State of California
California Natural Resources Agency
DEPARTMENT OF WATER RESOURCES

CALIFORNIA AQUEDUCT
HYDRAULIC CONVEYANCE CAPACITY

December 2023

Cover photo by Renato Espinoza Torres P.E. CFM.
Aqueduct profile with limited lined freeboard.
November 14, 2017.

State of California
California Natural Resources Agency
DEPARTMENT OF WATER RESOURCES
Division of Engineering
California Aqueduct Subsidence Program

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California Aqueduct Subsidence Program
California Aqueduct Hydraulic Conveyance Capacity

ENGINEERING CERTIFICATION

This report has been prepared under my direction as the professional engineer in direct responsible charge of the work, in accordance with the provisions of the Professional Engineers Act of the State of California.

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ABBREVIATIONS AND ACRONYMS

af	acre-feet
Aqueduct	California Aqueduct
BV	Buena Vista
CASP	California Aqueduct Subsidence Program
CASS	California Aqueduct Subsidence Study
cfs	cubic feet per second
DA	Dos Amigos
DOE	Division of Engineering
DWR	California Department of Water Resources
ft/s	feet per second
HCC	hydraulic conveyance capacity
HDR	HDR Engineering
HEC	Hydrologic Engineering Center
HEC-RAS	Hydrologic Engineering Center-River Analysis System
in	inch
LiDAR	light detection and ranging
Max	maximum
Min	minimum
MP	milepost
NAD 27	North American Datum of 1927
NAVD 88	North American Vertical Datum of 1988
NGVD 29	National Geodetic Vertical Datum of 1929
Reclamation	US Bureau of Reclamation
SLFD	San Luis Field Division
SJFD	San Joaquin Field Division
SOO	Standing Operating Order
SWP	State Water Project
WSE	water surface elevation

1.0 Introduction

1.1 Purpose and Scope

The purpose of this report is to present the hydraulic conveyance capacity (HCC) of the California Aqueduct (Aqueduct), including the San Luis Canal, in its current (2023) subsided condition and as currently operated. The calculated value of the HCC depends on many factors, including the analytical method used to calculate HCC, the physical conditions represented in the hydraulic model used, and the operating criteria applied.

HCC is the calculated, long-term, sustainable, maximum steady flow rate at which water can be conveyed through an Aqueduct pool or section (such as a check), given a specified physical condition of the Aqueduct and a specified set of operational criteria. HCC is typically represented in cubic feet per second (cfs). When calculated for a pool in this report, the HCC represents the flow rate at the upstream end of the given pool.

The HCC has been calculated for two analytical scenarios:

1. Scenario 1: 2020 SOO with Coalinga Canal Special Condition
    a. Physical geometry reflecting the Aqueduct’s 2023 subsided condition.
    b. Standing Operating Order (SOO) 600.22 dated May 20, 2020 (hereinafter, SOO 2020) operating criteria, with water surface elevations corrected for 2018-2023 subsidence.
    c. Special conditions implemented to manage the operational impacts of subsidence.
    d. Special condition at check 18 to service the Coalinga Canal.
2. Scenario 2: 2020 SOO with Coalinga Canal and Coastal Branch Special Conditions
    a. Physical geometry reflecting the Aqueduct’s 2023 subsided condition.
    b. SOO 2020 operating criteria, with water surface elevations corrected for 2018-2023 subsidence.
    c. Special conditions implemented to manage the operational impacts of subsidence.
    d. Special condition at check 18 to service the Coalinga Canal.
    e. Special condition at check 22 to service the Coastal Branch.

The HCC estimates provided in this report were calculated using a detailed hydraulic model of the Aqueduct that represents its current physical features, including recent changes caused by subsidence. Details about this model, along with technical modeling approach considerations, are documented in the California Aqueduct Hydraulic Model Development Report (DWR, 2023).

Prior to any use of the results presented herein, readers must have a thorough understanding of the results, consider their specific analytical needs and assumptions, and identify criteria that are appropriate for their specific application. Typical considerations include necessary corrections to published Aqueduct water surface elevations, timescale of a given scenario, freeboard objectives, velocity limits, gate operations, turnout loading, and special conditions. The selection of appropriate operating criteria and modeling assumptions depends on the purpose of the analysis and is critical to the applicability of the results presented herein.

It is important to note that the 2020 SOO criteria were developed using a model which reflected 2018 subsided conditions in the Aqueduct. Therefore, the HCC calculations in this report are based on water surface elevation criteria derived from the 2020 SOO normal operating range, but corrected for subsidence that has occurred since the normal operating range listed in the 2020 SOO was calculated with the 2018 conditions model. The 2020 SOO normal operating range is presented in Appendix A. (in National Geodetic Vertical Datum of 1929 [NGVD 29]). Appendix B shows the corrections made at the checks for 2018-2023 subsidence, the conversion factors from NGVD 29 to North American Vertical Datum of 1988 (NAVD 88) by milepost (MP), and the corrected normal operating range elevations in NAVD 88.

The estimated capacities presented in this report do not provide a basis for evaluating the feasibility of long-term actions to address subsidence because they represent a limited set of HCC estimates computed only for specific analytical scenarios representing the current (2023) subsided condition of the Aqueduct.

1.2 Background

The Aqueduct is a key feature of the State Water Project (SWP). It is owned and operated by the California Department of Water Resources (DWR), with the exception of a federally owned portion that extends for 102 miles from the O'Neill Forebay to the federal terminus at Kettleman City. This federally owned portion—the San Luis Canal—was designed and constructed by the US Bureau of Reclamation (Reclamation); however, it is operated and maintained by DWR's San Luis Field Division (SLFD).

The Aqueduct is made up of segments, referred to as pools, bounded by either pump stations or check structures. Water travels through the pools by gravity until it is lifted by pumping plants and then continues its journey south by gravity until the next pumping plant. Relying on gravity to move the water requires that water surface profile maintain a downstream slope, while maintaining a minimum amount of lined freeboard, vertically between the water surface elevation and the top of concrete liner.

Since initiation of operations in the mid-1960s, subsidence in California's San Joaquin Valley has caused differential changes in elevation along the profile of the Aqueduct. Subsidence has degraded the elevations of the Aqueduct's embankments, canal invert, checks, and the top of concrete channel liner (top of liner). Consequently, the elevation profiles of the Aqueduct have become irregular and uneven, instead of a constant downstream slope, and certain segments even have negative slope (locations where the downstream end of a segment or pool has higher elevation than the upstream end).

Five distinct subsidence “bowls” have been identified along the alignment of the Aqueduct in the San Joaquin Valley (DWR, 2017). Figure 1-1 shows these subsidence bowls in relation to the Aqueduct extents within DWR’s SLFD, which coincides with the San Luis Canal, and within DWR’s San Joaquin Field Division (SJFD), which extends from pool 21 to the Edmonston Pumping Plant (pool 40).

The irregular Aqueduct profile of affected pools inhibits the ability of SWP operators to convey water as originally intended (DWR, 2017). Certain severely subsided areas within these subsidence bowls have resulted in “choke points” along the Aqueduct, i.e., points where subsidence has lowered the elevation of the top of liner so much that the top of liner now encroaches into the operable water surface elevation profile creating obstacles to normal operations.

In response to subsidence and other operational challenges, including water surface elevation requirements for specific turnouts or bifurcations, the operational criteria for the Aqueduct have been updated multiple times since original construction, including in 1998, 2013, and 2020. The most recent updates to the operational water surface elevation criteria are documented in the 2020 SOO.

