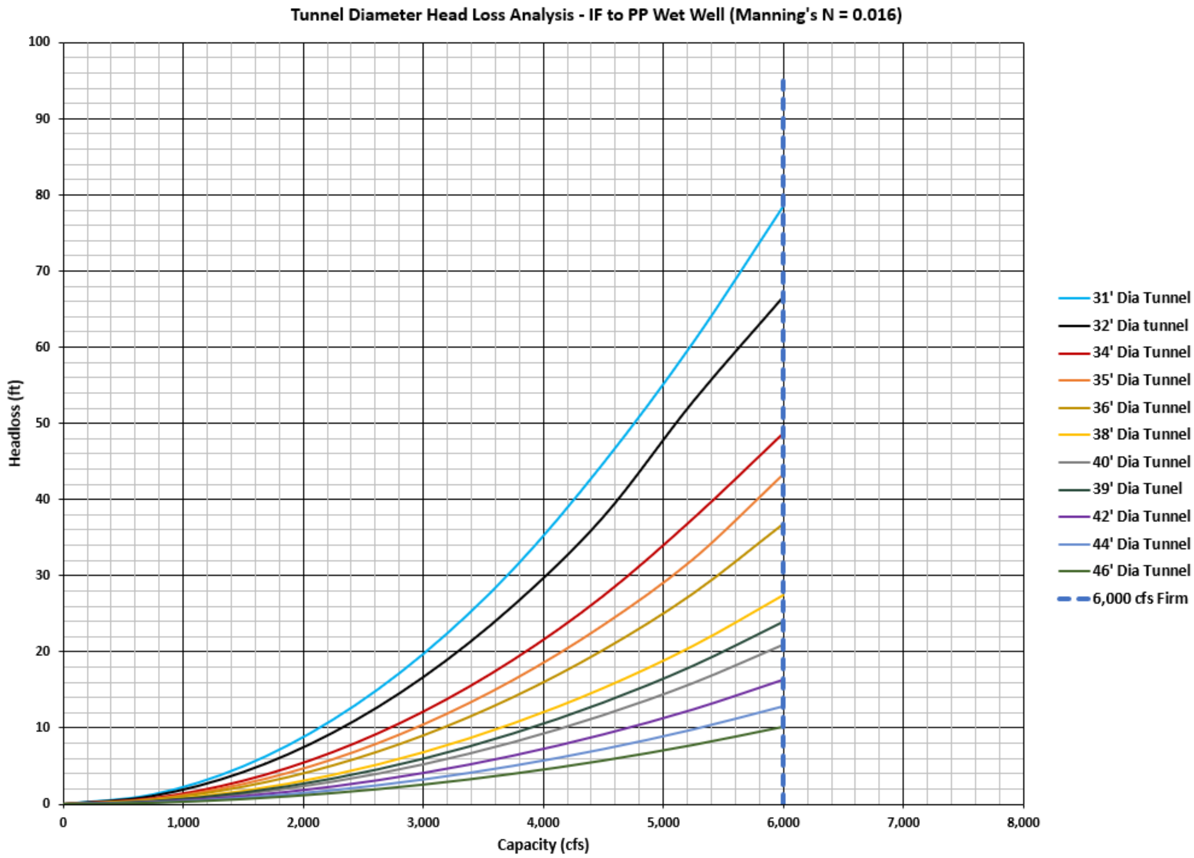

Figure 6. Flow Capacity versus Head Loss for Candidate Tunnel Diameters up to Maximum Design Flow 6,000 cfs

Table 6. Tunnel Head Loss and Pumping Plant Wet Well Water Surface Elevation at Design Flow Capacity 6,000 cfs

Tunnel Diameter (feet)	Flow Velocity (fps)	Head Loss from IF to Pumping Plant Wet Well (feet)	Pumping Plant Wet Well WSEL (feet)
47	3.5	9.2	-14.2
46	3.6	10.2	-15.5
45	3.8	11.4	-16.9
44	3.9	12.8	-18.7
43	4.1	14.4	-20.7
42	4.3	16.3	-23.0
41	4.5	18.5	-25.7
40	4.8	21.0	-28.8
39	5.0	24.0	-32.6
38	5.3	27.5	-36.9
37	5.6	31.7	-42.1
36	6.0	36.8	-48.5
35	6.2	43.3	-56.6

Table 6. Tunnel Head Loss and Pumping Plant Wet Well Water Surface Elevation at Design Flow Capacity 6,000 cfs

Tunnel Diameter (feet)	Flow Velocity (fps)	Head Loss from IF to Pumping Plant Wet Well (feet)	Pumping Plant Wet Well WSEL (feet)
34	6.6	48.7	-62.8
33	7.0	57.8	-74.3
32	7.5	66.6	-84.7
31	7.9	78.5	-99.1

Referring to Figure 6 and Table 6, at the maximum design flow capacity of 6,000 cfs, the minimum tunnel diameter of 31 feet (7.9-fps flow velocity at 6,000 cfs) develops a steady-state head loss of 78.5 feet and results in a WSEL in the pumping plant wet well of –99.1 feet. At the same design flow capacity, the maximum tunnel diameter of 47 feet (3.5-fps flow velocity at 6,000 cfs) develops a steady-state head loss of 9.2 feet and results in a WSEL in the pumping plant Wet Well of -14.2 feet. The head loss comparison between the minimum and maximum tunnel diameters (31 and 47 feet) at the design flow capacity of 6,000 cfs is 78.5 versus 9.2 feet.

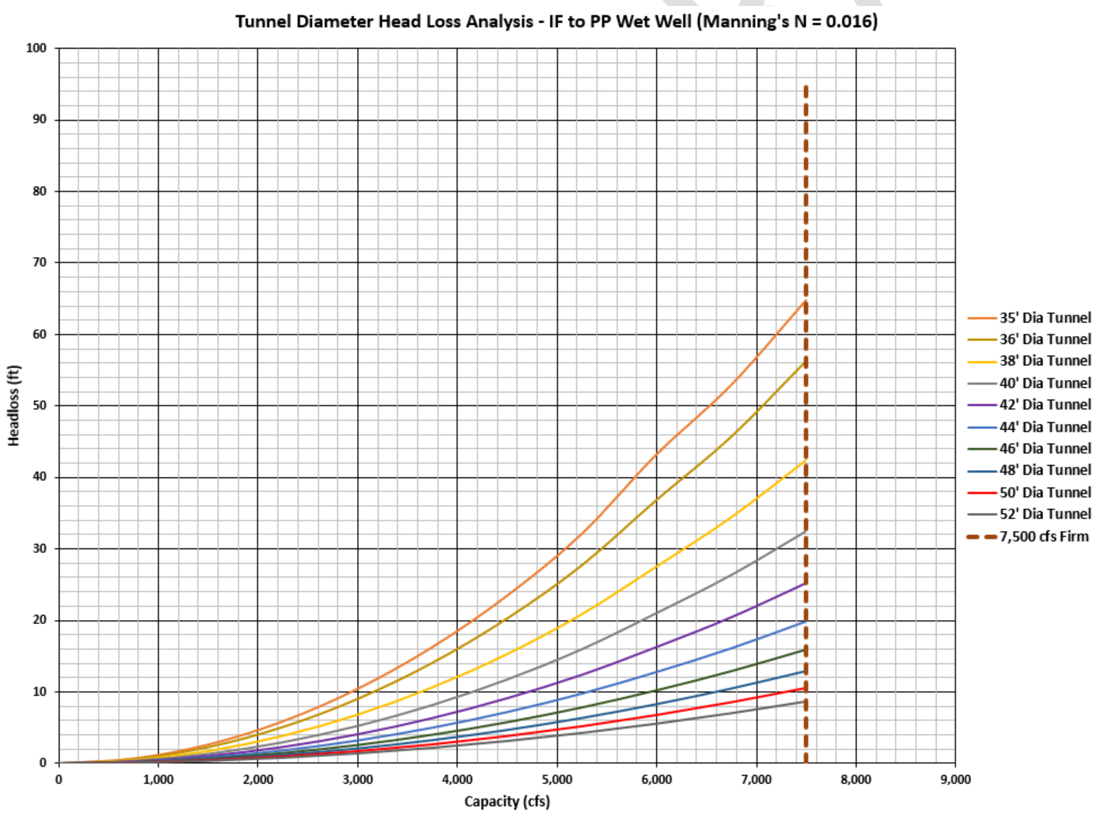

Figure 7. Flow Capacity versus Head Loss for Candidate Tunnel Diameters up to Maximum Design Flow 7,500 cfs

Table 7. Tunnel Head Loss and Pumping Plant Wet Well Water Surface Elevation at Design Flow Capacity 7,500 cfs

Tunnel Diameter (feet)	Flow Velocity (fps)	Head Loss from IF to Pumping Plant Wet Well (feet)	Pumping Plant Wet Well WSEL (feet)
52	3.5	8.7	-12.9
51	3.7	9.5	-13.9
50	3.8	10.5	-15.0
49	4.0	11.6	-16.3
48	4.1	12.8	-17.6
47	4.3	14.3	-19.3
46	4.5	15.9	-21.2
45	4.7	17.7	-23.2
44	4.9	19.9	-25.7
43	5.2	22.4	-28.7
42	5.4	25.2	-31.9
41	5.7	28.5	-35.7
40	6.0	32.4	-40.2
39	6.3	37.0	-45.5
38	6.6	42.3	-51.7
37	7.0	48.7	-59.1
36	7.4	56.2	-67.9
35	8.0	64.8	-78.1

Referring to Figure 7 and Table 7, at the maximum design flow capacity of 7,500 cfs, the minimum tunnel diameter of 35 feet (8.0-fps flow velocity at 7,500 cfs) develops a steady-state head loss of 64.8 feet and results in a WSEL in the pumping plant wet well of –78.1 feet. At the same design flow capacity, the maximum tunnel diameter of 52 feet (3.5-fps flow velocity at 7,500 cfs) develops a steady-state head loss of 8.7 feet and results in a WSEL in the pumping plant wet well of -12.9 feet. The head loss comparison between the minimum and maximum tunnel diameters (35 and 52 feet) at the design flow capacity of 6,000 cfs is 64.8 versus 8.7 feet.

3.2 Evaluate Tunnel Head Loss with Candidate Pump Selection

The results of the steady-state head loss analysis conducted for each tunnel diameter for the separate flow ranges between 750 to 4,500, 6,000, and 7,500 cfs were compared against the achievable system head operating envelopes for pumping equipment sized for the capacity and total dynamic head (TDH) conditions for the Project. The purpose of this evaluation was to identify head loss constraints associated with achievable pump performance for further screening feasible tunnel diameter options. This analysis was conducted with hydraulic performance characteristics associated with single-stage, pull-out-style vertical pumps.

General pump capacity sizing charts from Flygt-Xylem (a current candidate pump manufacturer considered for the conceptual design development of the pumping plant), shown on Figure 8, indicate a maximum achievable capacity for single-stage, vertical pull-out-style pumps at rated TDH conditions of up to 90 feet is 400,000 gallons per minute (gpm) (890 cfs) per pump (as shown on the left chart on Figure 8). For TDH conditions between 90 and 350 feet, the maximum achievable capacity is limited to 250,000 gpm (557 cfs) per pump and requires multiple impeller stages for TDH conditions between 150 to 350 feet to achieve a 557 cfs (as shown in the gray regions on the right chart on Figure 8).

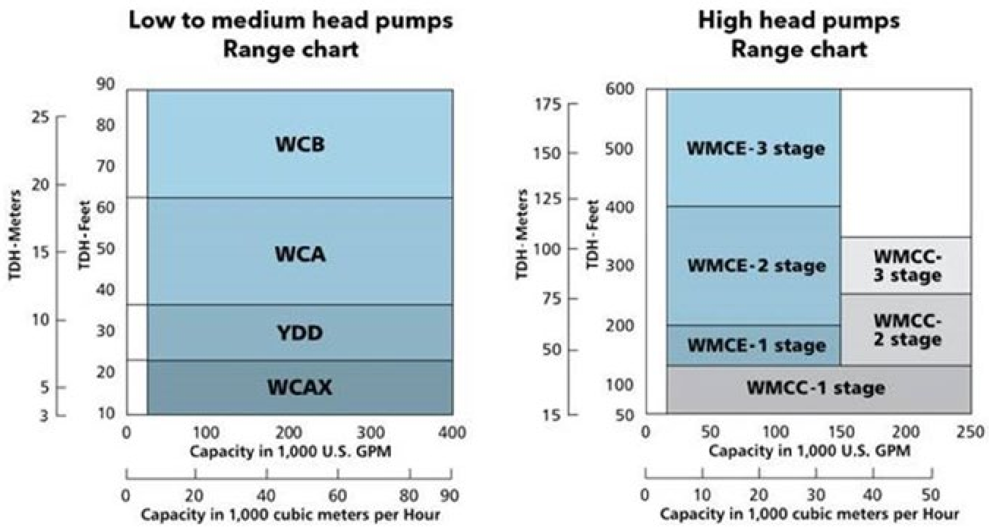

Figure 8. Low to High Head Range Chart for Flygt Large Vertical Column Pull-Out-Style Pumps

The high WSEL of the Project's Southern Forebay is currently set at 17.5 feet. As such, the minimum WSEL within the pumping plant wet well cannot be lower than -72.5 feet (17.5 feet minus 90 feet) to use the 890-cfs capacity pumps. For pumping plant wet well WSELs lower than -72.5 feet, the lower-capacity (557 cfs) pumps must be used. The lower-capacity pumps would require substantially larger pumping plant and wet well structures to achieve the required maximum design flow capacity as compared to using the higher-capacity pumps.

In addition to achieving the required maximum design flow capacity, the pumping plant must also be capable of operating at lower flow capacities at lower TDH conditions. The current design flow capacity range for the pumping plant is from 10 percent of the maximum design flow capacity to the maximum established design flow capacity and includes steady-state operation for all flows in between (that is, no flow gaps within the design flow range). To use a pump capable of achieving the TDH conditions at the required maximum and minimum TDH conditions, variable speed drives would be required.

For the purpose of this preliminary evaluation, the variable speed turndown of the candidate pumps would be limited to 50%. For example, for pumps with a maximum rated speed of 200 revolutions per minute (rpm), the minimum pump speed would be limited to 100 rpm (50 percent of 200 rpm). Using the pump industry standard Affinity Laws, the lowest TDH conditions for a pump with a maximum rated speed of 200 rpm that is operated at 100 rpm would be 0.25 times the head conditions at maximum speed. So that steady-state flows can be achieved at lower flow and head conditions associated with the Project (high Sacramento River elevations, low Southern Forebay WSELs, and lower tunnel head loss conditions) within the pump's recommended performance range, the rated TDH condition limit is established by the minimum achievable TDH at maximum speed (within the pump manufacturer's allowable performance range) multiplied by 0.25.

For the maximum design capacity range of 4,500 to 7,500 cfs, the estimated design head condition at 450 to 750 cfs (10 percent of the maximum design flow capacities) is approximately 10 feet (static head plus system head loss). Static head conditions associated with the Project's design head conditions associated with this evaluation consider the Sacramento River elevation of 6.9 feet and the Southern Forebay WSEL of 14.8 feet (7.9 feet of static head). Therefore, the manufacturer's recommended minimum TDH operating condition at the pump's maximum speed condition cannot be more than 40 feet (10 feet divided by 0.25).

Figure 9 shows a pump performance curve for the Flygt pump model 180X120 WCF high-capacity pump operating at the rated maximum speed of 200 rpm. This preliminary pump selection was evaluated and found to be feasible for conceptual-level evaluation for the Project. As shown on Figure 9 the minimum recommended head condition (runout head condition) within the pump manufacturer's allowable operating region (AOR) is 30 feet. The minimum head condition within the preferred operating range (POR) is 50 feet at maximum speed. This minimum head condition results in pump operation just outside of the POR but well within the AOR, and stable pump operation would be expected.

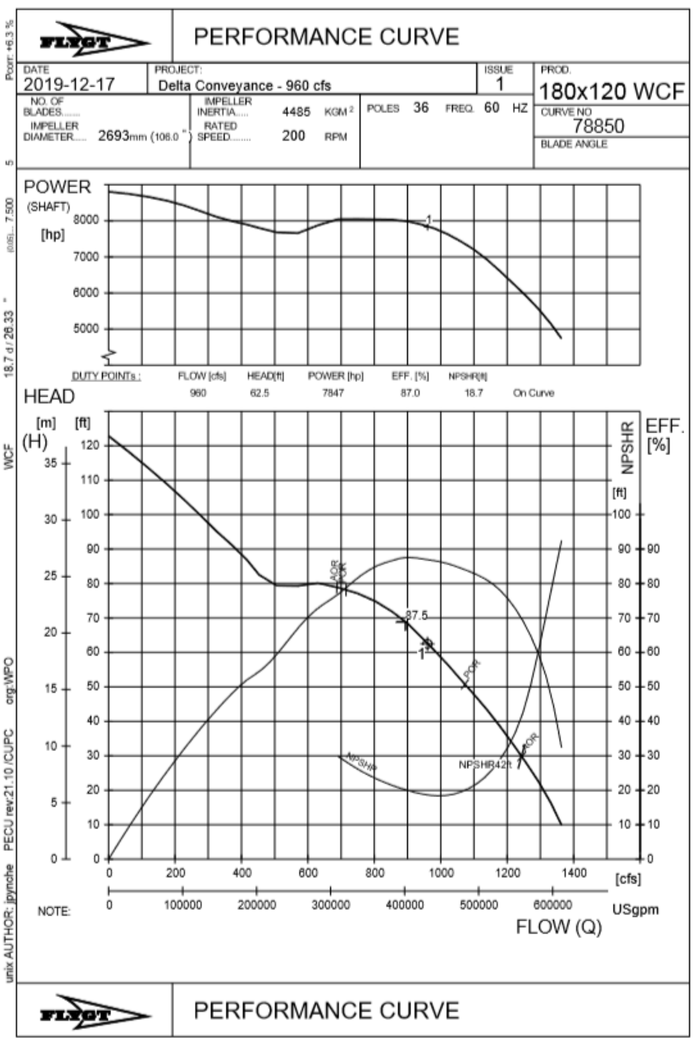

Figure 9. Flygt Pumps' Characteristic Pump Performance Curve, Model 180x120 WCF

The maximum head condition within the pumps' POR is around 79 feet. Note, in this pump characteristic performance curve, the manufacturer has identified the POR and AOR to be essentially the same performance condition. At this level of preliminary analysis, steady-state operation of at least 5 feet below the maximum POR and AOR head of 79 feet is recommended to provide a suitable margin below the AOR head because operation exceeding the AOR head condition is where the manufacturer considers significant pump recirculation to begin. When the margin is applied to the maximum POR and AOR TDH, the maximum operating head condition is limited to 74 feet (79 feet minus 5 feet) at maximum speed.

Based on the Southern Forebay's calculated maximum WSEL of 17.5 feet, and to use the same pump for the entire envelope of total design head conditions, the WSEL in the pumping plant wet well must not be below -56.5 feet (17.5 feet minus 74 feet) for the maximum design flow and TDH conditions associated. Therefore, to use the higher-capacity pumps, head loss associated with each tunnel diameter for each maximum design flow capacity cannot not exceed 58.4 feet based on the minimum Sacramento River elevation of 1.9 feet, the maximum WSEL of 17.5 feet in the Southern Forebay, and the tunnel head loss calculated with a Manning's N of 0.016.

3.2.1 Evaluate High-Capacity Candidate Pumps at Maximum Design Flow Capacity 4,500 cfs

Table 8 shows the head loss summary between C-E-2 and the pumping plant wet well for each tunnel diameter option at the maximum design flow capacity of 4,500 cfs. The steady-state WSELs at the IF and the pumping plant wet well are also shown for each tunnel diameter option. WSELs at the IF and wet well are based on the minimum Sacramento River elevation of 1.9 feet at C-E-2.

Table 8. Head Loss and Water Surface Elevation Summary, Maximum Design Flow Capacity 4,500 cfs

Tunnel Diameter (feet)	Flow Velocity (fps)	Head Loss Intakes to IF (feet)	IF WSEL (feet)	Head Loss IF to Pumping Plant Wet Well (feet)	Pumping Plant Wet Well WSEL (feet)
40	3.6	6.0	-4.1	11.7	-15.8
39	3.8	6.3	-4.4	13.3	-17.7
38	4.0	6.8	-4.9	15.3	-20.2
37	4.2	7.3	-5.4	17.5	-22.9
36	4.4	7.9	-6.0	20.2	-26.2
35	4.7	8.7	-6.8	23.5	-30.3
34	5.0	9.6	-7.7	27.3	-35.0
33	5.3	10.9	-9.0	44.2	-53.2
32	5.6	12.0	-10.1	37.7	-47.8
31	6.0	13.7	-11.8	44.2	-56.0
30	6.4	15.3	-13.4	52.7	-66.1
29	6.8	17.9	-16.0	63.4	-79.4
28	7.3	20.5	-18.6	75.7	-94.3
27	7.9	24.2	-22.3	91.5	-113.8

To use the higher-capacity pumps, the head loss must not be more than 58.4 feet, resulting in a minimum WSEL in the pumping plant wet well of -56.5 feet with the minimum Sacramento River elevation of 1.9 feet at C-E-2. As can be seen in Table 8, the 31-foot-diameter tunnel results in a head loss of 57.9 feet (13.7 feet plus 44.2 feet) and a minimum pumping plant wet well WSEL of -56.0 feet at the maximum design flow capacity of 4,500 cfs. Therefore, tunnel diameters smaller than 31 feet are eliminated from further consideration because their head loss exceeds 58.4 feet and results in WSELs in the pumping plant wet well below -56.5 feet at the maximum design flow condition of 4,500 cfs.

3.2.2 Evaluate High-Capacity Candidate Pumps at Maximum Design Flow Condition 6,000 cfs

Table 9 shows the head loss summary between C-E-2 and the pumping plant wet well for each tunnel diameter option at the maximum design flow capacity of 6,000 cfs. The steady-state WSELs at the IF and the pumping plant wet well are also shown for each tunnel diameter option. WSELs at the IF and wet well are based on the minimum Sacramento River elevation of 1.9 feet at C-E-2.

Table 9. Head Loss and Water Surface Elevation Summary, Maximum Design Flow Capacity 6,000 cfs

Tunnel Diameter (feet)	Flow Velocity (fps)	Head Loss Intakes to IF (feet)	IF WSEL (feet)	Head Loss IF to Pumping Plant Wet Well (feet)	Pumping Plant Wet Well WSEL (feet)
47	3.5	6.9	-5.0	9.2	-14.2
46	3.6	7.2	-5.3	10.2	-15.5
45	3.8	7.4	-5.5	11.4	-16.9
44	3.9	7.8	-5.9	12.8	-18.7
43	4.1	8.2	-6.3	14.4	-20.7
42	4.3	8.6	-6.7	16.3	-23.0
41	4.5	9.1	-7.2	18.5	-25.7
40	4.8	9.7	-7.8	21.0	-28.8
39	5.0	10.4	-8.6	24.0	-32.6
38	5.3	11.3	-9.4	27.5	-36.9
37	5.6	12.3	-10.4	31.7	-42.1
36	6.0	13.6	-11.7	36.8	-48.5
35	6.2	15.2	-13.3	43.6	-56.9
34	6.6	16.0	-14.1	48.7	-62.8
33	7.0	18.4	-16.5	57.8	-74.3
32	7.5	20.0	-18.1	66.6	-84.7
31	7.9	22.5	-20.6	78.5	-99.1

As can be seen in Table 9, the 35-foot-diameter tunnel results in a head loss of 58.8 feet (15.2 feet plus 43.3 feet) and a minimum pumping plant wet well WSEL of -56.9 feet at the maximum design flow capacity of 6,000 cfs. Therefore, tunnel diameters 35 feet and smaller are eliminated from further consideration because their head loss exceeds 58.4feet and results in WSELs in the pumping plant wet well below -56.5 feet at the maximum design flow condition of 6,000 cfs.

3.2.3 Evaluate High-Capacity Candidate Pumps at Maximum Design Flow Condition 7,500 cfs

Table 10 shows the head loss summary between C-E-2 and the pumping plant wet well for each tunnel diameter option at the maximum design flow capacity of 7,500 cfs. The steady-state WSELs at the IF and the pumping plant wet well are also shown for each tunnel diameter option. WSELs at the IF and wet well are based on the minimum Sacramento River elevation of 1.9 feet at C-E-2.

Table 10. Head Loss and Water Surface Elevation Summary, Maximum Design Flow Capacity 7,500 cfs

Tunnel Diameter (feet)	Flow Velocity (fps)	Head Loss (feet)	IF WSEL (feet)	Head Loss IF to Pumping Plant Wet Well (feet)	Pumping Plant Wet Well WSEL (feet)
52	3.5	6.1	-4.2	8.7	-12.9
51	3.7	6.3	-4.4	9.5	-13.9
50	3.8	6.4	-4.5	10.5	-15.0
49	4.0	6.6	-4.7	11.6	-16.3
48	4.1	6.7	-4.8	12.8	-17.6
47	4.3	6.9	-5.0	14.3	-19.3
46	4.5	7.2	-5.3	15.9	-21.2
45	4.7	7.4	-5.5	17.7	-23.2
44	4.9	7.7	-5.8	19.9	-25.7
43	5.2	8.2	-6.3	22.4	-28.7
42	5.4	8.6	-6.7	25.2	-31.9
41	5.7	9.1	-7.2	28.5	-35.7
40	6.0	9.7	-7.8	32.4	-40.2
39	6.3	10.4	-8.5	37.0	-45.5
38	6.6	11.3	-9.4	42.3	-51.7
37	7.0	12.3	-10.4	48.7	-59.1
36	7.4	13.6	-11.7	56.2	-67.9
35	8.0	15.2	-13.3	64.8	-78.1

As can be seen in Table 9, the 38-foot-diameter tunnel results in a head loss of 53.6 feet (11.3 feet plus 42.3 feet) and a minimum pumping plant wet well WSEL of -51.7 feet at the maximum design flow capacity of 7,500 cfs. Therefore, tunnel diameters smaller than 38 feet are eliminated from further consideration because their head loss exceeds 58.4 feet and results in WSELs in the pumping plant wet well below -56.5 feet at the maximum design flow condition of 7,500 cfs.

3.2.4 Tunnel Sizing Summary

Based on the steady-state hydraulic analysis and the high-capacity candidate pump performance envelope criteria, it recommended to maintain the minimum tunnel diameters of:

- 31 feet for the Project maximum design flow capacity option of 4,500 cfs, which results in a flow velocity of 6.0 fps
- 36 feet for the Project maximum design flow capacity option of 6,000 cfs, which results in a flow velocity of 6.0 fps
- 38 feet for the maximum design flow capacity of 7,500 cfs, which results in a flow velocity of 6.6 fps

However, for the purposes of screening feasible tunnel diameters for the Project at this level of conceptual analysis, the maximum tunnel flow velocity is recommended to be limited to a maximum of 6.0 fps for all design flow capacities (4,500, 6,000, and 7,500 cfs) because the system end-to-end InfoWorks ICM model was developed with a tunnel alignment and connecting hydraulic facilities reflecting a conceptual-level design along the central corridor. Other corridors under consideration would require additional tunnel length and additional head loss. At 6 fps design velocity, it is expected the tunnel diameters would be suitable for all corridors.