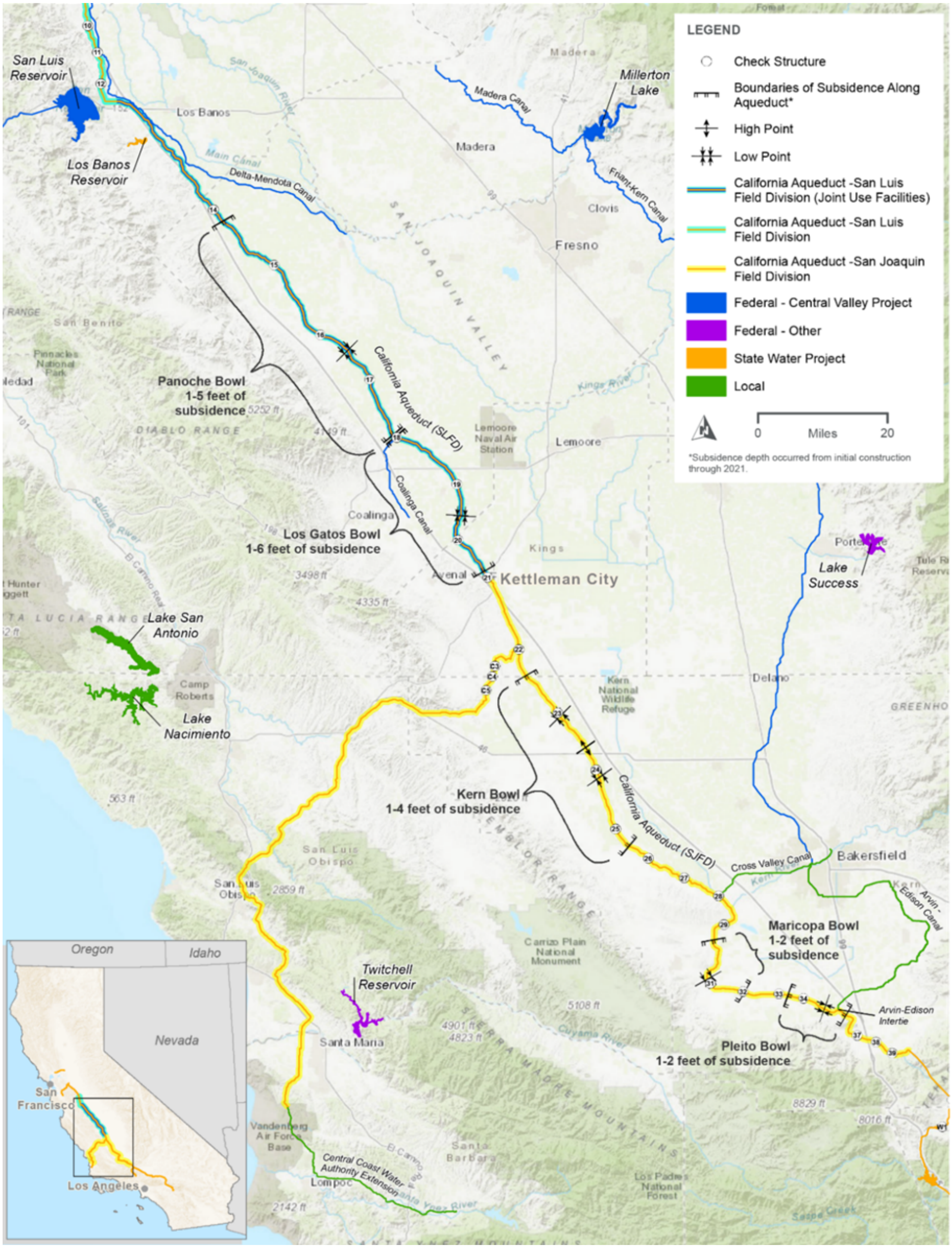

Figure 1-1. Locations of subsidence along the California Aqueduct

Periodically, DWR reevaluates the Aqueduct's ability to convey water and make deliveries within the SLFD and SJFD. The HCC estimates previously reported by DWR in the California Aqueduct Subsidence Study (2017 CASS Report) (DWR, 2017) and the California Aqueduct Subsidence Study, Supplemental Report (2019 CASS Supplemental Report) (DWR, 2019) were based on operations, subsided conditions, and calculation methods in place at the time of those reports.

The 2017 CASS Report and 2019 CASS Supplemental Report presented HCC estimates that were derived from a steady-state, "pool-by-pool" analysis which is useful to show subsidence-related effects on individual pools independent of the performance of upstream or downstream pools. The pool-by-pool methodology (described in Appendix C) is representative of limited scenarios and does not consider conservation of mass across pools. The pool-by-pool analysis includes instantaneous flow rate changes across check structures (i.e., flow rate discontinuities between pools), which is a useful mathematical modeling approach, but cannot occur in real-world operations.

Furthermore, the HCC estimates presented in the 2017 CASS Report and 2019 CASS Supplemental Report were based on a criterion of maintaining 6 inches of freeboard from top of liner, which is not consistent with either the 2020 SOO or the freeboard incorporated in the original design for the original operating criteria. Since 2019, DWR has refined the California Aqueduct Hydraulic Model and modeling methods to include greater detail about the physical condition of the Aqueduct and more precise representations of operational adjustments.

Consequently, although the estimates presented in the 2017 CASS Report and 2019 CASS Supplemental Report are informative, the following caveats are important to consider:

- The 2017 and 2019 HCC values do not represent the current (2023) operation of the subsided Aqueduct.
- The 2017 and 2019 HCC values do not provide a representative base condition for evaluating and selecting potential corrective actions to address subsidence over the long term.
- Direct comparisons cannot be made between the 2017 and 2019 results and the results herein.

2.0 Original Design Capacity, Freeboard, and Normal Operating Range

2.1 Chapter Overview

Understanding how the Aqueduct was originally design and operated, including how elevation criteria were established and implemented, is helpful in formulating credible methods to estimate the HCC of a subsided Aqueduct. This chapter discusses the Aqueduct's original design capacity, freeboard considerations, and the normal operating range, all in the context of estimating the Aqueduct's current HCC. Additional details about the original design process are provided in Appendix D.

2.2 Original Design Capacity by Pool

This report introduces the term "original design capacity" to specify the Aqueduct's original HCC when it was first built according to its design criteria and operated according to its original operating criteria. In this report, original design capacity is equal in value to the "design discharge" in the 1965 Aqueduct Design Criteria (DWR, 1965) and in the SWP Data Handbook (DWR, 2018).

Table 2-1 shows the original design capacity for pools 14-40. The original design capacity was used, by the original Aqueduct designers, to size the concrete-lined and earth embankment sections of each pool.

Table 2-1. Original design capacity by pool

Pool	Original design capacity (cfs)	Pool	Original design capacity (cfs)
14	13,100	28	5,950
15	13,100	29	5,350
16	11,800	30	5,350/5,050
17	11,800	31	5,050
18	11,800	32	4,900
19	9,350	33	4,900
20	8,350	34	4,900
21	8,350	35	4,600
22	8,100	36	4,400
23	7,300	37	4,400
24	7,150	38	4,400
25	7,150/6,350	39	4,400
26	5,950	40	4,400
27	5,950		

Source: DWR Data Handbook 2018

2.3 Calculation of Original Freeboard

This section provides an overview of considerations made as part of the original design when calculating the appropriate Aqueduct freeboard. It also discusses factors related to freeboard that are important to consider when calculating HCC under current or future subsided conditions of the Aqueduct.

The Aqueduct has two primary types of freeboard: lined and unlined. Lined freeboard is the vertical distance from the water surface elevation profile to the top of concrete liner. It guards against damage to the Aqueduct that could occur through erosion or through seepage from normal wave conditions, which are dictated by the top width of the water surface or by operational fluctuations (DWR, 1965).

Unlined freeboard is the vertical distance from the water surface to the top of the canal embankment (i.e., the sum of lined and unlined embankment above the water surface).[1] Referred to as "berm freeboard" in the Aqueduct Design Criteria (DWR, 1965), unlined freeboard provides additional protection (a factor of safety) during unplanned surges in water levels that may be caused by sudden gate closures, pumping plant failures, and/or large unplanned inflows. The design of Aqueduct freeboard depths was based on the size of the canal, expected flow rates, expected velocities in the canal, forecasted subsidence, and other expected operations such as flood flows.

The original SLFD design included a minimum lined freeboard of 3.0 ft above the original design water surface elevation, and the minimum unlined freeboard in the original design was approximately 5.5 ft in the SLFD. The original SJFD design included a minimum lined freeboard of 2.5 ft above the original design water surface elevation, and the minimum unlined freeboard in the original design was approximately 5.0 in the SJFD.

Some reaches of the Aqueduct included additional lined and/or unlined freeboard to accommodate expected subsidence (DWR, 1965). For the original Aqueduct design, the total canal depth was sized to be equal to the normal depth[2], plus the calculated appropriate lined and unlined freeboard, plus an additional height of lined and/or unlined embankment to account for the anticipated subsidence.

For a more complete description of the freeboard considerations made during the original design, refer to Appendix D and/or the Aqueduct Design Criteria (DWR, 1965).

[1] Unlined freeboard is sometimes defined from the top of the concrete liner to the top of embankment. In this report, it is defined from the top of the water surface to the top of the embankment.

[2] Normal depth is known in hydraulics as the depth at which uniform flow will occur in an open channel. For the Aqueduct, normal depth also represents the distance from the canal invert to the original design water surface elevation needed to convey the original design capacity in the original design of the Aqueduct.

2.4 Original Design Normal Operating Range

After determining the required normal depth to convey the original design capacity, a normal operating range was established for each pool. The design normal operating range was set as the original design normal water surface elevation (hereinafter, original design water surface elevation), plus and minus 1 ft. Hence the original range of operating water surface elevations had a normal vertical variability of 2 ft, from 1 ft below to 1 ft above the original design water surface elevation. This range provided operators flexibility to initiate and accommodate flow changes. To summarize:

- The design normal maximum water surface elevation refers to the water surface elevation 1 ft above the original design water surface elevation.
- The design normal minimum water surface elevation refers to the water surface elevation 1 ft below the original design water surface elevation.

Figure 2-1 provides an illustration showing the relationship between freeboard, the normal operating range, and various elevations critical to operations and hydraulic conveyance capacity.