In summary, the minimum recommended tunnel diameters associated with the maximum flow velocity of 6 fps are:

- 31-foot diameter at the maximum design flow capacity of 4,500 cfs
- 36-foot diameter at the maximum design flow capacity of 6,000 cfs
- 40-foot diameter at the maximum design flow capacity of 7,500 cfs

3.3 Hydraulic Transient-Surge Analysis

In accordance with the methodology and criteria described, a hydraulic transient-surge analysis was performed for the Delta Conveyance system between the C-E-2 drop shaft and the pumping plant wet well. This analysis was conducted to establish the maximum and minimum HGLs along the entire tunnel system alignment resulting from transient-surge events at selected tunnel diameters and design flow conditions.

The maximum tunnel flow velocity of 6 fps was recommended based on the results of the steady-state hydraulic head loss evaluation. For this analysis, tunnel diameters between the C-E-3 drop shaft and the pumping plant wet well were selected based on the tunnel steady-state flow velocities of 6 and 7 fps at the maximum design flow rates of 6,000 and 7,500 cfs, respectively, to evaluate the single tunnel system at the recommended maximum velocity of 6 fps, and to evaluate transient-surge results sensitivity with higher flow velocities at the two design flow conditions of 6,000 and 7,500 cfs. The tunnel diameters and corresponding flow velocities are shown in Table 11 for the two design flow capacities evaluated.

The tunnel design flow capacity of 4,500 cfs was not evaluated in this transient-surge analysis. The tunnel diameters between C-E-2 and C-E-3, and between C-E-5 and the IF were maintained at 28 feet. The lower friction factor (Manning’s N of 0.014) was used for this analysis to provide conservative transient-surge results.

Table 11. Selected Design Flow Capacities and Candidate Tunnel Diameters for Transient-Surge Analysis

Design Flow Capacity = 6,000 cfs		Design Flow Capacity = 7,500 cfs	
Tunnel Diameter (feet)	Tunnel Flow Velocity (fps)	Tunnel Diameter (feet)	Tunnel Flow Velocity (fps)
36	6	40	6
33	7	37	7

Each candidate diameter was evaluated with three IF volumes corresponding with IF facility footprints of 1,500 by 800, 1,000 by 800, and 500 by 500 feet, and with no IF included in the Project. The minimum working WSEL for each IF footprint was set at the steady-state, free WSEL determined in the head loss analysis for each tunnel diameter identified in Table 11 at the design flow conditions of 6,000 and 7,500 cfs. In each condition, initial steady-state conditions were established prior to simulating the transient-surge event. Each candidate tunnel diameter was evaluated with both the maximum and minimum Sacramento River elevations at each intake and with the invert elevation of the overflow weir at the pumping plant overflow shaft set at 19 feet.

Transient surge events were simulated by simultaneously stopping all pumps in operation at the pumping plant, followed by the closure of sediment basin outlet gates within 6 minutes at each intake in operation. Outlet gates remained open in their last steady-state setpoint condition (prior to pump shutdown) for 5 minutes following pump shutdown and were then simultaneously closed within 1 minute. Closure of the intake control gates prevented reverse flow into the Sacramento River from the intakes during the transient-surge event.

Figures 10 through 17 show the hydraulic transient maximum and minimum HGL elevations that occur throughout the transient-surge events along the tunnel alignment using the four IF configurations described. The maximum and minimum HGL elevations are shown as the envelope of the maximum and minimum HGLs and are plotted across the tunnel alignment from the C-E-2 drop shaft to the overflow shaft at the pumping plant. The tunnel and drop shafts at the C-E-2, C-E-3, IF, and pumping plant overflow are depicted and noted in each graph. The tunnel crown elevation for each diameter evaluated is shown as a dashed blue line on each graph. Other shafts, including maintenance shafts along the tunnel alignment, are not shown but were included in the transient-surge model.

The horizontal axis of each graph depicts the tunnel alignment length and general location (in linear feet) of the drop shafts, the IF inlet and outlet drop shafts (near 50,000 feet along the alignment), and the overflow shaft located at the end of the tunnel alignment. The vertical axis defines the invert and crown elevations of the tunnel along the alignment, and the invert and top elevations of the shafts, IF inlet and outlet shafts, and the overflow shafts. The vertical axis also defines the calculated steady-state, and maximum and minimum transient-surge HGLs that are plotted along the alignment.

On Figures 10 through 17, the tunnel diameter between C-E-2 and C-E-3 drop shafts was maintained at 28 feet. The tunnel diameter between the C-E-3 drop shaft and the pumping plant overflow shaft was varied based on Table 11 for each tunnel flow velocity and design flow capacity condition. A reference HGL of 32 feet (depicted as a dashed line across the entire tunnel alignment) is shown to assist in visually inspecting the maximum transient-surge HGL across the alignment for all conditions evaluated.

The invert elevations of the C-E-2 and C-E-3 drop shafts at their tunnel connections are -122.63 feet and -131.04 feet, respectively. For the tunnel to maintain full pressurized flow throughout each section of the alignment, the minimum HGL elevation along the alignment must be higher than the interior crown of the tunnel (invert elevation of the tunnel plus the tunnel finished ID). For example, the crown of the tunnel at the C-E-2 drop shaft is -94.63 feet (invert elevation of -122.63 feet plus 28 feet). The crown of the tunnel at the C-E-3 drop shaft is the invert elevation of -131.04 plus the candidate tunnel diameter being evaluated.

For all IF volume configurations, including when the IF was removed from the tunnel, surge overflow was only permitted at the pumping plant overflow shaft for all transient-surge conditions evaluated. Top-of-shaft elevations for all tunnel shafts along the tunnel alignment (including the C-E-2 and C-E-3 drop shafts) were raised above their respective high-WSELs to contain the tunnel volume so that the maximum resultant maximum HGL conditions along the tunnel alignment could be evaluated.

3.3.1 Condition 1: 36-foot Tunnel Diameter, Tunnel Flow = 6,000 cfs, Minimum Sacramento River Elevation at C-E-2, Tunnel Flow Velocity = 6.0 fps

Figure 10 shows the results of the maximum and minimum transient-surge HGL elevations with a tunnel diameter of 36 feet between the C-E-3 drop shaft and the pumping plant overflow shaft. Prior to the transient-surge condition, the tunnel flow velocity was 6 fps at the design flow condition of 6,000 cfs. To evaluate the lowest resultant minimum transient-surge HGL elevations along the tunnel, the Sacramento River was set to its minimum elevations at C-E-2 and C-E-3.

For the three IF sizes simulated and with the IF removed, the minimum transient-surge HGL elevations are maintained above -94.63 feet in the C-E-2 drop shaft and -95.04 feet in the C-E-3 drop shaft, and above the crown of the tunnel along the entire alignment.

The minimum HGL condition along the alignment for each of the three IF volumes and with the IF removed occurs at the C-E-2 drop shaft. The minimum WSEL within the C-E-2 drop shaft is just below -50 feet for all IF configurations evaluated. For the three IF volumes, the minimum HGL was generally equivalent to steady-state conditions for the tunnel section between the IF and pumping plant overflow shaft. For the configuration with the IF removed, the minimum HGL was generally equivalent to the steady-state conditions within the last 50,000 feet of the tunnel. No negative pressures were determined to be within the tunnel sections throughout the entire alignment for all IF configurations.

The resultant maximum HGL elevation for all IF configurations occurred at the C-E-2 drop shaft. The highest resultant maximum HGL occurred with the IF footprint volumes of 800 feet by 1,500 feet (width by length), which was slightly above 32 feet. The maximum HGL elevation at C-E-2 drop shaft for the IF footprint configuration of 800 feet by 1,000 feet was 32 feet. For the IF configuration of 500 feet by 500 feet and with the IF removed, the maximum HGL at the C-E-2 drop shaft was below 32 feet. For the configuration with the IF removed, the maximum HGL remained generally linear, below 32 feet along the entire tunnel alignment.

3.3.2 Condition 2: 36-foot Tunnel Diameter, Tunnel Flow = 6,000 cfs, Maximum Sacramento River Elevation at C-E-2, Tunnel Flow Velocity = 6.0 fps

Figure 11 shows the results of the maximum and minimum transient-surge HGL elevations with a tunnel diameter of 36 feet between the C-E-3 drop shaft and the pumping plant overflow shaft with the maximum Sacramento River elevations at C-E-2 and C-E-3, and an initial steady-state flow of 6,000 cfs and flow velocity of 6 fps. This analysis was conducted to evaluate the highest resultant transient-surge HGL elevations along the tunnel at this design flow condition.

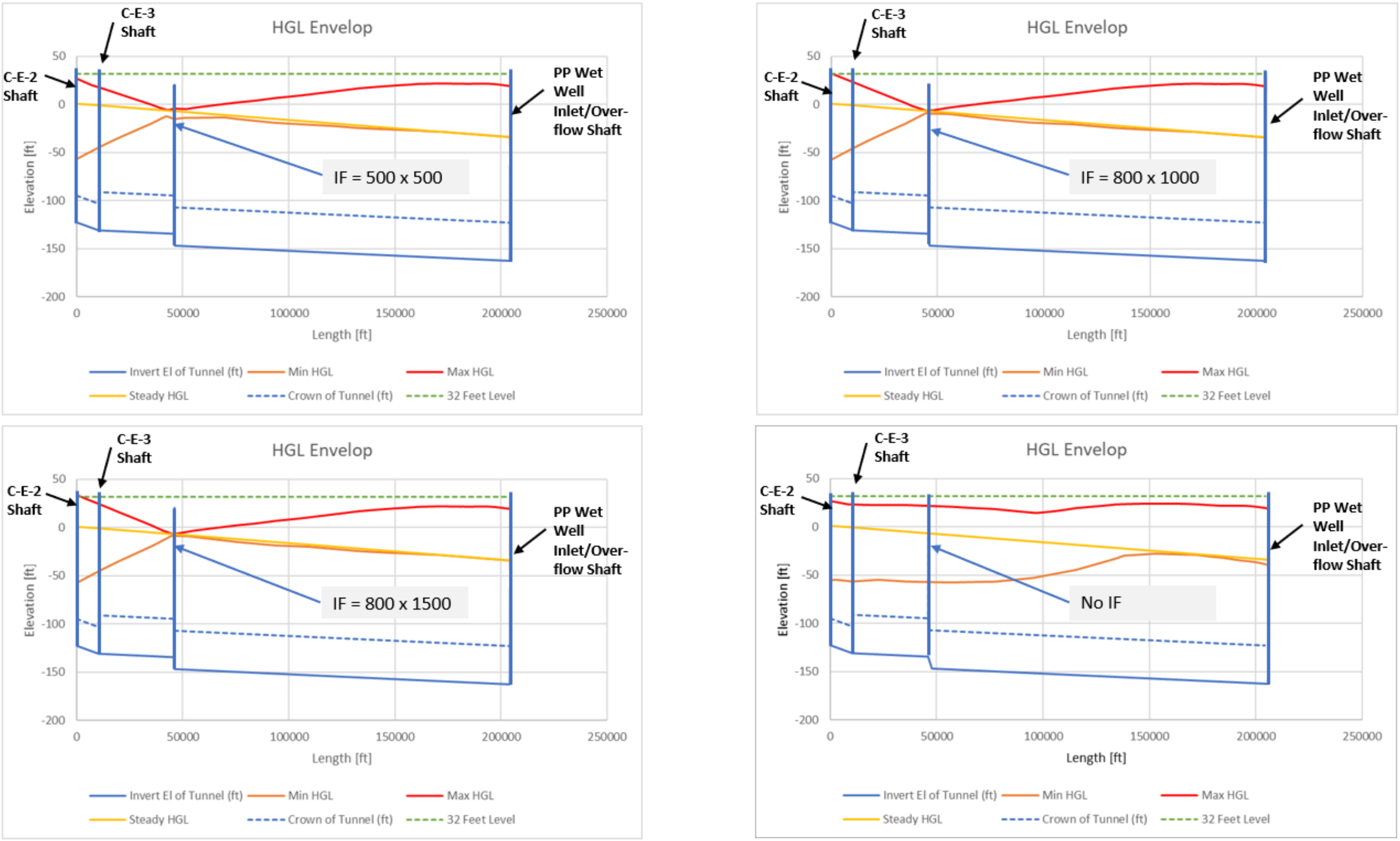

Figure 10. Condition 1, Maximum and Minimum Hydraulic Grade Lines

Note: Graphs show minimum Sacramento River elevation, tunnel flow velocity of 6 fps at 6,000 cfs design flow capacity using various IF sizes

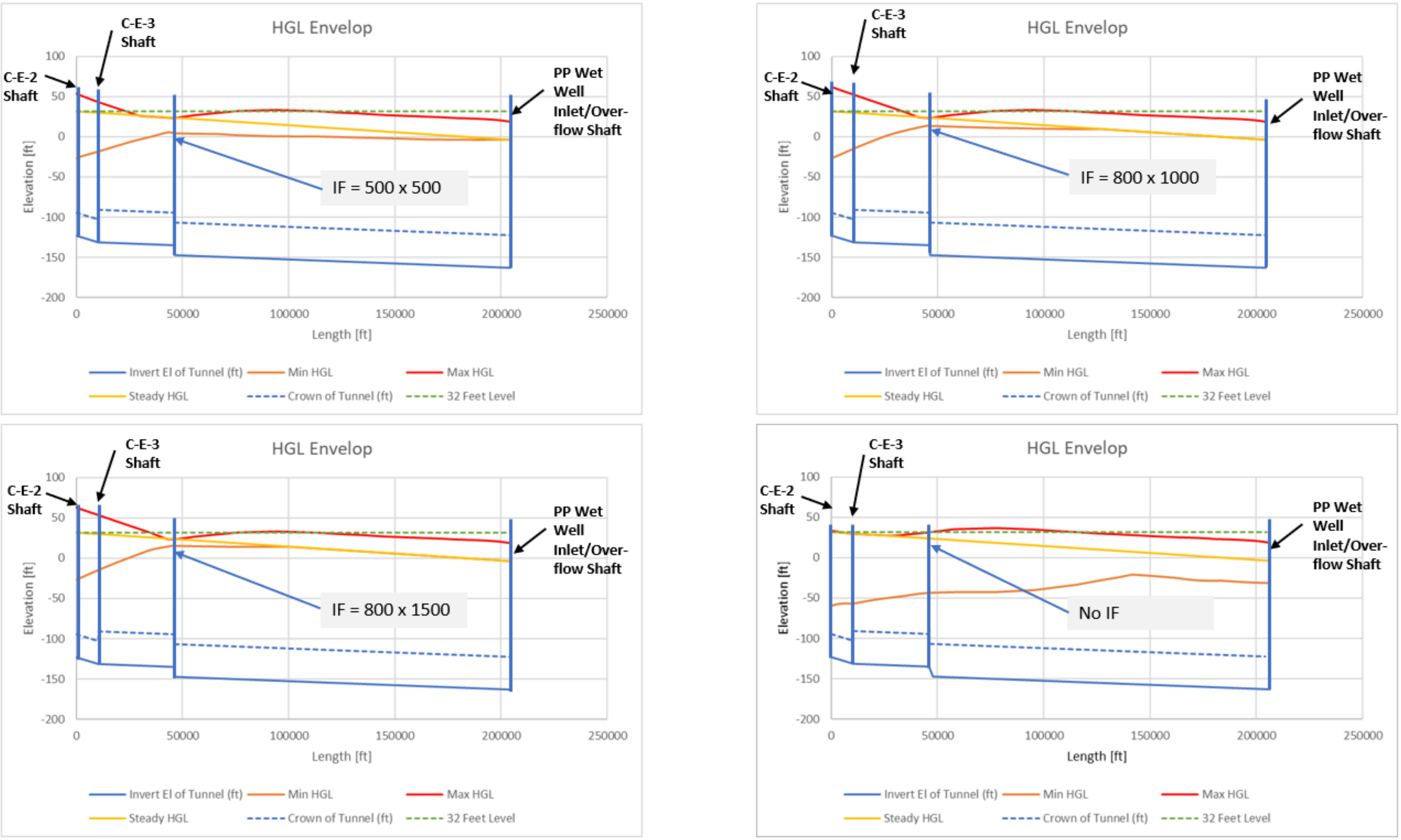

Figure 11. Condition 2, Maximum and Minimum Hydraulic Grade Lines

Note: Graphs show maximum Sacramento River elevation, tunnel flow velocity of 6 fps at 6,000 cfs design flow capacity using various IF sizes

For the three IF sizes simulated and with the IF removed, the minimum transient-surge HGL elevations are maintained well above -94.63 feet in the C-E-2 drop shaft and -95.04 feet in the C-E-3 drop shaft, and above the crown of the tunnel along the entire alignment.

The minimum HGL condition along the alignment for each of the three IF volumes and the configuration with the IF removed occurs at the C-E-2 drop shaft. The minimum WSEL within the C-E-2 drop shaft was generally the same for the three IF footprint volumes (approximately -25 feet). For the tunnel configuration with the IF removed, the WSEL in the C-E-2 drop shaft falls just below -50 feet and is generally the same as that determined for Condition 1. This is primarily due to the higher Sacramento River elevations at C-E-2 and C-E-3 (under Condition 2 versus Condition 1), which caused a larger overflow volume at the pumping plant overflow shaft in Condition 2 than the overflow volume that occurred in Condition 1 (with the lower Sacramento River elevations at C-E-2 and C-E-3). The differences in these overflow volumes result in similar minimum WSELs at the C-E-2 drop shaft for both Conditions 1 and 2. No negative pressures were determined to be within the tunnel sections throughout the entire alignment.

The resultant maximum HGL elevation for each of the three IF volumes occur at the C-E-2 drop shaft. The maximum HGL elevations for the two IF footprints of 800 feet by 1,500 feet and 800 feet by 1,000 feet were well above 50 feet. The maximum HGL elevation for the IF footprint of 500 feet by 500 feet was slightly above 50 feet. For all IF footprint volumes, the HGL was slightly above 32 feet immediately downstream of the IF location and remained at or just below 32 feet (and well above the steady-state HGL) for the remaining tunnel section between the IF and the pumping plant overflow shaft. With the IF removed from the tunnel, the maximum HGL occurred downstream of the shaft, where the IF was previously located, and did not exceed the steady-state HGL at the C-E-2 drop shaft. With the IF removed, the maximum HGL profile was generally uniform between the IF and the pumping plant overflow shaft. Based on the maximum HGL elevations described for this transient-surge condition, an overflow condition would occur at the C-E-2 and C-E-3 facilities for all IF footprint volume configurations. An overflow condition at the C-E-2 or the C-E-3 facilities would not occur for the tunnel configuration with the IF removed.

3.3.3 Condition 3: 33-foot Tunnel Diameter, Tunnel Flow = 6,000 cfs, Minimum Sacramento River Elevation at C-E-2, Tunnel Flow Velocity = 7.0 fps

Figure 12 shows the results of the maximum and minimum transient-surge HGL elevations with a tunnel diameter of 33 feet between the C-E-3 drop shaft and the pumping plant overflow shaft. Prior to the transient-surge condition, the tunnel flow velocity was 7 fps at the design flow condition of 6,000 cfs. To evaluate the lowest resultant minimum transient-surge HGL elevations along the tunnel, the Sacramento River was set to its minimum elevations at C-E-2 and C-E-3.

For the three IF sizes simulated and with the IF removed, the minimum transient-surge HGL elevations are maintained well above -94.63 feet in the C-E-2 drop shaft and -98.04 feet in the C-E-3 drop shaft, and above the crown of the tunnel along the entire alignment.

The minimum HGL condition along the alignment for each of the three IF volumes and with the IF removed occurs at the C-E-2 drop shaft. The minimum WSEL within the C-E-2 drop shaft is approximately -70 feet for all IF configurations evaluated and is similar to the Condition 1 results. For the three IF volumes, the minimum HGL was generally equivalent to steady-state conditions for the tunnel section between the IF and pumping plant overflow shaft. No negative pressures were determined to be within the tunnel sections throughout the entire alignment. Based on these results increasing the flow velocity from 6 to 7 fps had little effect to the minimum HGL when comparing the results of Condition 3 to Condition 1.

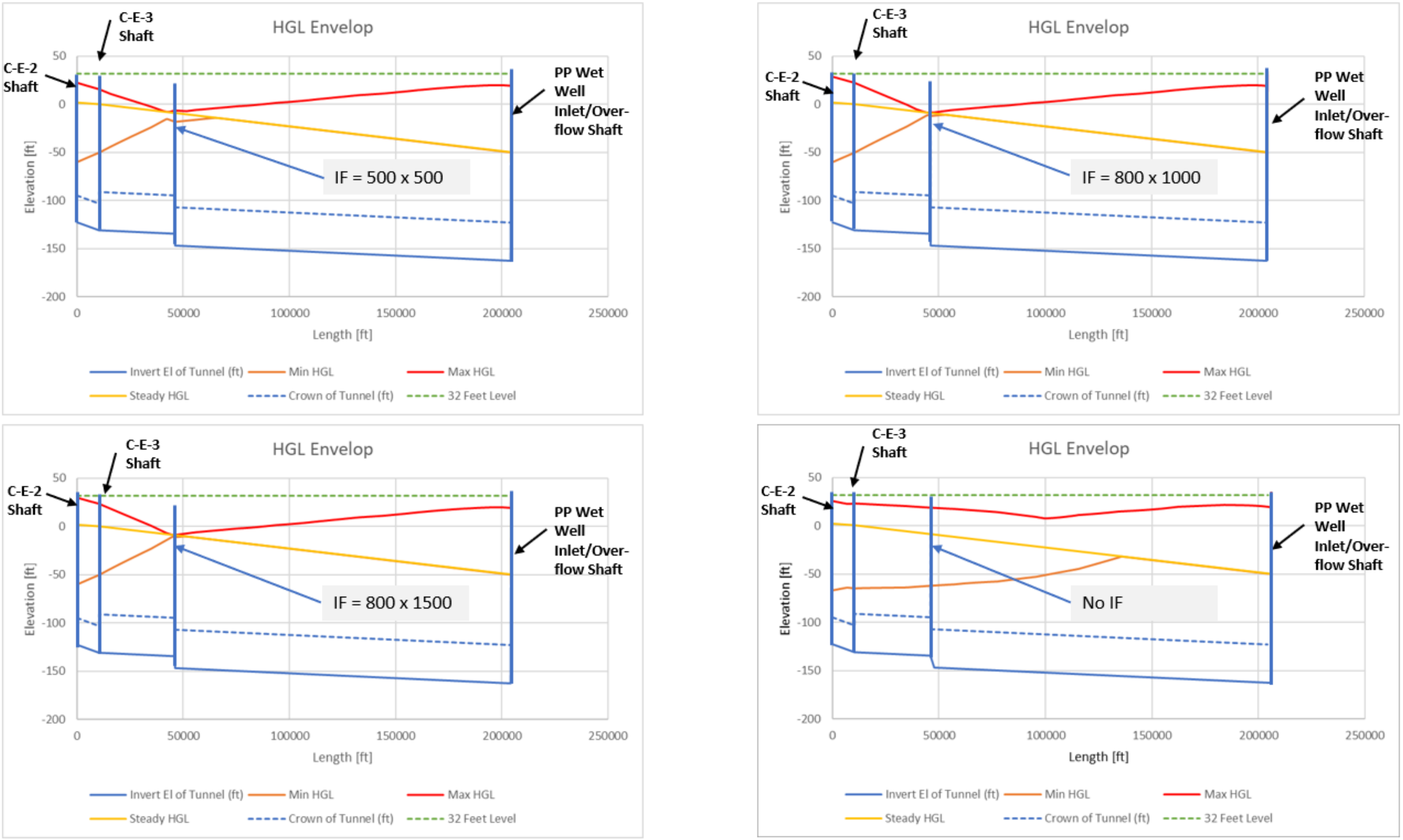

Figure 12. Condition 3, Maximum and Minimum Hydraulic Grade Lines

Note: Graphs show minimum Sacramento River elevation, tunnel flow velocity of 7 fps at 6,000 cfs design flow capacity using various IF sizes

The resultant maximum HGL elevation for all IF configurations occurred at the C-E-2 drop shaft. The highest resultant maximum HGL occurred with the IF footprint configuration of 800 feet by 1,500 feet, which was 32 feet. The maximum HGL elevations associated with the IF footprint configurations of 800 feet by 1,000 feet and with the IF removed were slightly below the maximum level determined for the 800-foot by 1,500-foot footprint configuration. For the configuration with the IF removed, the maximum HGL remained generally linear along the entire alignment. Based on these results, increasing the flow velocity from 6 to 7 fps had little effect to the maximum HGL when comparing the results of Condition 3 to Condition 1.

3.3.4 Condition 4: 33-foot Tunnel Diameter, Tunnel Flow = 6,000 cfs, Maximum Sacramento River Elevation at C-E-2, Tunnel Flow Velocity = 7.0 fps

Figure 13 shows the results of the maximum and minimum transient-surge HGL elevations with a tunnel diameter of 33 feet between the C-E-3 drop shaft and the pumping plant overflow shaft with the maximum Sacramento River elevations at C-E-2 and C-E-3, and an initial steady-state flow of 6,000 cfs and corresponding flow velocity of 7 fps. This analysis was conducted to evaluate the highest resultant transient-surge HGL elevations along the tunnel at this design flow condition.

For the three IF sizes simulated and with the IF removed, the minimum transient-surge HGL elevations are maintained well above -94.63 feet in the C-E-2 drop shaft and -98.04 feet in the C-E-3 drop shaft, and above the crown of the tunnel along the entire alignment.

The minimum HGL condition along the alignment for each of the three IF volumes and with the IF removed occurs at the C-E-2 drop shaft. The minimum WSEL within the C-E-2 drop shaft was generally the same for the three IF footprint configurations (approximately -25 feet) and similar to the results in Condition 2. For the tunnel configuration with the IF removed, the WSEL in the C-E-2 drop shaft falls below -50 feet and is about the same as Condition 3. This is primarily due to the higher Sacramento River elevations at C-E-2 and C-E-3 (under Condition 4 versus Condition 3), which caused a larger overflow volume at the pumping plant overflow shaft in Condition 4 than the overflow volume in Condition 3 (with the lower Sacramento River elevations at C-E-2 and C-E-3). The differences in these overflow volumes result in similar minimum WSELs at the C-E-2 drop shaft for both Conditions 3 and 4. No negative pressures were determined to be within the tunnel sections throughout the entire alignment. As such, increasing the flow velocity from 6 to 7 fps had little effect to the minimum HGL when comparing Condition 2 and Condition 4.

The resultant maximum HGL elevations for all IF volumes occur at the C-E-2 drop shaft. The maximum HGL for the IF footprints of 800 feet by 1,500 feet and 800 feet by 1,000 feet was well above 50 feet. The maximum HGL for the IF footprint of 500 feet by 500 feet was slightly below 50 feet. For all IF footprint volumes, the HGL was slightly above 32 feet immediately downstream of the IF location and remained at or just below 32 feet (and well above the steady-state HGL) for the remaining tunnel section between the IF and the pumping plant overflow shaft. With the IF removed, the maximum HGL occurred downstream of the shaft, where the IF was previously located, and did not exceed the steady-state HGL at the C-E-2 drop shaft. For all IF configurations, the maximum HGL profile was generally uniform between the IF and the pumping plant overflow shaft.