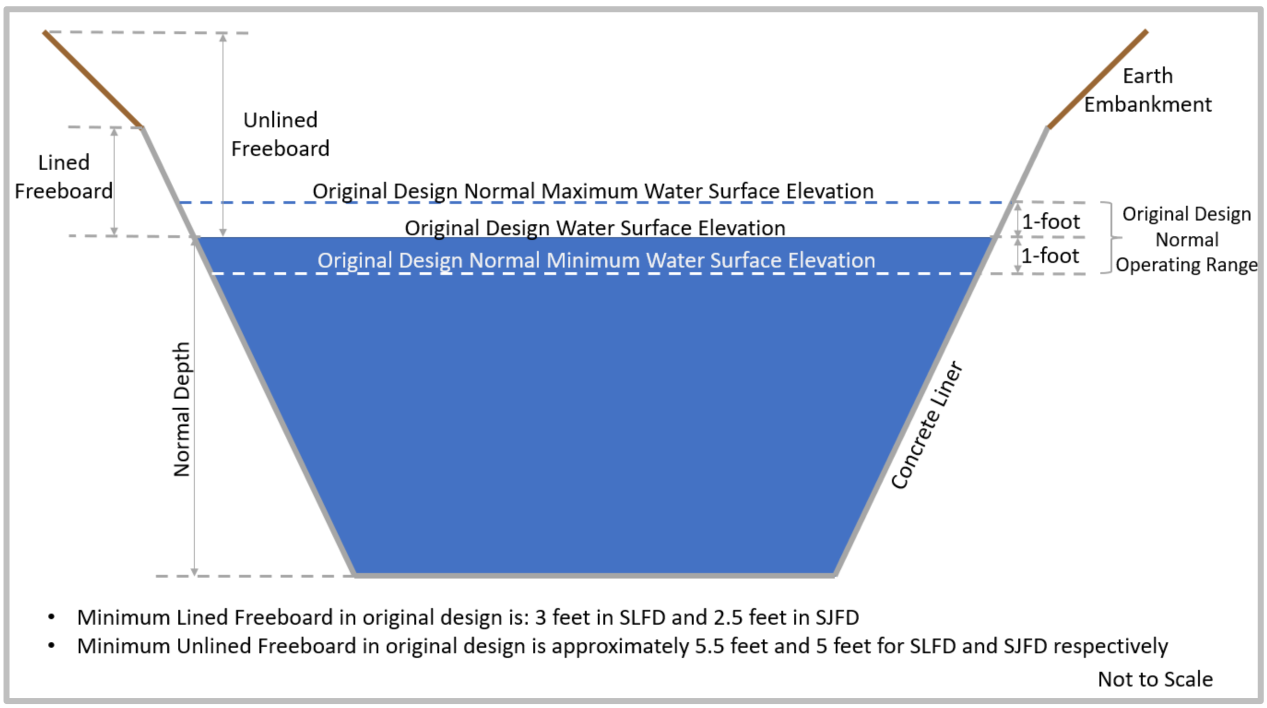

Figure 2-1. Canal cross section with water surface elevations and operating criteria

3.0 California Aqueduct Hydraulic Model Overview

3.1 Chapter Overview

This chapter provides an overview of the California Aqueduct Hydraulic Model, the HEC-RAS hydraulic model that was used to simulate the analytical scenarios described in Chapter 5.0. The overview in this chapter is limited to information that is useful in understanding the application of the model to calculate the HCC values presented herein. Please refer to the California Aqueduct Hydraulic Model Development Report (DWR, 2023) for more information about the model and detailed descriptions of the model development.

3.2 Model Version and Specifications

The California Aqueduct Hydraulic Model L21PS23-V6.2-01 was used to compute the HCC estimates presented in this report. The L21PS23 label in the model's name indicates that the model elevations are based on 2021 LiDAR updated with 2023 Precise Survey; V6.2 indicates that the model was executed using HEC-RAS software Version 6.2; and 01 indicates this is the first model version with this combination of data.

The California Aqueduct Hydraulic Model was first developed as part of the 2017 CASS Report. Since that time, DWR has performed multiple iterations of geometry and gate algorithm refinements, as well as annual elevation updates using the latest available elevation data from the Precise Survey, which is conducted by a surveying unit with DWR's Division of Operations and Maintenance. The elevation dataset is also commonly referred to as the Precise Survey dataset.

3.3 Model Parameters

3.3.1 Model Extents

The California Aqueduct Hydraulic Model extends from Dos Amigos Pumping Plant to Edmonston Pumping Plant (Figure 3-1), i.e., pools 14 through 40. The model linework is projected using the North American Datum of 1927 (NAD 27), State Plane – California IV.

3.3.2 Model Elevation Data

The California Aqueduct elevations were first put into the hydraulics model using design plan sheets and details. Since then, the model has gone through several rounds of elevation updates. The elevations of the model used for the hydraulic analysis presented herein reflect adjustments made to the model using 2021 LiDAR to represent the subsided terrain trends and slopes more accurately. Model elevations were further adjusted by the difference between the 2023 and 2021 Precise Survey datasets to account for subsidence that happened between 2021 and 2023. Linear interpolation was used between points to adjust the model features located where Precise Survey points are not available.

The California Aqueduct Hydraulic Model elevations reference the North American Vertical Datum of 1988 (NAVD 1988).

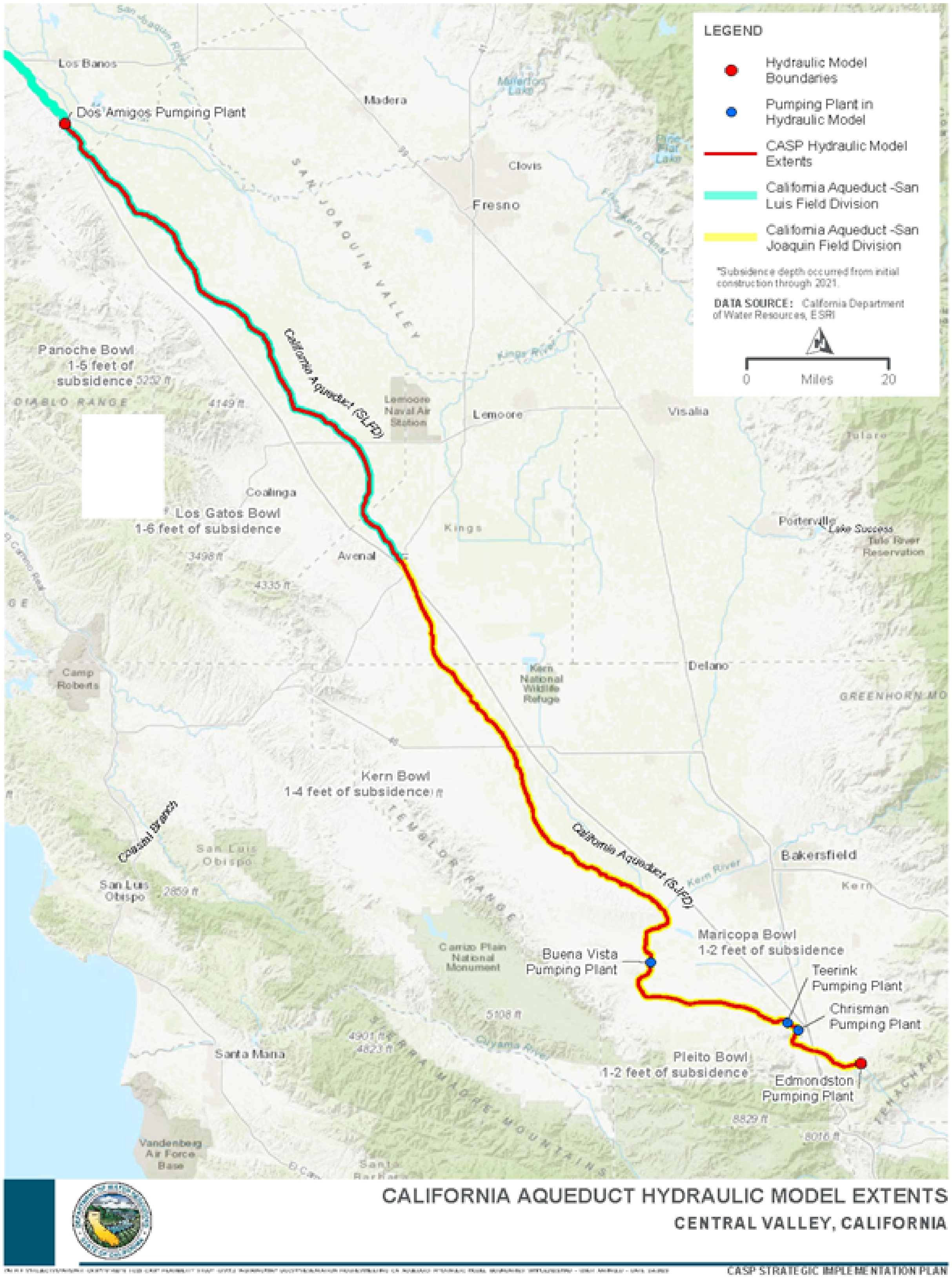

Figure 3-1. California Aqueduct Hydraulic Model extents

3.3.3 Model Cross Sections

Approximately 1,130 cross sections were used to model the Aqueduct channel. Cross sections are placed to capture variations in the channel such as widening, narrowing, bends, and changes in slope, roughness, or depth. The Aqueduct is a generally uniform channel, which allows for large distances between cross sections. Cross sectional spacing in the California Aqueduct Hydraulic Model varies from tens of feet to over a mile, with an average distance of 980 ft. Most cross sections represent the trapezoidal channel of the Aqueduct; a smaller subset of these represents the rectangular channel approach at check structures, siphons, or pump stations. Where the bottoms of overchutes are known to become submerged, the overchutes impact the upstream water surface by backing up water behind the overchute structure.

The Aqueduct is designed to be operated with water surface elevations below the top of liner; thus, the model was originally developed to represent the main channel and not overbank areas (e.g., outside of the bank stations). The cross-section bank stations were set at the concrete top of liner.

Channel base widths, side slopes, and depths (vertical distance between the base and the top of liner) for Aqueduct typical sections were developed in the model using design plan drawings. Cross-sections have been adjusted to include all the liner raises constructed as of the date of this report, including relatively recent liner raises (constructed between 2020-2021) in the vicinity of check 24 and check 25. Table 3-1 summarizes the Aqueduct segments that have been updated to account for concrete liner raises.