Based on the results described for this transient-surge condition, overflow would occur at the C-E-2 and C-E-3 facilities for all IF footprint volumes. Overflow at the C-E-2 or C-E-3 facilities would not occur for the tunnel configuration with the IF removed. Based on these results, increasing the flow velocity from 6 to 7 fps had little effect to the maximum HGL when comparing Condition 2 and Condition 4.

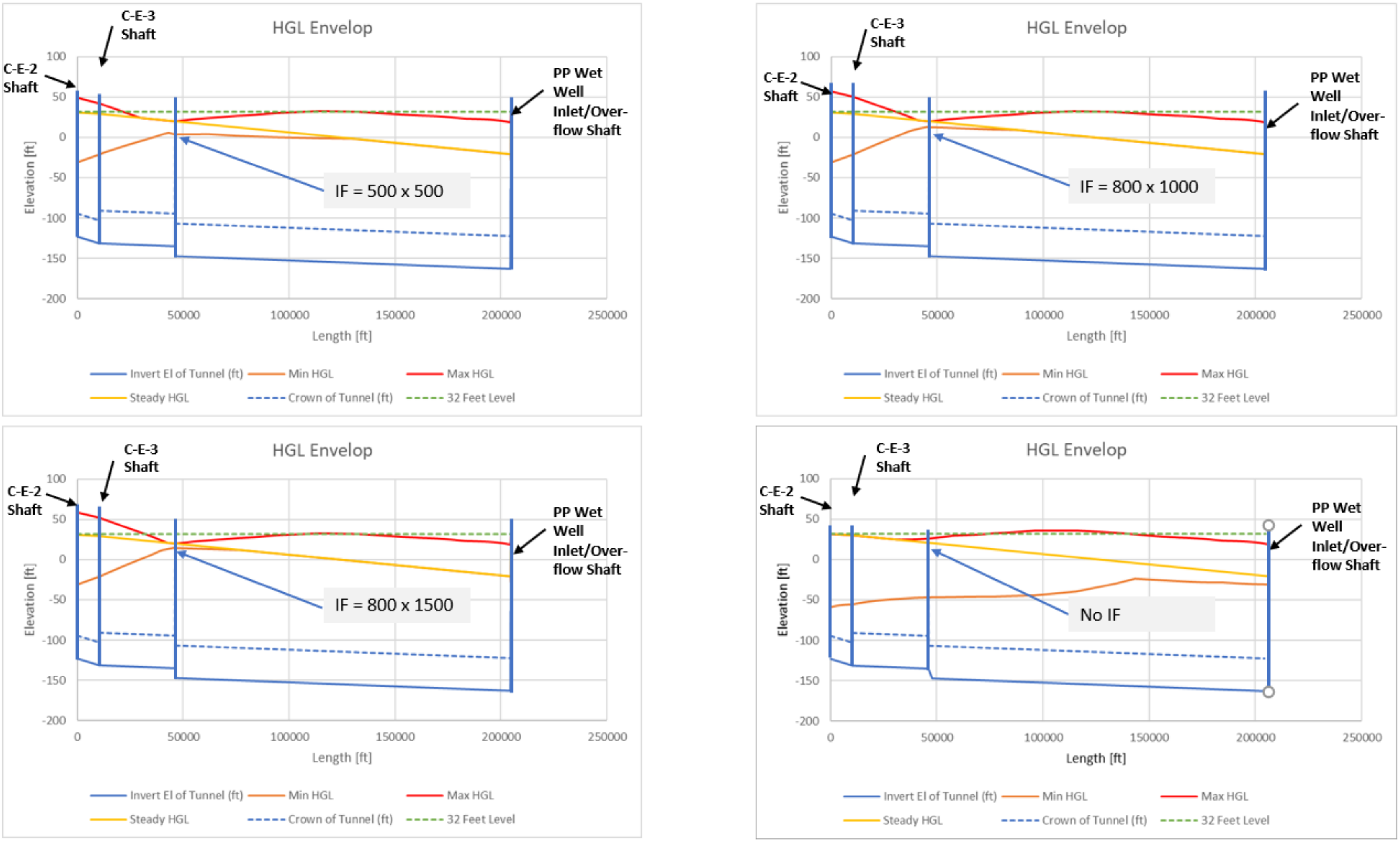

Figure 13. Condition 4, Maximum and Minimum Hydraulic Grade Lines

Notes: Graphs show maximum Sacramento River elevation, tunnel flow velocity of 7 fps at 6,000 cfs design flow capacity using various IF sizes

3.3.5 Condition 5: 40-foot Tunnel Diameter, Tunnel Flow =7,500 cfs, Minimum Sacramento River Elevation at C-E-2, Tunnel Flow Velocity = 6.0 fps

Figure 14 shows the results of the maximum and minimum transient-surge HGL elevations with a tunnel diameter of 40 feet between the C-E-3 drop shaft and the pumping plant overflow shaft. Prior to the transient-surge condition, the tunnel flow velocity was 6 fps at the design flow condition of 7,500 cfs. To evaluate the lowest resultant minimum transient-surge HGL elevations along the tunnel, the Sacramento River was set to its minimum elevations at C-E-2 and C-E-3.

For the three IF sizes simulated and with the IF removed, the minimum transient-surge HGL elevations are maintained above -94.63 feet in the C-E-2 drop shaft and -91.04 feet in the C-E-3 drop shaft, and above the crown of the tunnel along the entire alignment.

The minimum HGL condition along the alignment for each of the three IF volumes and with the IF removed occurs at the C-E-2 drop shaft. The minimum WSEL within the C-E-2 drop shaft is approximately below -60 feet for all IF configurations evaluated. For the three IF volumes, the minimum HGL was generally slightly below steady-state conditions for the tunnel section between the IF and pumping plant overflow shaft. For the configuration with the IF removed, the minimum HGL was below the steady-state conditions along the entire tunnel alignment. No negative pressures were determined to be within the tunnel sections throughout the entire alignment for all IF configurations.

The resultant maximum HGL elevation for all IF configurations occurred at the C-E-2 drop shaft. The resultant maximum HGL with the IF footprint volumes of 800 feet by 1,500 feet and 800 feet by 1,000 feet was well above 40 feet. The maximum HGL elevations at C-E-2 drop shaft for the IF footprint configuration of 800 feet by 1,000 feet was slightly above 32 feet. For the IF configuration of 500 feet by 500 feet and with the IF removed, the maximum HGL at the C-E-2 drop shaft was 32 feet. For the configuration with the IF removed, the maximum HGL remained generally linear just below 32 feet along the entire alignment.

Based on the results described for this transient-surge condition, overflow would occur at the C-E-2 and C-E-3 facilities for the IF footprint volumes of 800 feet by 1,500 feet and 800 feet by 1,000 feet. Overflow at the C-E-2 or C-E-3 facilities would not occur for the tunnel configuration with the IF removed and the IF configuration footprint volume of 500 feet by 500 feet.

3.3.6 Condition 6: 40-foot Tunnel Diameter, Tunnel Flow =7,500 cfs, Maximum Sacramento River Elevation at C-E-2, Tunnel Flow Velocity = 6.0 fps

Figure 15 shows the results of the maximum and minimum transient-surge HGL elevations with a tunnel diameter of 40 feet between the C-E-3 drop shaft and the pumping plant overflow shaft with the maximum Sacramento River elevations at C-E-2 and C-E-3, and an initial steady-state flow of 7,500 cfs. This analysis was conducted to evaluate the highest resultant transient-surge HGL elevations along the tunnel at this design flow condition.

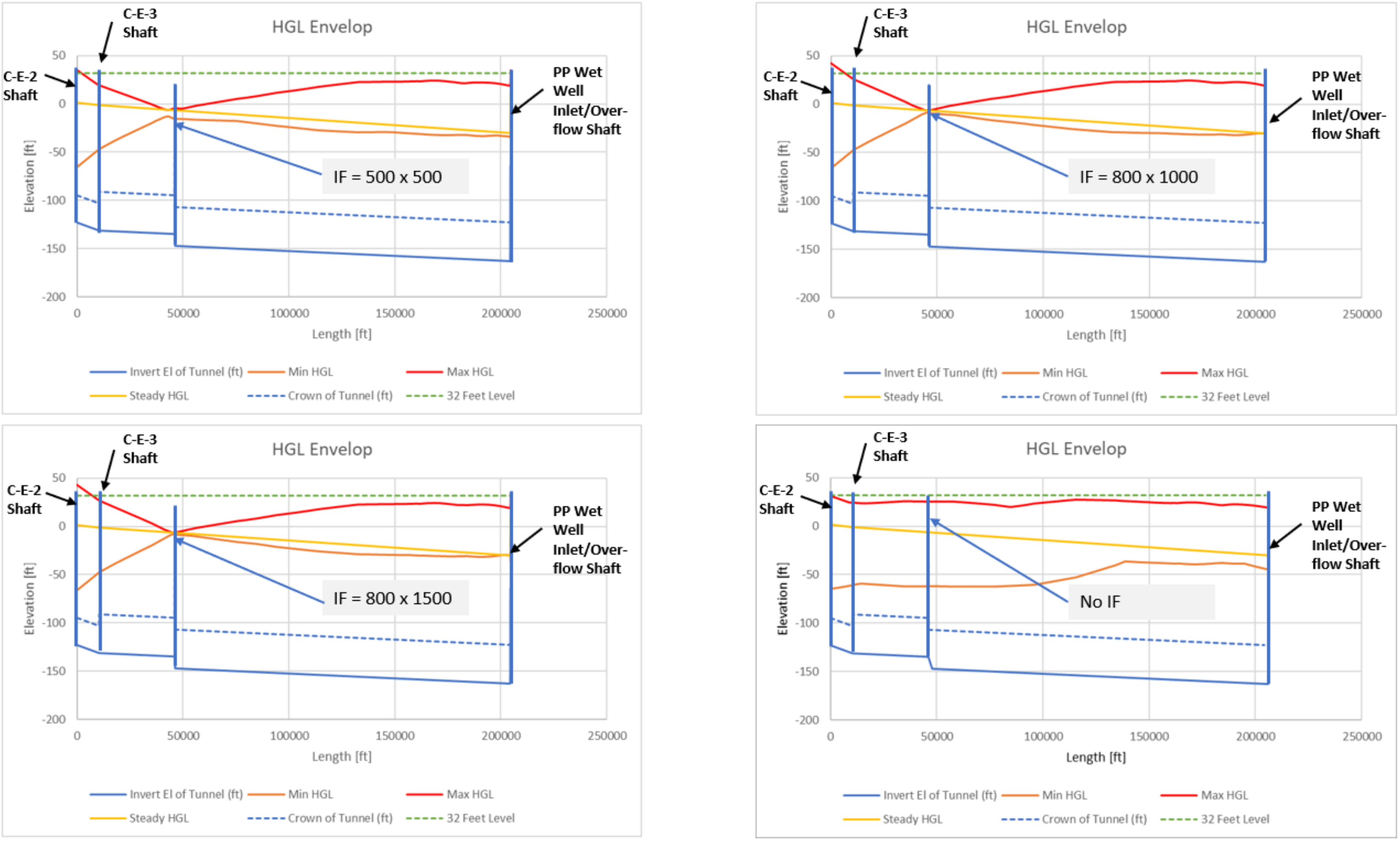

Figure 14. Condition 5, Maximum and Minimum Hydraulic Grade Lines

Notes: Graphs show minimum Sacramento River elevation, tunnel flow velocity of 6 fps at 7,500 cfs design flow capacity using various IF sizes

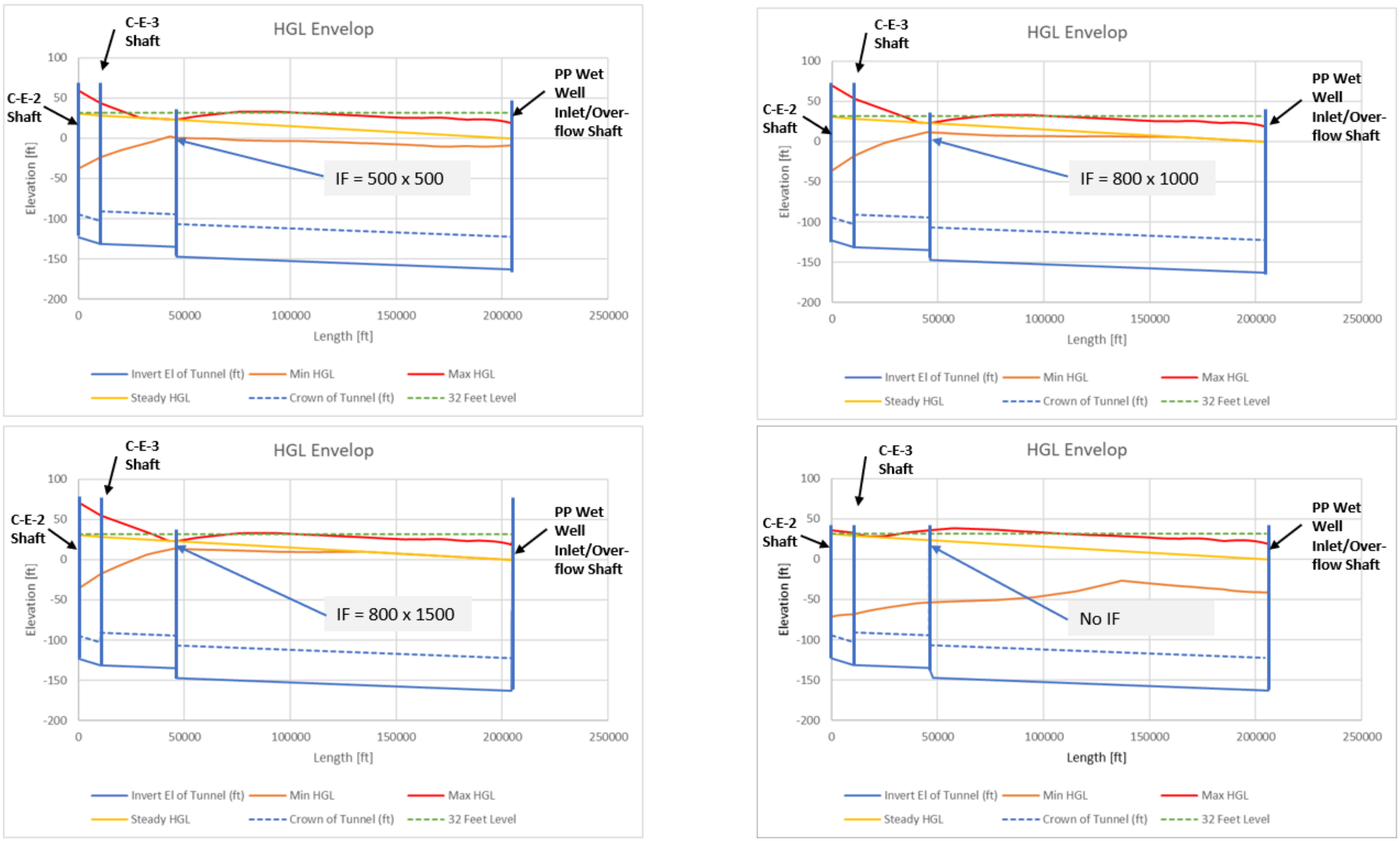

Figure 15. Condition 6, Maximum and Minimum Hydraulic Grade Lines

Note: Graphs show maximum Sacramento River elevation, tunnel flow velocity of 6 fps at 7,500 cfs design flow capacity using various IF sizes

For the three IF sizes simulated and with the IF removed, the minimum transient-surge HGL elevations are maintained above -94.63 feet in the C-E-2 drop shaft and -91.04 feet in the C-E-3 drop shaft, and above the crown of the tunnel along the entire alignment.

The minimum HGL condition along the alignment for each of the three IF volumes and with the IF removed occurs at the C-E-2 drop shaft. The minimum WSEL within the C-E-2 drop shaft was generally the same for the three IF footprint volumes (approximately -40 feet). For the tunnel configuration with the IF removed, the WSEL in the C-E-2 drop shaft falls to just above -75 feet and is generally the same as that determined for Condition 5. This is primarily due to the higher Sacramento River elevations at C-E-2 and C-E-3 (under Condition 6 versus Condition 5), which caused a larger overflow volume at the pumping plant overflow shaft in Condition 6 than the overflow volume that occurred in Condition 5 (with the lower Sacramento River elevations at C-E-2 and C-E-3). The difference in these overflow volumes resulted in similar minimum WSELs at the C-E-2 drop shaft for both Conditions 5 and 6. No negative pressures were determined to be within the tunnel sections throughout the entire alignment.

The resultant maximum HGL elevations for all IF volumes occur at the C-E-2 drop shaft. The maximum HGL for the IF footprints of 800 feet by 1,500 feet and 800 feet by 1,000 feet was well above 60 feet. The maximum HGL for the IF footprint of 500 feet by 500 feet was slightly above 50 feet. For all IF footprint volumes, the HGL was slightly above 32 feet immediately downstream of the IF location and remained at or just below 32 feet (and well above the steady-state HGL) for the remaining tunnel section between the IF and the pumping plant overflow shaft. With the IF removed, the maximum HGL at the C-E-2 drop shaft was generally equal to the steady-state condition, with the maximum HGL occurring downstream of the shaft, where the IF was previously located, which was slightly above 32 feet and then remained generally uniform between the IF and the pumping plant overflow shaft.

Based on the results described for this transient-surge condition, an overflow condition would occur at the C-E-2 and C-E-3 facilities for all IF footprint volumes. Overflow at the C-E-2 or C-E-3 facilities would not occur for the tunnel configuration with the IF removed.

3.3.7 Condition 7: 37-foot Tunnel Diameter, Tunnel Flow =7,500 cfs, Minimum Sacramento River Elevation at C-E-2, Tunnel Flow Velocity = 7.0 fps

Figure 16 shows the results of the maximum and minimum transient-surge HGL elevations with a tunnel diameter of 37 feet between the C-E-3 drop shaft and the pumping plant overflow shaft. Prior to the transient-surge condition, the tunnel flow velocity was 7 fps at the design flow condition of 7,500 cfs. To evaluate the lowest resultant minimum transient-surge HGL elevations along the tunnel, the Sacramento River was set to its minimum elevations at C-E-2 and C-E-3.

For the three IF sizes simulated, the minimum transient-surge HGL elevations are maintained well above -94.63 feet in the C-E-2 drop shaft and -94.04 feet in the C-E-3 drop shaft, and above the crown of the tunnel along the entire alignment. For the tunnel system with the IF removed, the minimum HGL fell below the crown of the tunnel at the C-E-2 drop shaft (admitting atmospheric air) and generally matched the crown of the tunnel at the C-E-3 drop shaft, allowing atmospheric air into the tunnel. Upon inspection of the results, negative pressures within the tunnel were not found.

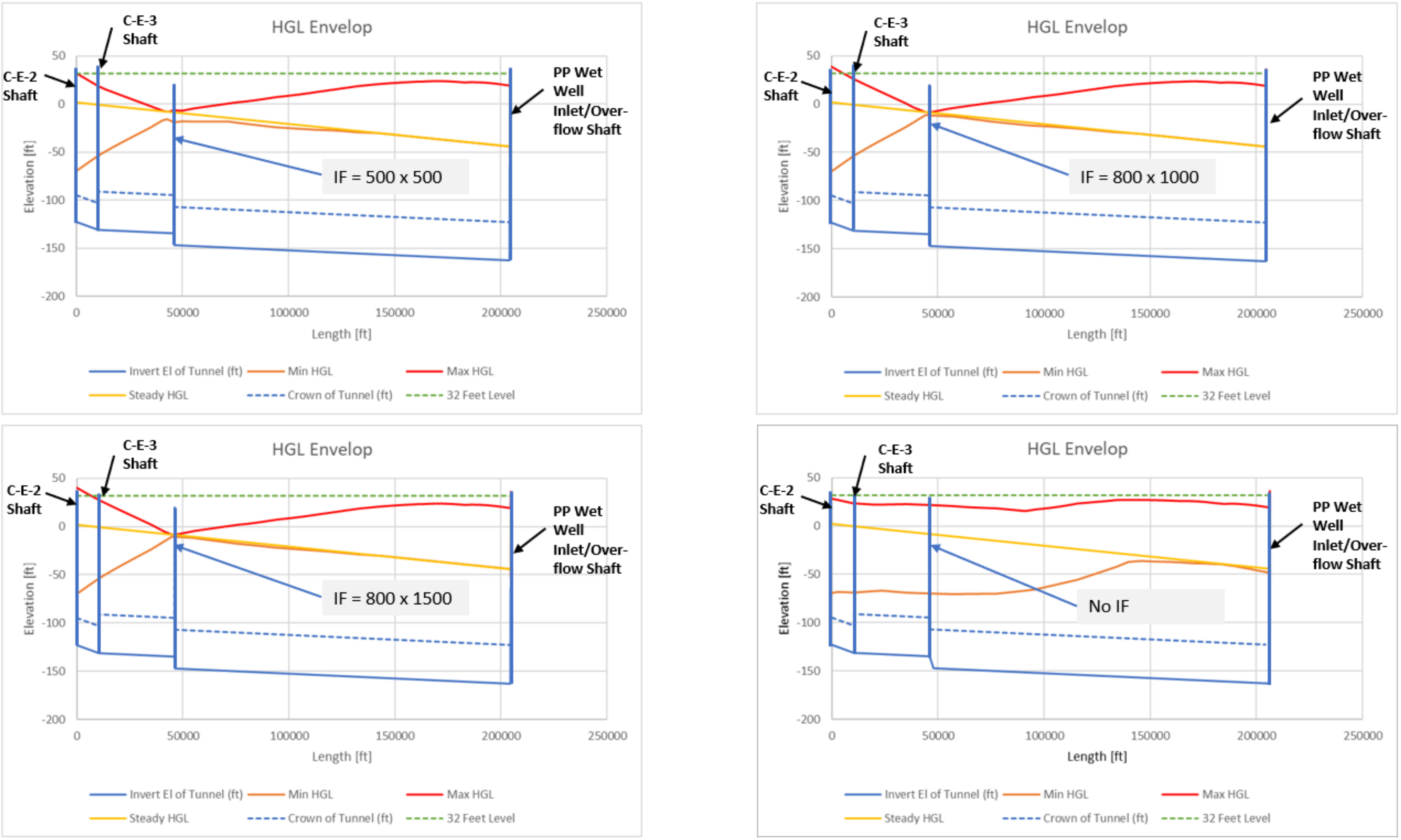

Figure 16. Condition 7, Maximum and Minimum Hydraulic Grade Lines

Note: Graphs show minimum Sacramento River elevation, tunnel flow velocity of 7 fps at 7,500 cfs design flow capacity using various IF sizes

The minimum HGL condition along the alignment for each of the three IF volumes and with the IF removed occurs at the C-E-2 drop shaft. The minimum WSEL within the C-E-2 drop shaft is approximately -70 feet for all IF configurations evaluated and is similar to the Condition 5 results. For the three IF volumes, the minimum HGL was generally equivalent to steady-state conditions for the tunnel section between the IF and pumping plant overflow shaft. No negative pressures were determined to be within the tunnel sections throughout the entire alignment. Based on these results, increasing the flow velocity from 6 to 7 fps had little effect to the minimum HGL when comparing the results of Condition 7 to Condition 5.

The resultant maximum HGL elevation for all IF configurations occurred at the C-E-2 drop shaft. The highest resultant maximum HGL occurred with the IF footprint configurations of 800 feet by 1,500 feet and 800 feet by 1,000 feet, which was above 32 feet. The maximum HGL elevations associated with the IF footprint configurations of 500 feet by 500 feet was 32 feet. The maximum HGL with the IF removed from the tunnel was below 32 feet. For the configuration with the IF removed, the maximum HGL remained generally linear along the entire alignment. Based on these results, increasing the flow velocity from 6 to 7 fps had little effect to the maximum HGL when comparing Condition 7 to Condition 5 results.

3.3.8 Condition 8: 37-foot Tunnel Diameter, Tunnel Flow =7,500 cfs, Maximum Sacramento River Elevation at C-E-2, Tunnel Flow Velocity = 7.0 fps

Figure 17 shows the results of the maximum and minimum transient-surge HGL elevations with a tunnel diameter of 37 feet between the C-E-3 drop shaft and the pumping plant overflow shaft with the maximum Sacramento River elevations at C-E-2 and C-E-3, and an initial steady-state flow of 7,500 cfs and corresponding flow velocity of 7 fps. This analysis was conducted to evaluate the highest resultant transient-surge HGL elevations along the tunnel at this design flow condition.

For the three IF sizes simulated and with the IF removed, the minimum transient-surge HGL elevations are maintained well above -94.63 feet in the C-E-2 drop shaft and -94.04 feet in the C-E-3 drop shaft, and above the crown of the tunnel along the entire alignment.

The minimum HGL condition along the alignment for each of the three IF volumes and with the IF removed occurs at the C-E-2 drop shaft. The minimum WSEL within the C-E-2 drop shaft was generally the same for the three IF footprint configurations (approximately -40 feet) and similar to the results in Condition 6. For the tunnel configuration with the IF removed, the WSEL in the E-C-2 drop shaft falls below -50 feet and is about the same as Condition 6. This is primarily due to the higher Sacramento River elevations at C-E-2 and C-E-3 (under Condition 8 versus Condition 6), which caused a larger overflow volume at the pumping plant overflow shaft in Condition 8 than the overflow volume that occurred in Condition 6 (with the lower Sacramento River elevations at C-E-2 and C-E-3). The differences in these overflow volumes result in similar minimum WSELs at the C-E-2 drop shaft for both Conditions 6 and 8. No negative pressures were determined to be within the tunnel sections throughout the entire alignment. As such, increasing the flow velocity from 6 to 7 fps had little effect to the minimum HGL when comparing the results of Condition 6 and Condition 8.