Table 3-1. Concrete liner raises in California Aqueduct Hydraulic Model

"Set" of Raises	Year	Spec no.	Start MP	End MP	Pool	Max Raise[3] (in)
Post Construction	1969	200C-752	132.19	132.95	17	24
	1970	200C-752	128.76	132.19	17	36
	1970	200C-752	132.19	132.95	17	24
	1970	200C-752	132.95	137.02	18	36
1982 Raises	1982	20-C0144	87.02	91.1	14	56
	1982	20-C0144	99.39	103.18	15	24
	1982	20-C0144	104.29	105.24	15	40
	1982	20-C0144	124.69	130.05	17	58
	1982	20-C0144	137	138.65	18	33
	1982	20-C0144	164.74	166.76	21	36
	1982	20-C0144	170.09	172.19	21	27
1989 Raises	1989	89-26	182.39	184.82	22	30
	1989	89-26	194.94	197.05	23	39
1996 Raises	1996	96-19	206.1	207.94	24	30
2018 Raises	2018	17-27	130.81	131.19	17	23
	2018	17-27	160.28	160.84	20	27
2021 Raises	2021	20-15	199.71	200.01	24	24
	2021	20-15	207.94	208.11	25	31
	2021	20-15	209.17	210.31	25	24

3.3.4 Effects of Subsidence

In several sections of the Aqueduct, subsidence has lowered the top of liner beyond what was originally anticipated: in some locations the resulting top of liner profile is below the original design freeboard elevation profile and, in some places, approaches the original intended operating water surface elevation profile, as shown in Figure 3-2. Figure 3-3 shows a section of the Aqueduct with very little freeboard during high flows.

[3] "Max Raise" in Table 3-1 refers to the greatest vertical distance of the liner raise in that reach.

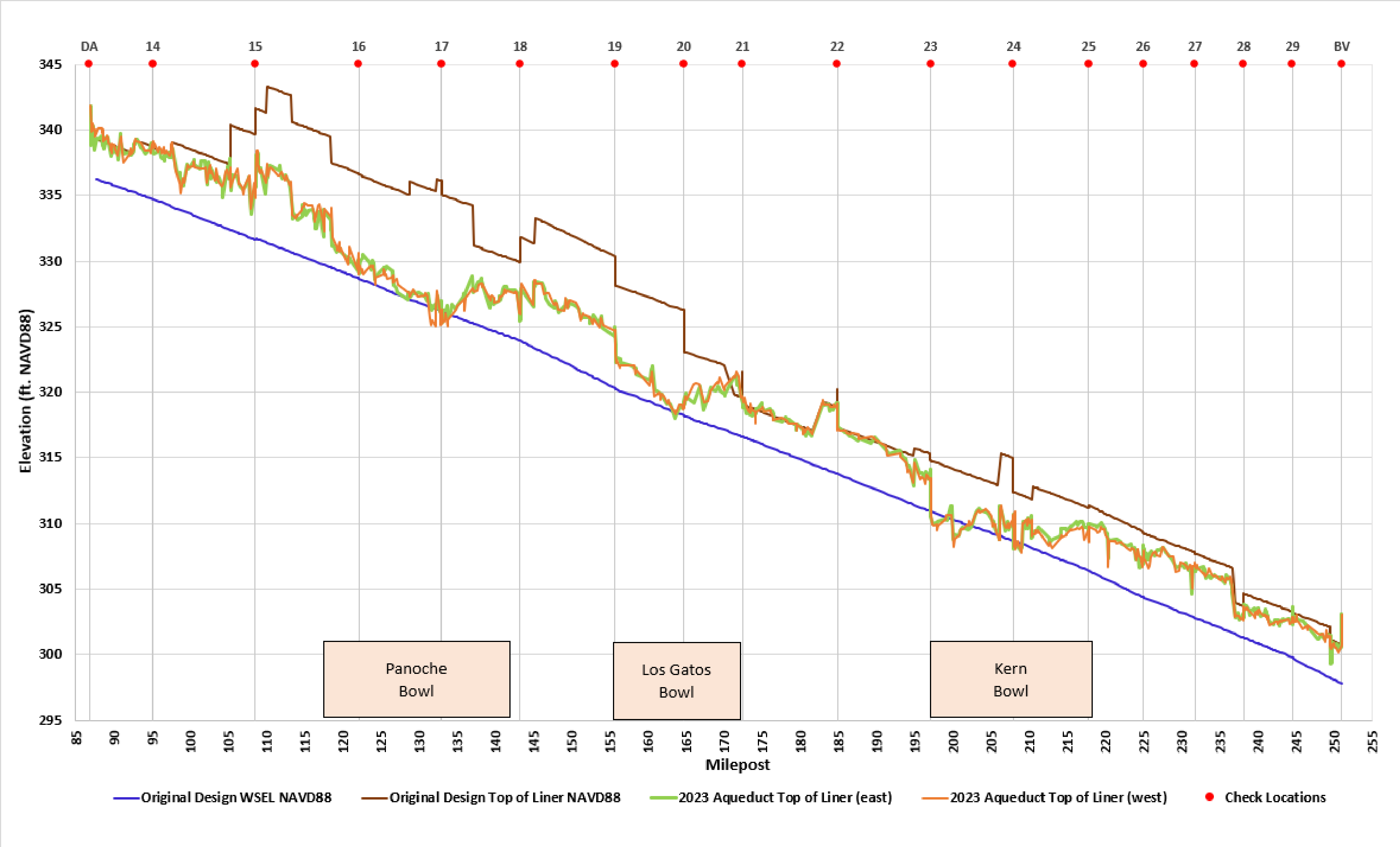

Figure 3-2. 2023 Top of liner compared to original design water service elevation (WSE) and original design top of liner

Figure 3-3. Section of the Aqueduct with reduced freeboard at high flows

3.3.5 Manning's Roughness Coefficient

In the California Aqueduct Hydraulic Model, Manning's n roughness coefficients for the main channel were set to a value of 0.02. This is consistent with recommendations made by the DWR Division of Engineering for current Aqueduct conditions (DWR, June 2017). It is also consistent with the analyses performed during the original design of the Aqueduct, which found that the roughness coefficient in Manning's equation must be increased to match the results generated by the Colebrook-White equation (for more details see Appendix D). Manning's roughness coefficients for concrete lined channels typically range from 0.011 to 0.027 depending on the smoothness of the finish (USACE, February 2016). During the original design of the San Luis Canal, a value of 0.016 was used (for more details see Appendix D). However, various calibration efforts indicated that 0.02 was more appropriate for the HEC-RAS hydraulic model of the Aqueduct (HDR, July 2018).

3.3.6 Modeled Structures

Several types of structures are represented in the California Aqueduct Hydraulic Model, as described below.

3.3.6.1 Check Structures (Inline Structures)

A total of 23 check structures (14-29, 31-34, and 37-39) are included in the California Aqueduct Hydraulic Model. Check structures are modeled as HEC-RAS inline structures with radial gate openings. Gate parameters including trunnion exponents, gate opening exponents, and head exponents were set to typical values of 0.16, 0.72 and 0.62, respectively. Radial discharge and orifice coefficient values were set to 0.7 and 0.8, respectively. These typical values are outlined in the HEC-RAS River Analysis System Hydraulic Reference Manual (USACE, February 2016).

3.3.6.2 Siphons (Lidded Cross Sections)

A total of thirteen siphons are included in the California Aqueduct Hydraulic Model, including Panoche Creek and Avenal Gap. Lidded cross sections were used to capture changes in elevation, height, and/or width of the siphons.

3.3.6.3 Pumping Plants (Pump Stations)

Four pumping plants are modeled as part of the California Aqueduct Hydraulic Model using the HEC-RAS pump station functions. The pumping plants modeled explicitly include Dos Amigos Pumping Plant (upstream end of pool 14), Buena Vista Pumping (downstream end of pool 30) Plant, Teerink Pump Plant (down stream end of pool 35), and Chrisman Pumping Plant (downstream end of pool 36). Edmonston Pumping Plant was modeled implicitly as the downstream boundary conditions for the model (downstream end of pool 40).

3.3.6.4 Lateral Structures

Lateral structures are included in the model to allow simulation of turnouts. The profile (weir elevations) of the lateral structures are not intended to be used for overtopping, so they do not accurately capture the top of embankment. Turnouts are simulated as gates. However, the gates are not modeled using weir or orifice equations. The model uses turnout gate rules and known turnout capacity to tell the model how much flow can be taken out of the system at that location.

3.3.6.5 Overcrossings

Overcrossings include overchutes, bridges, and pipelines. Hundreds of overcrossings exist between Dos Amigos Pumping Plant and Edmonston Pumping Plant. Because of subsidence, the low chord of some of these overcrossings is now encroaching below the originally intended operational water surface profile for some flow conditions. If partially submerged, these overcrossings can potentially experience uplift from buoyancy effects, can cause flow restrictions, or cause backwater effects. Twelve critical overcrossings that were deemed likely to be partially submerged during high flows (DWR, 2019) are included in the California Aqueduct Hydraulic Model using the HEC-RAS bridge modeling tool. These overcrossings are listed in Table 3-2.

Table 3-2. Modeled overcrossing

Overcrossing Type	MP	Pool
Check 17 Trunnion	132.95	17/18
Overchute	179.5	22
Overchute	196.58	23
Overchute	197.84	24
Overchute	207.18	24
Overchute	208.11	25
Overchute	209.36	25
Overchute	224.18	26
Overchute	225.05	27
Overchute	232.96	28
Pipeline	240.07	29
Overchute	246.51	30

3.4 Model Flow Data

The boundary conditions for HCC computations, including flow, downstream stages, and gate operations, are intrinsically tied to the modeling approach and objective of the simulations. The purpose of the HCC analyses described in this report is to estimate the maximum steady-state flow rate that can be achieved at specific Aqueduct locations (typically in each pool just below the upstream bounding check) while meeting a set of defined operating criteria. The process that is implemented to arrive at the various flow rates involves manipulation of downstream stages through gate settings.