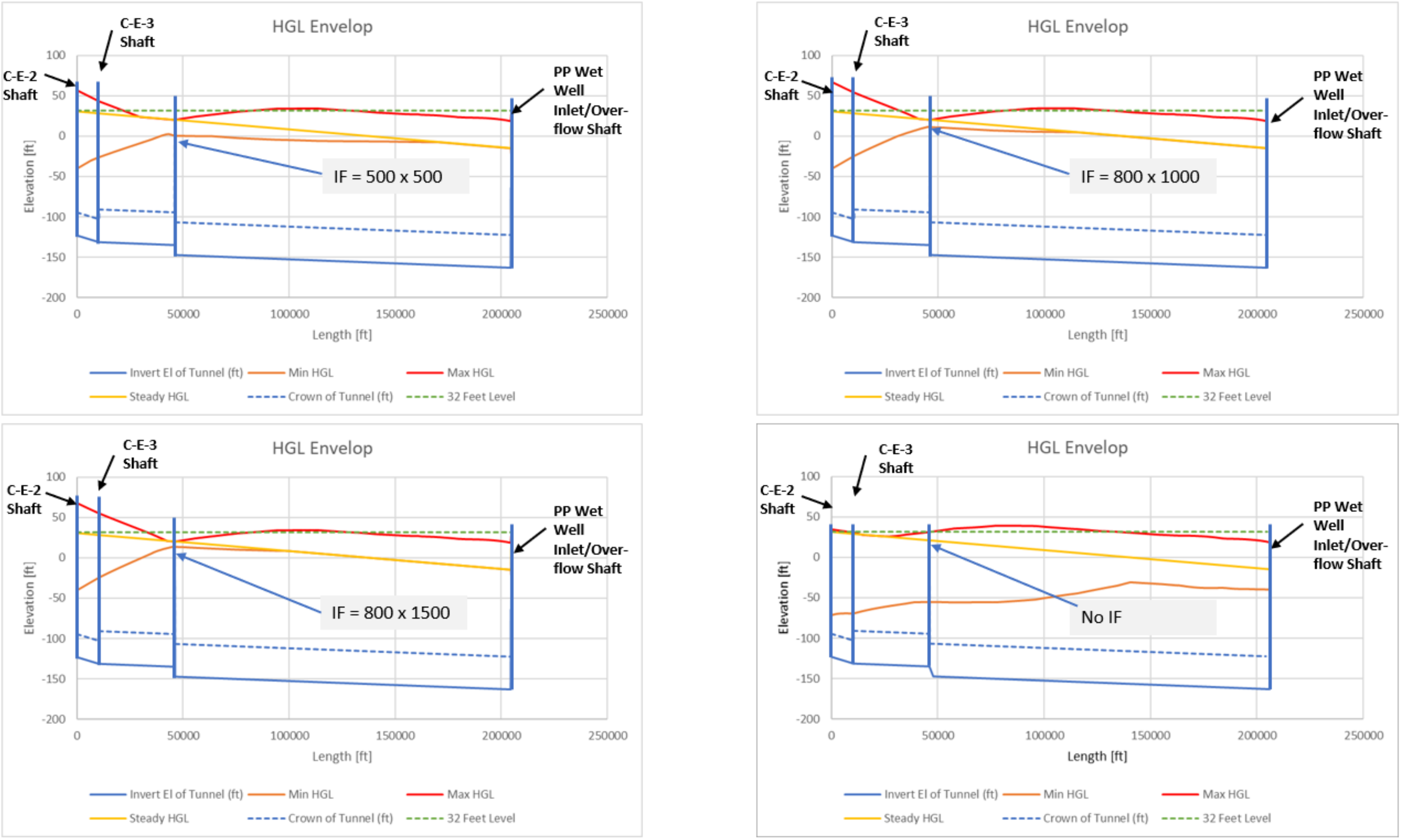

Figure 17. Condition 8, Maximum and Minimum Hydraulic Grade Lines

Note: Graphs show maximum Sacramento River elevation, tunnel flow velocity of 7 fps at 7,500 cfs design flow capacity using various IF sizes

The resultant maximum HGL elevations for all three IF volumes configurations occur at the C-E-2 drop shaft. The maximum HGL for the IF volume footprints of 800 feet by 1,500 feet and 800 feet by 1,000 feet was well above 50 feet. The maximum HGL for the IF footprint of 500 feet by 500 feet was slightly below 50 feet. For all IF footprint volumes, the HGL was slightly above 32 feet immediately downstream of the IF location and remained at or just below 32 feet (and well above the steady-state HGL) for the remaining tunnel section between the IF and the pumping plant overflow shaft. With the IF removed, the maximum HGL occurred downstream of the shaft, where the IF was previously located, and did not exceed the steady-state HGL at the C-E-2 drop shaft. For all IF configurations, the maximum HGL profile was generally uniform between the IF shaft and the pumping plant overflow shaft.

Based on the results described for this transient-surge condition, an overflow condition would occur at the C-E-2 and C-E-3 facilities for all IF footprint volume configurations. An overflow at the C-E-2 or C-E-3 facilities would not occur for the tunnel configuration with the IF removed. Based on these results, increasing the flow velocity from 6 to 7 fps had little effect to the maximum HGL when comparing the results of Condition 6 and Condition 8.

3.3.9 Hydraulic Transient-Surge Analysis Summary

Based on the results of the hydraulic transient-surge analysis, the maximum and minimum HGL envelope was found to be within the capability of a segmental concrete tunnel lining.

It was determined that with tunnel configurations that incorporated the three IF footprint volumes of 500 feet by 500 feet, 800 feet by 1,000 feet, and 800 feet by 1,500 feet, an overflow condition would occur at the C-E-2 and C-E-3 drop shafts, along with significantly higher maximum HGL conditions in the northern sections of the tunnel. It was further determined that the IF was not required to mitigate transient-surge conditions within the tunnel.

3.4 Screened Candidate Tunnel Diameter Cost Model Analysis

In accordance with the methodology and criteria described, a combined construction and operating cost evaluation was conducted comparing the 44-foot-diameter baseline tunnel against the candidate tunnel diameters sized for the maximum design flow capacities of 4,500, 6,000, and 7,500 cfs.

To establish the operating cost range, head loss was conducted for the baseline tunnel diameter of 44 feet and for each candidate tunnel diameter under two operating scenarios. These scenarios were:

1) Operating the Project at the specific design flow capacities of 4,500, 6,000, and 7,500 cfs for 24 hours per day and 365 days per year over a 100-year period to establish the upper highest operating cost value
2) Operating the project based on the monthly average flows per the DSM2 model runs presented in Appendix 5A, Section C, of the Final EIR/EIS (DWR and Reclamation 2016) over a 100-year period to establish a lower candidate operating cost value

These scenario assumptions result in a sufficiently conservative operating cost analysis range suitable for the conceptual-level analysis. The NPVs of the power costs were calculated for the head loss differences between the 44-foot baseline tunnel diameter and the candidate tunnel diameters at the design flow conditions of 4,500, 6,000, 7,500 cfs, and the monthly average flows per the DSM2 model runs over a 100-year period.

To establish the cost comparison range (power plus construction cost), the estimated cost of construction and the range of the NPV of the power costs associated with tunnel head loss for each candidate tunnel diameter (including the baseline diameter of 44 feet) were combined. The combined cost range of the baseline case tunnel diameter of 44 feet was then subtracted from the combined cost range of each candidate tunnel diameter to establish the cost difference.

Tables 12 through 14 summarize the cost difference comparison results that were developed for each candidate tunnel diameter sized for each maximum design flow capacity. Each figure lists the candidate diameter, tunnel flow velocity, calculated head loss, estimated construction cost and corresponding power cost associated with the head loss comparison to the base case tunnel diameter (44-foot). Each figure lists the design flow capacity and velocity criteria, programmatic cost adjustment, calculated head loss and head loss comparison to the base case tunnel diameter, the NPV cost range comparison and cost difference range for each tunnel diameter considered against the baseline tunnel diameter of 44 feet.

Table 12 summarizes the cost comparison results for candidate tunnel sizes selected for the design flow capacity of 4,500 cfs. The 44-foot-diameter tunnel is outside the tunnel velocity design criteria, as its flow velocity is less than 3.5 fps at the design flow capacity of 4,500 cfs but is still used as the baseline tunnel diameter.

The 31-foot-diameter tunnel, as recommended in the hydraulic analysis, has a flow velocity of 6.0 fps at 4,500 cfs and has a construction cost savings of $2,384,600,000 when compared to the 44-foot-diameter tunnel's construction cost. The 31-foot-diameter tunnel (at 4,500 cfs) has a calculated head loss of 57.9 feet, which is 45.7 feet higher than the calculated head loss associated with the 44-foot-diameter tunnel. This results in a power cost NPV increase of $427,100,000 (based on maximum operating cost). At the monthly average flows per the DSM2 model, the increase in the operating cost NPV between the 31-foot- tunnel and the 44-foot-diameter tunnel is $108,400,000. Subtracting the upper and lower NPVs for the power costs from the construction cost, the cost savings for the 31-foot-diameter tunnel versus the 44-foot-diameter tunnel ranges between $1,957,500,000 and $2,276,200,000.

Table 12. Tunnel Construction and Operating Cost Comparisons, Design Flow Capacity 4,500 cfs

Delta Conveyance Tunnel Diameter Evaluation Cost Difference Comparison

Design Flow Capacity	4,500 cfs
Baseline Case Tunnel Dia	44 ft (Note 1)
Minimum Flow Velocity Criteria	3.5 ft/s (Note 2)
Maximum Flow Velocity Criteria	8.0 ft/s (Note 3)

NPV Headloss Power Cost Delta 100 yr period @ 3.0% Discount and $.07 per kW/hr

Programatic Cost Adjustment 1.76

Candidate Tunnel Diameter (feet)	Tunnel Flow Velocity (ft/s) @ 4,500 cfs	Total System-wide Headloss (feet) @ 4,500 cfs	(Note 4) Tunel Construction Cost Savings Compared to Baseline Million ($)	Headloss Diff Compared to Baseline (feet)	NPV Max Headloss Power Cost Diff @ 4,500 cfs 24 hrs/day per year Compared to Baseline Million ($)	(Note 5) NPV Headloss Power Cost Diff @ Monthly Avg Flows per DSM2 Model Compared to Baseline Million ($)	Cost Diff Range: Total Construction and Headloss Power Cost Diff @ 4,500 cfs 24 hrs/day per year Compared to Baseline Million ($)	Cost Diff Range: Total Construction and Headloss Power Cost Diff @ Monthly Avg per SDM2 Model Results Compared to Baseline Million ($)
40	3.6	17.7	(733.7)	5.5	50.7	14.1	**(683.0)**	**(719.6)**
39	3.8	19.7	(917.1)	7.5	69.2	17.9	**(848.0)**	**(899.2)**
38	4.0	22.0	(1,100.5)	9.8	90.4	21.4	**(1,010.1)**	**(1,079.1)**
37	4.2	24.8	(1,284.1)	12.6	116.2	30.1	**(1,167.9)**	**(1,254.0)**
36	4.4	28.2	(1,467.5)	16.0	147.6	37.3	**(1,319.9)**	**(1,430.2)**
35	4.7	32.1	(1,650.9)	19.9	183.6	47.2	**(1,467.3)**	**(1,603.7)**
34	5.0	36.9	(1,834.3)	24.7	227.8	51.0	**(1,606.4)**	**(1,783.3)**
33	5.3	43.3	(2,018.0)	31.1	286.9	75.3	**(1,731.1)**	**(1,942.7)**
32	5.6	49.7	(2,201.2)	37.5	345.9	90.3	**(1,855.3)**	**(2,110.9)**
31	6.0	57.9	(2,384.6)	45.7	427.1	108.4	**(1,957.5)**	**(2,276.2)**
30	6.4	68.1	(2,568.2)	55.9	515.6	132.8	**(2,052.6)**	**(2,435.4)**
29	6.8	81.3	(2,751.6)	69.1	637.4	163.0	**(2,114.2)**	**(2,588.6)**
28	7.3	96.1	(2,935.0)	83.9	773.9	200.5	**(2,161.1)**	**(2,734.5)**
27	7.9	115.7	(3,118.5)	103.5	954.7	251.8	**(2,163.8)**	**(2,866.7)**

Note 1: Construction cost derived from estimate for 44 foot dia tunnel, central alignment, effect of diameter change extrapolated form partial estimates for 28, 36 and 40 foot diameter tunnels

Note 2: Tunnel flow velocity criteria of 3.5 ft/s established for the 4,500 cfs Firm-Design Flow Capacity per 2018 CER (dual tunnel concept)

Note 3: Tunnel flow velocity criteria of 8.0 ft/s established as the maximum flow velocity for this evaluation

Note 4: Tunnel construction cost delta comparison of the Candidate Tunnel Diameter against the Baseline Tunnel Diameter (44 feet)

Note 5: Average monthly flows from the DSM2 model runs presented in Appendix 5A, Section C, of the 2016 Final Envorinmental Impact Report/Environmental Impact Statement

A construction cost comparison was conducted for the pumping plant facilities associated with a 44-foot-diameter tunnel and a 31-foot-diameter tunnel for the design flow capacity of 4,500 cfs. This analysis was conducted to determine whether the construction cost difference associated with a deeper wet well due to the smaller-diameter tunnel developed more head loss and lower wet well operating depths as compared to a 44-foot-diameter tunnel would exceed the construction cost savings for the 31-foot-diameter tunnel, as described. This cost comparison included the same programmatic cost adjustment of 1.76 as used in Tables 12 through 14.

The results of this analysis indicate that the construction cost increase between the pumping plant facilities associated with the 31-foot- versus the 44-foot-diameter tunnel system is $37,500,000 and is not enough to offset the construction cost savings associated with the 31-foot-diameter tunnel when compared to the 44-foot-diameter tunnel. As such, the results summarized in Table 12 favor constructing a 31-foot-diameter tunnel as compared to the baseline tunnel diameter of 44 feet.

Table 13 summarizes the cost comparison results for candidate tunnel sizes selected for the design flow capacity of 6,000 cfs. The 44-foot-diameter tunnel is within the tunnel velocity design criteria and is shown in Table 4 with zero construction and operating power costs, as it is the baseline tunnel diameter.

For example, the 36-foot-diameter tunnel (which has a flow velocity of 5.9 fps at 6,000 cfs) has a construction cost savings of $1,467,500,000 when compared to the construction cost of the 44-foot-diameter tunnel. The 36-foot-diameter tunnel (at 6,000 cfs) has a calculated head loss of 50.4 feet, which is 29.8 feet higher than the calculated head loss associated with the 44-foot-diameter tunnel. This results in a power cost NPV increase of $366,500,000. At the monthly average flows per the DSM2 model, the increase in operating cost NPVs between the 36-foot- and the 44-foot-diameter tunnel is $37,300,000 (lowest operating cost difference). Subtracting the upper and lower NPVs for the power costs from the construction cost, the cost savings for the 36-foot- versus the 44-foot-diameter tunnel ranges between $1,101,000,000 and $1,430,200,000.

A construction cost comparison was conducted for the pumping plant facilities associated with a 44-foot- and a 36-foot-diameter tunnel for the design flow capacity of 6,000 cfs. This analysis was conducted to determine whether the construction cost difference associated with a deeper wet well due to the smaller-diameter tunnel developed more head loss and lower wet well operating depths as compared to a 44-foot-diameter tunnel would exceed the construction cost savings for the 36-foot-diameter tunnel, as described. This cost comparison included the same programmatic cost adjustment of 1.76 as used in Tables 12 through 14.

The results of this analysis indicate that the construction cost increase between the pumping plant facilities associated with the 31-foot- versus the 44-foot-diameter tunnel system is $119,000,000 and is not enough to offset the construction cost savings associated with the 36-foot-diameter tunnel when compared to the 44-foot-diameter tunnel. As such, the results summarized in Table 13 favor constructing a 36-foot-diameter tunnel as compared to the baseline tunnel diameter of 44 feet.

Table 13. Tunnel Construction and Operating Cost Comparisons, Design Flow Capacity 6,000 cfs

Delta Conveyance Tunnel Diameter Evaluation Cost Difference Comparison

Design Flow Capacity	6,000	cfs
Baseline Case Tunnel Dia	44	ft (Note 1)
Minimum Flow Velocity Criteria	3.5	ft/s (Note 2)
Maximum Flow Velocity Criteria	8.0	ft/s (Note 3)

NPV Headloss Power Cost Delta 100 yr period @ 3.0% Discount and $.07 per kW/hr

Programatic Cost Adjustment 1.76

Candidate Tunnel Diameter (feet)	Tunnel Flow Velocity (ft/s) @ 6,000 cfs	Total System-wide Headloss (feet) @ 6,000 cfs	(Note 4) Tunel Construction Cost Savings Compared to Baseline Million ($)	Headloss Diff Compared to Baseline (feet)	NPV Max Headloss Power Cost Diff @ 6,000 cfs 24 hrs/day per year Compared to Baseline Million ($)	(Note 5) NPV Headloss Power Cost Diff @ Monthly Avg Flows per DSM2 Model Compared to Baseline Million ($)	Cost Diff Range: Total Construction and Headloss Power Cost Diff @ 6,000 cfs 24 hrs/day per year Compared to Baseline Million ($)	Cost Diff Range: Total Construction and Headloss Power Cost Diff @ Monthly Avg per SDM2 Model Results Compared to Baseline Million ($)
46	3.6	17.4	366.8	(3.2)	(39.4)	(7.3)	327.42	359.48
45	3.8	18.9	183.4	(1.7)	(20.9)	(3.7)	162.5	179.7
44	3.9	20.6	0.0	0.0	0.0	0.0	0.0	0.0
43	4.1	22.6	(183.4)	2.0	24.6	2.7	(158.8)	(180.7)
42	4.3	24.9	(366.8)	4.3	52.9	4.2	(313.9)	(362.6)
41	4.5	27.6	(550.4)	7.0	86.1	8.6	(464.3)	(541.8)
40	4.8	30.7	(733.7)	10.1	124.2	14.1	(609.5)	(719.6)
39	5.0	34.4	(917.1)	13.8	169.7	15.8	(747.4)	(901.3)
38	5.3	38.8	(1,100.5)	18.2	223.8	21.4	(876.7)	(1,079.1)
37	5.6	44.0	(1,284.1)	23.4	287.8	30.1	(996.3)	(1,254.0)
36	5.9	50.4	(1,467.5)	29.8	366.5	37.3	(1,101.0)	(1,430.2)
35	6.2	58.8	(1,650.9)	38.2	466.1	47.2	(1,184.8)	(1,603.7)
34	6.6	64.7	(1,834.3)	44.1	542.4	51.0	(1,291.9)	(1,783.3)
33	7.0	76.1	(2,017.7)	55.6	682.6	75.3	(1,335.1)	(1,942.4)
32	7.5	86.6	(2,201.2)	66.1	811.7	90.3	(1,389.5)	(2,110.9)
31	7.9	101.0	(2,384.6)	80.4	988.8	108.4	(1,395.8)	(2,276.2)

Note 1: Construction cost derived from estimate for 44 foot dia tunnel, central alignment, effect of diameter change extrapolated form partial estimates for 28, 36 and 40 foot diameter tunnels

Note 2: Tunnel flow velocity criteria of 3.5 ft/s established for the 6,000 cfs Firm-Design Flow Capacity per 2018 CER (dual tunnel concept)

Note 3: Tunnel flow velocity criteria of 8.0 ft/s established as the maximum flow velocity for this evaluation

Note 4: Tunnel construction cost delta comparison of the Candidate Tunnel Diameter against the Baseline Tunnel Diameter (44 feet)

Note 5: Average monthly flows from the DSM2 model runs presented in Appendix 5A, Section C, of the 2016 Final Envorinmental Impact Report/Environmental Impact Statement

Table 14 summarizes the cost comparison results for candidate tunnel sizes selected for the design flow capacity of 7,500 cfs. The 44-foot-diameter tunnel is within the tunnel velocity design criteria and is shown in Table 4 with zero construction and operating power costs, as it is the baseline tunnel diameter.

For example, the 40-foot-diameter tunnel (which has a flow velocity of 6.0 fps at 7,500 cfs) has a construction cost savings of $733,700,000 when compared to the construction cost of the 44-foot-diameter tunnel. The 40-foot-diameter tunnel (at 7,500 cfs) has a calculated head loss of 42.1 feet, which is 14.4 feet higher than the calculated head loss associated with the 44-foot-diameter tunnel. This results in an increase in the power cost NPV of $221,400,000. At the monthly average flows per the DSM2 model, the increase in the operating cost NPVs between the 40-foot- versus the 44-foot-diameter tunnel is $14,100,000 (lowest operating cost difference). Subtracting the upper and lower NPVs for the power costs from the construction cost, the cost savings for the 40-foot- versus the 44-foot-diameter tunnel ranges between $512,400,000 and $719,100,000.

A construction cost comparison was conducted for the pumping plant facilities associated with a 44-foot- and a 40-foot-diameter tunnel for the design flow capacity of 7,500 cfs. This analysis was conducted to determine whether the construction cost difference associated with a deeper wet well due to the smaller-diameter tunnel developed more head loss and lower wet well operating depths as compared to a 44-foot-diameter tunnel would exceed the construction cost savings for the 40-foot-diameter tunnel, as described. This cost comparison included the same programmatic cost adjustment of 1.76 as used in Tables 12 through 14.

The results of this analysis indicate that the construction cost increase between the pumping plant facilities associated with the 40-foot versus the 44-foot-diameter tunnel system is $200,000,000 and is not enough to offset the construction cost savings associated with the 40-foot-diameter tunnel when compared to the 44-foot-diameter tunnel. As such, the results summarized in Figure 20 favor constructing a 40-foot-diameter tunnel as compared to the baseline tunnel diameter of 44 feet.

The cost analyses results for each of the maximum design capacities favor the tunnel diameters associated with the maximum velocity criteria of 6.0 fps of 31-foot diameter at 4,500 cfs, 36-foot diameter at 6,000 cfs and 40-foot diameter at 7,500 cfs when compared to the baseline tunnel diameter of 44 feet.

Table 14. Tunnel Construction and Operating Cost Comparisons, Design Flow Capacity 7,500 cfs

Delta Conveyance Tunnel Diameter Evaluation Cost Difference Comparison

Design Flow Capacity	7,500 cfs
Baseline Case Tunnel Dia	44 ft (Note 1)
Minimum Flow Velocity Criteria	4.9 ft/s (Note 2)
Maximum Flow Velocity Criteria	8.0 ft/s (Note 3)

NPV Headloss Power Cost Delta 100 yr period @ 3.0% Discount and $.07 per kW/hr

Programatic Cost Adjustment 1.76

Candidate Tunnel Diameter (feet)	Tunnel Flow Velocity (ft/s) @ 7,500 cfs	Total System-wide Headloss (feet) @ 7,500 cfs	(Note 4) Tunel Construction Cost Savings Compared to Baseline Million ($)	Headloss Diff Compared to Baseline (feet)	NPV Max Headloss Power Cost Diff @ 7,500 cfs 24 hrs/day per year Compared to Baseline Million ($)	(Note 5) NPV Headloss Power Cost Diff @ Monthly Avg Flows per DSM2 Model Compared to Baseline Million ($)	Cost Diff Range: Total Construction and Headloss Power Cost Diff @ 7,500 cfs 24 hrs/day per year Compared to Baseline Million ($)	Cost Diff Range: Total Construction and Headloss Power Cost Diff @ Monthly Avg per SDM2 Model Results Compared to Baseline Million ($)
52	3.5	14.8	1,467.5	(12.9)	(198.3)	12.5	**1,269.2**	**1,480.0**
51	3.7	15.4	1,284.1	(12.3)	(189.1)	11.5	**1,095.0**	**1,295.6**
50	3.8	16.9	1,100.5	(10.8)	(166.0)	10.2	**934.5**	**1,110.7**
49	4.0	17.4	917.1	(10.2)	158.4	9.2	**1,075.5**	**926.3**
48	4.1	19.6	733.7	(8.1)	124.5	7.5	**858.3**	**741.2**
47	4.3	20.1	550.4	(7.6)	116.8	6.6	**667.2**	**557.0**
46	4.5	23.1	366.8	(4.6)	70.2	5.0	**437.0**	**371.8**
45	4.7	25.2	183.4	(2.5)	38.4	3.7	**221.8**	**187.1**
44	4.9	27.7	0.0	0.0	0.0	0.0	**0.0**	**0.0**
43	5.2	30.5	(183.4)	2.8	43.1	2.7	**(140.3)**	**(180.7)**
42	5.4	33.8	(366.8)	6.1	93.8	4.2	**(273.0)**	**(362.6)**
41	5.7	37.6	(550.4)	10.0	152.2	8.6	**(398.2)**	**(541.8)**
40	6.0	42.1	(733.7)	14.4	221.4	14.1	**(512.4)**	**(719.6)**
39	6.3	47.4	(917.1)	19.7	302.9	15.8	**(614.3)**	**(901.3)**
38	6.6	53.6	(1,100.5)	25.9	398.2	21.4	**(702.3)**	**(1,079.1)**
37	7.0	61.0	(1,284.1)	33.3	512.0	30.1	**(772.1)**	**(1,254.0)**
36	7.4	69.8	(1,467.5)	42.1	647.2	37.3	**(820.2)**	**(1,430.2)**
35	7.8	80.0	(1,650.9)	52.4	804.1	47.2	**(846.8)**	**(1,603.7)**

Note 1: Construction cost derived from estimate for 44 foot dia tunnel, central alignment, effect of diameter change extrapolated form partial estimates for 28, 36 and 40 foot diameter tunnels

Note 2: Tunnel flow velocity criteria of 3.5 ft/s established for the 7,500 cfs Firm-Design Flow Capacity per 2018 CER (dual tunnel concept)

Note 3: Tunnel flow velocity criteria of 8.0 ft/s established as the maximum flow velocity for this evaluation

Note 4: Tunnel construction cost delta comparison of the Candidate Tunnel Diameter against the Baseline Tunnel Diameter (44 feet)

Note 5: Average monthly flows from the DSM2 model runs presented in Appendix 5A, Section C, of the 2016 Final Envorinmental Impact Report/Environmental Impact Statement

4. Conclusions and Recommendations

Based on the results of the hydraulic and capacity analysis of the tunnel diameters options evaluation as described in this TM for Project design flow capacities of 4,500, 6,000, and 7,500 cfs, the following is recommended for further development as part of the Project's conceptual design:

- The maximum flow velocity criteria should be limited to 6.0 fps for the maximum design flow capacities of 4,500, 6,000, and 7,500 cfs.
- The recommended minimum finished ID of the tunnel sections between C-E-3 and the pumping plant wet well are:
  - 31 feet for design flow capacities up to a maximum of 4,500 cfs
  - 36 feet for design flow capacities up to a maximum of 6,000 cfs
  - 40 feet for design flow capacities up to a maximum of 7,500 cfs
- The head loss associated with the minimum recommended tunnel diameters is:
  - 44.8 feet at the maximum design flow capacity of 4,500 cfs, with a 31-foot-diameter tunnel
  - 36.8 feet at the maximum design flow capacity of 6,000 cfs, with a 36-foot-diameter tunnel
  - 32.4 feet at the maximum design flow capacity of 7,500 cfs, with a 40-foot-diameter tunnel
- The IF does not provide significant surge mitigation at the maximum recommended tunnel flow velocity of 6 fps or slower. However, the IF facility may be beneficial for operating the system and further hydraulic modeling is recommended to determine if the IF should remain.

5. References

California Department of Water Resources (DWR) and Bureau of Reclamation (Reclamation). 2016. *Bay Delta Conservation Plan/California WaterFix Final Environmental Impact Report/Environmental Impact Statement*.

California Department of Water Resources (DWR) and Bureau of Reclamation (DWR and Reclamation). 2018. California WaterFix Draft Supplemental Environmental Impact Report/Environmental Impact Statement.

Wylie, Benjamin E., and Victor L. Streeter. 1993. *Fluid Transients in Systems.* Englewood Cliffs, NJ: Prentice Hall.

6. Document History and Quality Assurance

Reviewers listed have completed an internal quality review check and approval process for deliverable documents that is consistent with procedures and directives identified by the Engineering Design Manager (EDM) and the DCA.

Approval Names and Roles			
Prepared by	**Internal Quality Control review by**	**Consistency review by**	**Approved for submission by**
Tony Naimey / EDM Pumping Plant Lead	Ted Davis / EDM QC Reviewer	Gwen Buchholz / DCA Environmental Consultant	Terry Krause / EDM Project Manager
This interim document is considered preliminary and was prepared under the responsible charge of Anthony M. Naimey, California Professional Engineering License M28450.			

Note to Reader

This is an early foundational technical document. Contents therefore reflect the timeframe associated with submission of the initial and final drafts. Only minor editorial and document date revisions have been made to the current Conformed Final Draft for Administrative Draft Engineering Project Report version.