The HCC results for each analytical scenario used in this report are described in Chapter 5.0.

3.5 Model Calibration and Validation

Model geometry settings such as the Manning’s n roughness coefficients and gate orifice coefficients were calibrated as part of the base model refinement during 2018. Hourly stage and flow data for the SJFD and SLFD reaches of the Aqueduct were compiled for a period of 16 days extending from September 20, 2017, to October 5, 2017. Modeled results matched observed values with an average percent error ranging from 0.96 percent at check 18 to 4.22 percent at check 21 (where the maximum error occurred). The calibration process and results are presented in the 2019 CASS Supplemental Report.

An exercise was conducted in 2019 to validate a previous version of the California Aqueduct Hydraulic Model. At that time, the model elevations reflected 2018 conditions. Simulations were executed to estimate the HCC of the Aqueduct considering a 0.5-foot freeboard criterion. Using the flows derived from the simulations, an on-the-ground flow test was performed by the SLFD and SJFD. Results from these simulations were used to make predictions about the pools and mileposts where water surface elevations were expected to exceed the target freeboard amounts. DWR and HDR personnel then made on-sight field observations to scrutinize the model predictions. The flow test was performed from July 26, 2019, to July 31, 2019.

During the validation exercise, flow percent error values were calculated at locations where observed flow data were recorded, including check structures 18 – 22 and 25. Flow percent error is defined as the difference between observed and modeled flow rates divided by the observed flow rates. Flow percent error values ranged from 0.64 percent at check 18 to 5.18 percent at check 25. The flow percent error generally increased in the downstream direction. This is due to a lack of complimentary data including lateral outflow time series values and gate opening values which would allow modeled outputs to be more refined and mimic observed data. Stage data were recorded at checks 14 – 29. Water depth percent error values were calculated at these check structures. The water depth percent error was defined as the difference between observed and modeled depth relative to observed depth. The depth percent error values ranged from 0.7 percent at check 16 to 4.9 percent at check 25. The depth percent error ranged from 0.18 ft at check 29 to 1.28 ft at check 25. Observed water surface elevations at check 25 were consistently under the pool absolute minimum elevation of 304.9 ft NGVD 29 (307.9 ft in NAVD 88). However, this is acceptable because the model objective is to keep stages, i.e., depths, within a normal operating range, not at a specific target depth, similar to normal Aqueduct operations.

In general, the calibration and validation exercises have shown that the model produces reasonable results. These exercises were performed in 2018 and 2019, respectively, and since that time, subsidence has continued to alter Aqueduct elevations and modeling methods have been further refined, as described in Chapter 5.0.

4.0 Approach for Calculating Hydraulic Conveyance Capacity

4.1 Chapter Overview

This chapter provides an overview of the approach used to calculate HCC. It describes what an analytical scenario is, and the procedural steps that were used to estimate HCC.

4.2 Role of Operating Criteria in Analytical Scenarios

The appropriate modeling approach depends on the modeling objective. Here, the objective is to provide HCC estimates for specific analytical scenarios.

An analytical scenario is a combination of:

- Physical condition/geometry of the pool(s), including the subsidence effects on the Aqueduct's elevations. For the HCC estimates reported herein, the Aqueduct is modeled to represent its 2023 subsided condition.
- Operating criteria, including freeboard requirements, velocity restrictions, special conditions, the normal operating range.

In all the scenarios investigated herein, the physical condition of the Aqueduct (the 2023 subsided condition) is held constant. The scenarios vary in the operating criteria applied:

- Scenario 1: 2020 SOO with Coalinga Special Condition includes these conditions:
    - Physical condition/geometry: The Aqueduct's current subsided condition, as of 2023.
    - Operations criteria:
        - SOO 2020 operating criteria, with water surface elevations corrected for 2018-2023 subsidence.
        - General special conditions implemented to manage the operational impacts of subsidence.
        - Special Condition at check 18 to service the Coalinga Canal.
- Scenario 2: 2020 SOO with Coalinga Canal and Coastal Branch Special Conditions includes these conditions:

- Physical condition/geometry: The Aqueduct's current subsided condition, as of 2023.
- Operations criteria:
    - SOO 2020 operating criteria, with water surface elevations corrected for 2018-2023 subsidence.
    - General special conditions implemented to manage the operational impacts of subsidence.
    - Special Condition at check 18 to service the Coalinga Canal.
    - Special Condition at check 22 to service the Coastal Branch.

The operating criteria used to perform a simulation are fundamental elements of the HCC analyses. They help define the various analytical scenarios and are inextricably tied to the results. Given the subsided condition of the Aqueduct and the complex relationship between various facilities (e.g., pools, checks, turnouts, pumping plants), it is important to define the analytical scenario explicitly, including its operating criteria, as well as the modeling objective, before selecting the approach for estimating the HCC of a pool.

Thus, the intended use of the estimated HCC informs the operating criteria to be specified, which will, in turn, have a strong influence on the results and how the results may be interpreted and applied. Operating criteria include parameters such as:

- Lined freeboard constraints.
- Unlined freeboard constraints.
- Freeboard constraints at overcrossings such as checks, bridges, overchutes, and pipes.
- Velocity constraints.
- Special Conditions.
- Normal operating range.

Given the subsided condition of the Aqueduct, criteria such as the normal operating range may be difficult to define or may require modifications. Also, analysis of future scenarios and scenarios with greater freeboard requirements have shown that, at times, conflicts occur between criteria. If a conflict occurs between two criteria (for example freeboard and minimum water surface elevation), a decision must be made about which criterion to prioritize during the model simulation while trying to find a feasible solution; the appropriate decision will depend on the intended use of the modeling results.

4.3 Hydraulic Conveyance Capacity Computation Procedure

4.3.1 Model Simulation Process

A systematic approach for evaluating the HCC of Aqueduct pools was developed using HEC-RAS hydraulic structure rules capability algorithms; this approach allows for consideration of the Aqueduct as an interconnected system. The rules control the model gates (checks), lateral structures (turnouts), and pump stations (pumping plants). The modeling process consists of a long duration unsteady simulation during which algorithms evaluate the results at every timestep based on defined operational criteria.

Typically, the goal of each simulation is to maximize the flow in each pool. Criteria are set to meet specified lined freeboard at each pool and to operate within the normal operating range[4]. To maximize the HCC, target water surface elevations are set near the normal minimum water surface elevation at the downstream end of each pool. Based on the results of a simulation, the model automatically and systematically adjusts gate openings, turnout flow rates, and/or pumping plant flow rates to arrive at a solution.

The model implements the following process:

1. At the start of the simulation, the downstream boundary of each reach (i.e., each segment of the Aqueduct between two pumping plants) is set to specified water surface elevations (typically the average between the normal max and normal min water surface elevations per the 2020 SOO corrected for 2018-2023 subsidence), all check structure gates are open, and all turnouts are off.
2. The Dos Amigos Pumping Plant at the upstream end of the model supplies a flow rate of 13,100 cfs (original design capacity of pool 14) into the system at pool 14. Intermediate pumping plants (Buena Vista, Teerink, and Chrisman) convey flow at the end of their relative upstream reaches to the next reach downstream. For instance, the Buena Vista Pumping Plant transfers the flow from pool 30 to pool 31.
3. After the initial time-step, and each subsequent time-step, the model checks water surface elevations at every cross section (in all pools). The large flow rate at the beginning of the simulation will start to exceed the targeted operational criteria in pools as it moves downstream.
4. To meet the intended operational criteria, the model first attempts to adjust the flow rates in the system by engaging turnouts. Turnouts are turned on and ramped up in pools that have a simulated freeboard deficiency (less than the user-defined freeboard criteria). Turnouts are engaged starting with the most downstream turnout in a pool, sequentially moving upstream as necessary. As turnouts divert flow, the

[4] The range of water surface elevations between the normal maximum water surface elevation and normal minimum water surface elevation

modeled flow reaching downstream pools will decrease. The reduced flow rate will result in a reduction in water surface elevations.

5. If all the turnouts in a freeboard-deficient pool have been engaged at maximum capacity and freeboard deficiencies still exist in that pool, the algorithm begins to turn on turnouts in pools upstream of the freeboard-deficient pool. Turnouts in the pools above the freeboard-deficient pool are also engaged sequentially, from downstream to upstream.
6. If water surface elevations cannot be regulated to meet the preset freeboard criteria with turnouts alone, then the algorithm begins to decrease the upstream flow rate at Dos Amigos Pumping Plant.
7. In some locations of the Aqueduct, water surface elevations need to be adjusted upward to meet minimum water surface elevation criteria. Choke points can limit the system's ability to convey the flow rates needed through downstream pools to sustain water surface elevations within the normal operating range. In these pools, gates are used to check up water levels and maintain water surface elevations at or above the preset minimum water surface elevation criteria.
8. The model evaluates the results of every timestep. If necessary, based on the preset criteria, the model adjusts check gate openings, turnout flows, and finally, the pumping plant flow (if needed) until conditions that meet the preset criteria are met. Eventually, when the modeled hydraulic conditions within all the pools satisfy the specified criteria, equilibrium is achieved.

If a conflict occurs between two criteria (for example freeboard and minimum water surface elevation), a decision must be made about which criterion to prioritize during the model simulation. Criteria prioritization will depend on the intended use or application of the analyses. This was not the case in any of the scenarios presented in this report. Although additional subsidence may lead to such scenarios in the future.

It is important to note that estimates of HCC can vary depending on the turnout loading[5]. A particular pool (e.g., pool X1) can have a limited HCC when looking at the system as a whole. However, in scenarios where large amounts of flow (in the range of 20 percent to 40 percent of the total flow) are being diverted from the pool directly downstream (e.g., pool X2), the HCC of pool X1 can experience significant increases in capacity (in the range of 10 percent to 40 percent more flow) while still meeting freeboard criteria. This happens because the additional flow diverted from pool X2 helps produce a lower water surface elevation profile, which ultimately leads to less backwater

[5] Turnout loading refers to the configuration of engaged turnouts (i.e., the number of flowing turnouts, their location along a pool, and their flow rate). When calculating HCC, variations in the configuration of engaged turnouts can lead to differences in HCC estimates for a particular pool.

resistance for flows through pool X1. This phenomenon particularly affects significantly subsided pools, such as pool 20 and pool 24 under the 2023 conditions.

4.3.2 Normal Operating Range Subsidence Adjustments

During the first iteration of calculating the HCC for this report, the normal operating range elevations listed in the 2020 SOO were used to define the modeled target water surface elevations upstream of check structures. The 2020 SOO lists a normal minimum water surface elevation above check 24, approximately at MP 207.93, as 303.9 ft (NGVD 29) (306.9 ft in NAVD 88). However, the lowest point in the pool 24 liner has an elevation of 305.25 ft (NGVD 29) (308.25 in NAVD 88), approximately at MP 200.02. Maintaining 1 ft of freeboard from the top of liner yields a flat slope in pool 24 (approximately 1.4 x 10-5), which, in turn, results in an HCC of less than 2,000 cfs. Inquiries into the flow rates being conveyed through pool 24 during the summer of 2023 showed that the HCC of pool 24 is closer to 4,000 cfs.

Further investigation showed that check operations were not actually based on the elevations listed in the 2020 SOO, but rather on the relative depths these elevations produced at the time they were set in 2018. At the time the 2020 SOO normal operating range elevations were calculated, they produced a specific depth at the stilling wells above the check structures, including check 24. Stilling wells calculate water surface elevations using pressure based on depth. However, because subsidence has continued since the analyses in 2018 to establish the 2020 SOO elevations, and the stilling wells have subsided with the local conditions and their elevations have not been adjusted for subsidence, operators are actually operating at the same relative depth as when the 2020 SOO analysis was performed, rather than at the published elevations listed in the 2020 SOO.

Therefore, the normal operating range elevations actually being used to operate the Aqueduct are equal to the 2020 SOO elevations minus the subsidence that has occurred at each stilling well between 2018 and 2023; this subtraction is referred to in this report as “corrections” to the 2020 SOO. The normal operating range elevations corrected for 2018-2023 subsidence are presented in Appendix B (in NGVD 29).

Note that, whereas the 2020 SOO refers to NGVD 29, the California Aqueduct Hydraulics Model water surface elevations refer to NAVD 88. The conversion factors used to convert water surface elevations from NGVD 29 to NAVD 88 are shown in Appendix B. The normal operating range elevations shown in Appendix B were used to calculate all the HCCs presented in this report.

4.3.3 Modeling and Operation Constraints

Scenario simulations include a modeling constraint that gates must have a minimum opening of 1 ft. This minimum opening constraint prevents check structure gates from closing all the way, which can create simulation instabilities.

The simulations also honor an operational constraint that requires gates to remain submerged by at least 1 ft. This is a field practice to help measure flow through checks and to prevent the wind from potentially damaging the gates. The model limits the gate opening and closing rates to 1 ft per minute, consistent with the actual speed of gates at check structures. Check 17 is an exception; there has been so much subsidence at this check structure that its gates are no longer operated as originally intended–DWR keeps the gates at check 17 fully open year-round. Therefore, check 17 gates were modeled fully open for all scenarios analyzed herein.

4.3.4 Special Conditions

When applicable, scenarios must also include constraints in consideration of special conditions. Special conditions are field constraints (i.e., orders) operators must abide by to help with the functionality of a particular facility or to give operators additional flexibility in a constrained part of the Aqueduct.

There are special conditions in several locations along the Aqueduct. Each special condition is set for a unique reason. Typically, special conditions are requirements for water surface elevations to be maintained above or below a specific value. Their purpose is typically to maintain service to a specific turnout or for structural safety reasons.

One example is the special condition currently implemented at the Coalinga Canal diversion at approximately check 18. The water surface elevation at milepost (MP) 146.13 must be kept at least 2.5 ft higher than the 2020 SOO normal min water surface elevation [above elevation 320.5 ft (NGVD 29) (323.4 in NAVD 88] to prevent the pumps in the Pleasant Valley Pumping Plant from losing suction. A similar special condition is defined for the Coastal Branch. Many other special conditions have been established along the SLFD and SJFD. When calculating the HCC, one must check whether the appropriate special conditions are being applied for the given scenario.

5.0 Hydraulic Conveyance Capacities for 2023 Conditions

5.1 Chapter Overview

This chapter presents the estimated HCC, reported by pool, for two scenarios. These HCC estimates were computed as if the Aqueduct were operated in steady state under 2023 conditions. The two scenarios are:

1. Scenario 1: 2020 SOO with Coalinga Canal Special Condition
    a. Physical geometry reflecting the Aqueduct’s 2023 subsided condition.
    b. SOO 2020 operating criteria, with water surface elevations corrected for 2018-2023 subsidence.
    c. General special conditions implemented to manage the operational impacts of subsidence.
    d. Special Condition at check 18 to service the Coalinga Canal.
2. Scenario 2: 2020 SOO with Coalinga Canal and Coastal Branch Special Conditions
    a. Physical geometry reflecting the Aqueduct’s 2023 subsided condition.
    b. SOO 2020 operating criteria, with water surface elevations corrected for 2018-2023 subsidence.
    c. General special conditions implemented to manage the operational impacts of subsidence.
    d. Special Condition at check 18 to service the Coalinga Canal.
    e. Special Condition at check 22 to service the Coastal Branch.

Additional information is provided about the special conditions in Section 5.2.2.

5.2 Analytical Criteria Used for Hydraulic Conveyance Capacity Scenarios

5.2.1 Operating Criteria Common to Both Scenarios

The general operating criteria common to both analytical scenarios — Scenario 1: 2020 SOO with Coalinga Canal Special Condition and Scenario 2: 2020 SOO with Coalinga Canal and Coastal Branch Special Conditions — are summarized in Table 5-1.

Table 5-1. Operating criteria used in both scenarios to calculate HCC

Operating Criterion	Original Design Value	Scenario 1	Scenario 2
Lined freeboard	SLFD: 3.0 ft SJFD: 2.5 ft	1 ft	1 ft
Unlined freeboard	SLFD: 5.5 ft SJFD: 5.0 ft	N/C	N/C
Freeboard at checks	0.3 ft	N/C	N/C
Freeboard at overcrossings	Varies	N/C	N/C
Max velocity in the unreinforced concrete liner trapezoidal sections of the canal	8 ft per second	8 ft per second	8 ft per second
Target WSE at downstream end of pools, above checks and at pumping plant forebays	Original Normal Operating Range	2020 SOO Normal Operating Range*, except for special condition in pool 18	2020 SOO Normal Operating Range*, except for special conditions in pools 18 and 22

* Adjusted for subsidence as described in Section 4.3.2

5.2.2 Special Conditions and Constraints

The special conditions and constraints used in Scenario 1: 2020 SOO with Coalinga Canal Special Condition and Scenario 2: 2020 SOO with Coalinga Canal and Coastal Branch Special Conditions are summarized in Table 5-2.

Table 5-2. Summary of special conditions and constraints

No.	Description	Type	In Scenario 1	In Scenario 2
1	Max 3.8-ft submergence of check 17 trunnion deck	special condition	yes	yes
2	Max 2-ft overcrossing submergence	special condition	yes	yes
3	Max 0.75-ft submergence overchute at MP 197.84	special condition	yes	yes
4	Max 1.3-ft submergence overchute at MP 208.11	special condition	yes	yes
5	Check 17 locked at 24 ft opening	special condition	yes	yes
6	1 ft gate submergence	constraint	yes	yes
7	1 ft min gate opening	constraint	yes	yes
8	320.5 ft* (323.4 ft in NAVD 88) min water surface elevation upstream of check 18 to allow flow into the Coalinga Canal and prevent the pumps in the Pleasant Valley Pumping Plant from losing suction; this condition is implemented year-round	special condition	yes	yes
9	311.9 ft** (314.9 in NAVD 88) min water surface elevation upstream of check 22 to allow flow into the Coastal Branch and facilitate algae and weed management; this condition is generally implemented from spring through fall each year	special condition	no	yes

* 320.5 ft (NGVD 29) is water surface elevation not corrected for 2018-2023 subsidence; when corrected, the water surface elevation value is 319.98 ft (NGVD 29) (322.88 ft in NAVD 88).

** 311.9 ft (NGVD 29) is water surface elevation not corrected for 2018-2023 subsidence; when corrected, the water surface elevation value is 311.97 (NGVD 29) (314.97 in NAVD 88).

5.2.3 Description of Scenario 1: 2020 SOO with Coalinga Canal Special Condition

Scenario 1: 2020 SOO with Coalinga Canal Special Condition provides HCC estimates while implementing the operating criteria described in the 2020 SOO (adjusted for subsidence), with exception for special conditions. This scenario is representative of

Aqueduct year-round conditions, except when seasonal or temporary special conditions are in place.

Some special conditions are applied in the field to mitigate for the subsidence impacts to HCC. The special conditions included in this scenario to mitigate for the subsidence impacts to HCC are presented in Table 5-2 (items 1-5). These special conditions are in place year-round, with no exceptions.

This scenario also includes general constraints, as listed in Table 5-2 (items 6 & 7), that apply to all operating checks. Check 17 is the exception because it has been rendered inoperable by subsidence. The 1 ft gate submergence rule is a field constrained that is implemented to measure flows and protect gates from wind gusts. The 1 ft min gate opening rule is a modeling constrain that is implemented for model stability. This rule doesn’t impact the modeling results because the gates generally need to be mostly open to increase the HCC.

The Coalinga Canal Special Condition at check 18, listed in Table 5-2 (item 8), keeps the water surface above a specified elevation to allow flow into the Coalinga Canal and to prevent the pumps in the Pleasant Valley Pumping Plant from losing suction and/or starting to cavitate. The SJFD indicated that the Coalinga Canal Special Condition is in place year-round. To apply the Coalinga Canal special condition, the model uses the check 18 gates to prop the minimum water surface elevation just upstream of check 18 to an elevation of 322.88 ft (NAVD 88); approximately 2.4 ft higher than the 2020 SOO normal minimum water surface elevation. More information about the water surface elevations for special conditions used in Scenario 1 is provided in Table 5-3.

Table 5-3. Water surface elevation (WSE) values for special conditions used to compute hydraulic conveyance capacity for Scenario 1

Location	Referenced to 2020 SOO WSE	Corrected for 2018-2023 subsidence
Check 18 w/ Coalinga Canal Special Condition WSE, ft NAVD 88 (NGVD 29)	323.4 (320.5)	322.88 (319.98)
Check 22 normal min WSE, ft NAVD 88 (NGVD 29)	313.8 (310.8)	313.87 (310.87)

5.2.4 Description of Scenario 2: 2020 SOO with Coalinga Canal and Coastal Branch Special Conditions

Scenario 2: 2020 SOO with Coalinga Canal and Coastal Branch Special Conditions is like Scenario 1, but it includes an additional special condition. The additional special condition is implemented to help with functionality when operating the Coastal Branch of the Aqueduct near check 22.

As noted in Section 5.2.3, the Coalinga Canal special condition is set year-round to prevent the Pleasant Valley Pumping Plant from losing suction and causing the pumps to cavitate. The Coastal Branch special condition, however, is seasonal and it is set to help with algae and weed management at certain times of year. The Coastal Branch special condition typically becomes effective each year sometime in the April to June timeframe; and stays in place typically through sometime in the September to November timeframe.

The modeled Coalinga Canal and Coastal Branch Special Conditions force a higher water surface elevation at check 18 (same as Scenario 2) but also, force a water surface elevation at check 22 that is 1.1 ft higher than the 2020 SOO normal minimum water surface elevation. (Note: Check 22 has had a 0.07-foot bounce back since 2018.) The water surface elevations for special conditions used in Scenario 2 are provided in Table 5-4.

Table 5-4. Water surface elevation (WSE) values for special conditions used to compute hydraulic conveyance capacity for Scenario 2

Location	Referenced to 2020 SOO WSE	Corrected for 2018-2023 subsidence
Check 18 Coalinga Canal Special Condition min WSE, ft NAVD 88 (NGVD 29)	323.4 (320.5)	322.88 (319.98)
Check 22 Coastal Branch Special Condition min WSE, ft NAVD 88 (NGVD 29)	314.9 (311.9)	314.97 (311.97)

5.3 Simulation Results for Scenario 1: 2020 SOO with Coalinga Canal Special Condition

For Scenario 1: 2020 SOO with Coalinga Canal Special Condition, model simulations show three distinct choke points: check 17, pool 20, and pool 24. Check 17 flow is restricted at the trunnion deck. Pool 20 and pool 24 are restricted by the top of concrete liner. The check 17 restriction propagates all the way upstream to pool 14. The pool 20 restriction propagates all the way upstream to pool 18. The restricted pools above check 17 and pool 20 must make releases (or convey less flow) to accommodate the low capacity of each choke point. The low capacity in pool 20 also propagates downstream to pools 21, 22, and 23. The pool 24 restriction propagates downstream, limiting the amount of flow that can reach all the way down to pool 40.

Figure 5-1 shows the Scenario 1: 2020 SOO with Coalinga Canal Special Condition steady state HCC profile compared to the original design capacity. Note that the Model Output HCC shows the magnitude and locations of turnout flows (turnout loading) that are needed to accommodate the calculated steady state HCC at the upstream end of each pool. This indicates that more than 5,000 cfs would have to be delivered between pool 14 and pool 18 to get such high HCCs in the SJFD pools. The results presented herein are approximations limited by assumptions applied due to data gaps and variability in field conditions.

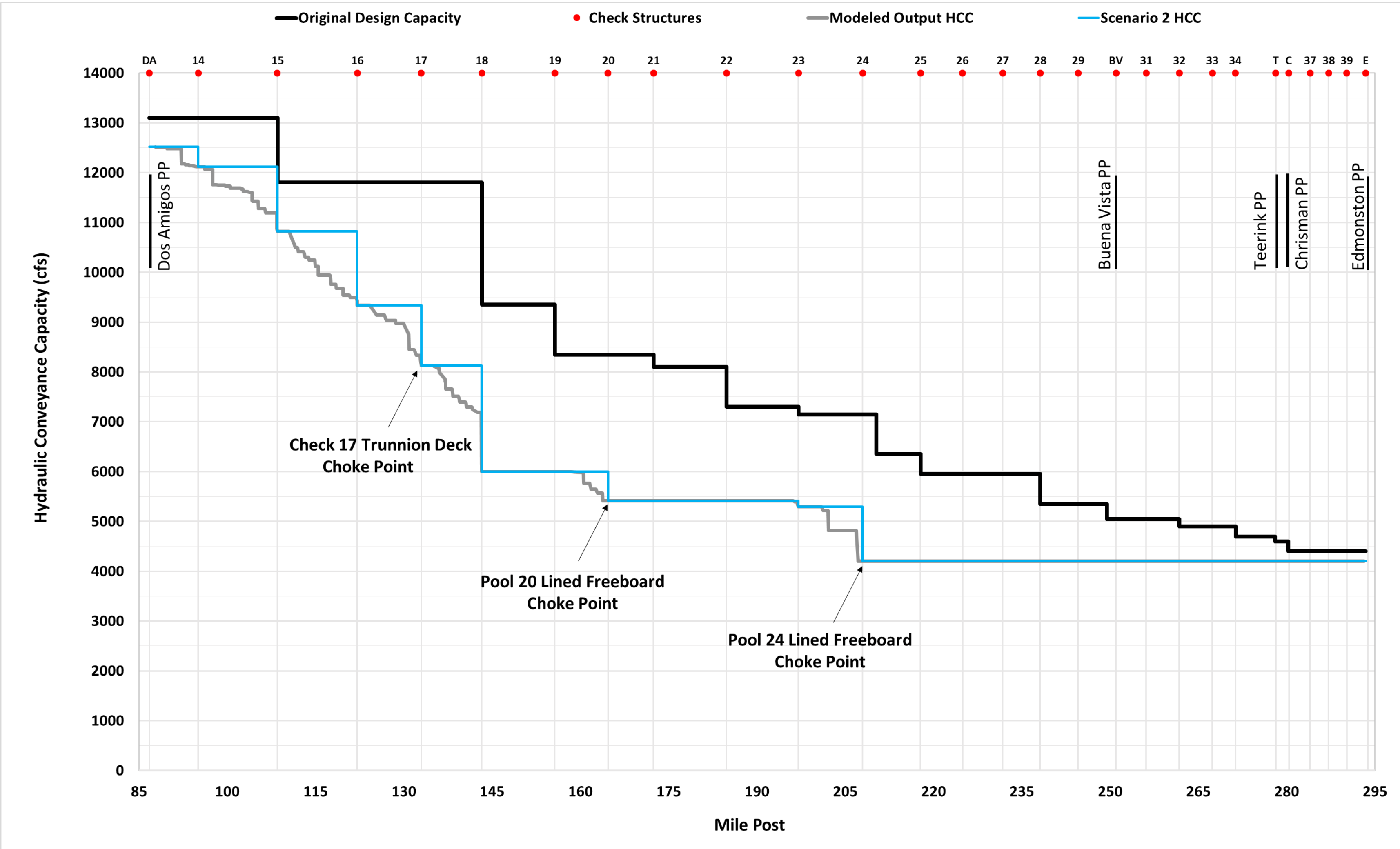

Figure 5-1. 2023 HCC profile for Scenario 1: 2020 SOO with Coalinga Canal Special Condition

5.4 Simulation Results for Scenario 2: 2020 SOO with Coalinga Canal and Coastal Branch Special Conditions

For Scenario 2: 2020 SOO with Coalinga Canal and Coastal Branch Special Conditions, model simulations show three distinct choke points: check 17, pool 20, and pool 24. check 17 flow is restricted at the trunnion deck. Pool 20 and pool 24 are restricted by the top of concrete liner. The check 17 restriction propagates all the way upstream to pool 14. The pool 20 restriction propagates upstream to pool 18. The restricted pools above check 17 and pool 20 must make releases (or convey less flow) to accommodate the low capacity of each choke point. The low capacity in pool 20, which is exacerbated by the Coastal Branch special condition, also propagates downstream to pools 21, 22, and 23. The pool 24 restriction propagates downstream, limiting the amount of flow that can reach pool 36.

Capacities downstream of pool 23 are greater than in Scenario 1: 2020 SOO with Coalinga Canal Special Condition because flows through pool 20 significantly less in Scenario 2. Therefore, flows can stay higher through pool 24 without creating a backwater effect that would lead to a freeboard violation in pool 20. Inversely, the flows can be increased through pool 24 for Scenario 1, but that would lead to reduced flows through pool 20. Figure 5-2 shows the Scenario 2: 2020 SOO with Coalinga Canal and Coastal Branch Special Conditions HCC profile compared to the original design capacity. As for Scenario 1, the Model Output HCC shows the magnitude and locations of turnout flows (turnout loading) that are needed to accommodate the calculated steady state HCC at the upstream end of each pool. This indicates that more than 5,000 cfs would have to be delivered between pool 14 and pool 18 to get such high HCCs in the SJFD pools. The results presented herein are approximations limited by assumptions applied due to data gaps and variability in field conditions.

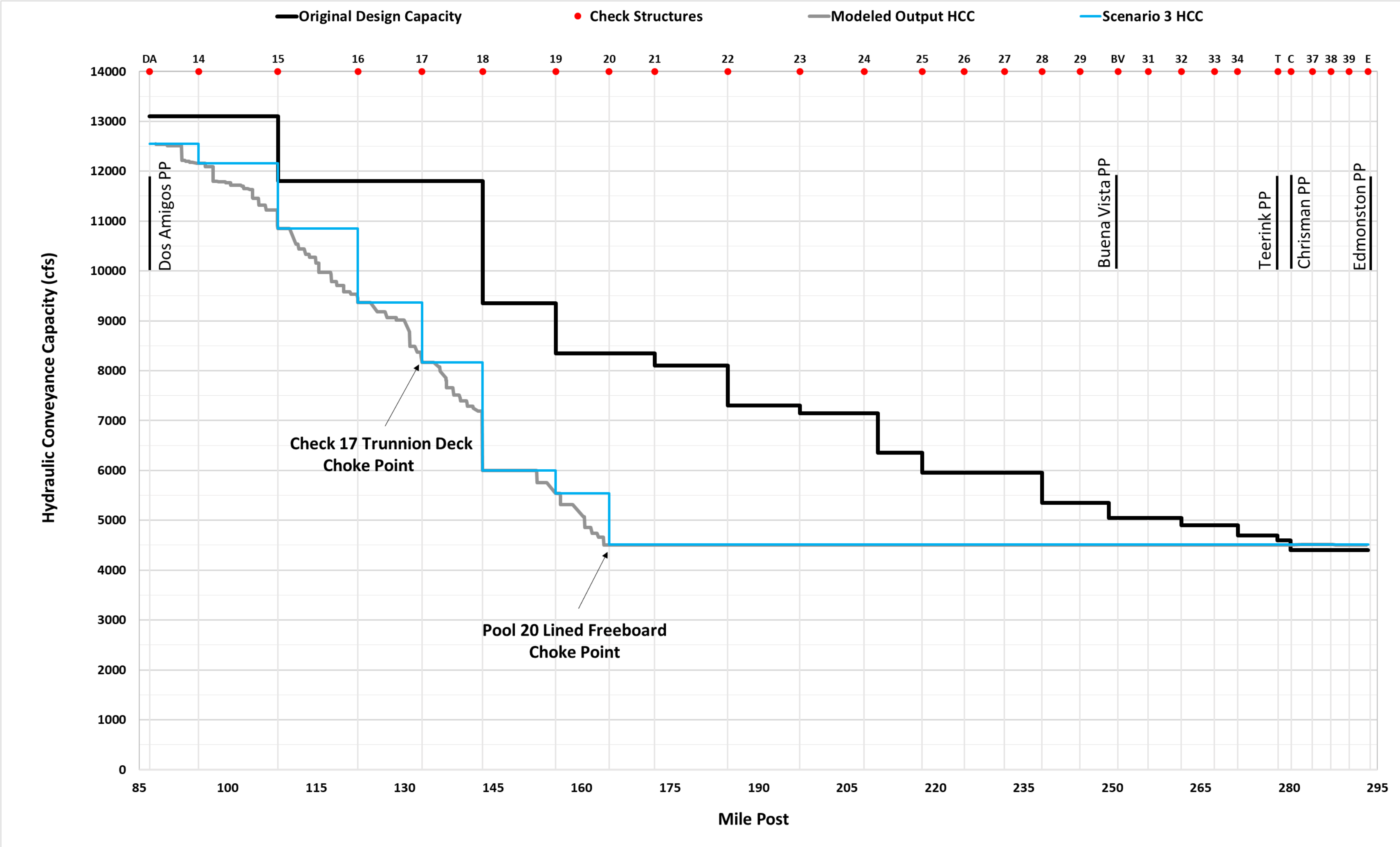

Figure 5-2. 2023 HCC Profile for Scenario 2: 2020 SOO with Coalinga Canal and Coastal Branch Special Conditions

Compiling the results of both analytical scenarios, Table 5-5 provides a summary of the estimated HCC for pools 14-40 for Scenarios 1 & 2. Scenario 1 likely represents the 2023 HCC between the months of November and May. Scenario 2 likely represents the 2023 HCC from late spring through fall, depending on algae and weed conditions.

The Aqueduct is a complex system that can be operated in many ways, including creating conditions which are favorable to specific pools. For example, by lowering the water surface elevations in a downstream pool, and “stacking” the water surface elevations in an upstream pool, this can create a short pulse (usually in a timescale of hours) of higher flows to increase the HCC through a specific pool. However, these conditions do not allow for a long-term, sustainable, flow rate. And averaged over a timeframe in the scale of a month, these short-term flow rates would be significantly lower. Therefore, they may not be useful in long-term planning.

The values presented in Table 5-5 represent current estimates of the maximum steady flow rates that may be sustained long-term (days or weeks) given adequate pumping and turnout loading. The results presented herein are approximations limited by assumptions applied due to data gaps and variability in field conditions.

Table 5-5. Hydraulic conveyance capacity estimates for pools 14-40 under both analytical scenarios

Pool	Original Design Capacity (cfs)	Scenario 1: 2020 SOO with Coalinga Canal Special Condition (cfs)	Scenario 2: 2020 SOO with Coalinga Canal and Coastal Branch Special Conditions (cfs)
14	13,100	12,520	12,550
15	13,100	12,120	12,160
16	11,800	10,820	10,860
17	11,800	9,340	9,370
18	11,800	8,130	8,160
19	9,350	6,000	6,000
20	8,350	6,000	5,540
21	8,350	5,410	4,510
22	8,100	5,410	4,510
23	7,300	5,410	4,510
24	7,150	5,300	4,510
25	6,350	4,200	4,510
26	5,950	4,200	4,510
27	5,950	4,200	4,510
28	5,950	4,200	4,510
29	5,350	4,200	4,510
30	5,050	4,200	4,510
31	5,050	4,200	4,510
32	5,050	4,200	4,510
33	4,900	4,200	4,510
34	4,900	4,200	4,510
35	4,700	4,200	4,510
36	4,600	4,200	4,510
37	4,400	4,200	4,510
38	4,400	4,200	4,510
39	4,400	4,200	4,510
40	4,400	4,200	4,510

6.0 References

California Department of Water Resources (DWR). 1965. Aqueduct Design Criteria.

California Department of Water Resources (DWR). 2017. California Aqueduct Subsidence Study (CASS). June 2017.

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US Army Corps of Engineers (USACE). 2016. HEC-RAS River Analysis System Hydraulic Reference Manual. February 2016.

7.0 Definitions

TERM	DEFINITION
California Aqueduct (Aqueduct)	A system of canals, pumping plants, tunnels, and pipelines that conveys water collected from the Sierra Nevada Mountains and valleys of Northern and Central California over 400 miles to Central and Southern California; the Aqueduct is a key feature of the State Water Project (SWP). It is owned and operated by the California Department of Water Resources (DWR), with the excep. on of a federally owned portion that extends for 102 miles from the O'Neill Forebay to the federal terminus at Kettleman City (pool 21 of the Aqueduct). This federally owned portion—the San Luis Canal—was designed and constructed by the US Bureau of Reclamation (Reclamation); however, it is operated and maintained by DWR's San Luis Field Division (SLFD).
California Aqueduct Hydraulic Model	A detailed hydraulic (HEC-RAS) model of the Aqueduct that represents its current physical features and their current condition, including recent elevational changes caused by subsidence.
Design normal maximum water surface elevation	The water surface elevation 1 ft above the original design water surface elevation.
Design normal minimum water surface elevation	The water surface elevation 1 ft below the original design water surface elevation.
Freeboard	The Aqueduct has two primary types of freeboard: lined and unlined. Lined freeboard is the vertical distance from the water surface to the top of the concrete liner. In this report, unlined freeboard is the vertical distance from the water surface to the top of the canal embankment (i.e., the sum of lined and unlined embankment above the water surface). This vertical distance above the water surface provides a factor of safety.
Hydraulic conveyance capacity (HCC)	The maximum steady flow rate at which water can be conveyed through an Aqueduct pool or section (such as a check), under specific physical conditions and operating criteria. In this report, the HCC value given for a pool represents the flow rate at the upstream end of the pool.
Maximum steady flowrate	The maximum flow rate that a particular Aqueduct feature, such as a pool or a check structure, can sustain for long periods. Typically, in a timescale of days and up to a month.

TERM	DEFINITION
Model reach	Various stretches of the Aqueduct between pumping plants represented in the California Aqueduct Hydraulic Model. For example, the model has a "model reach" between Dos Amigos Pumping Plant and Buena Vista Pumping Plant, which includes pool 14 through pool 30.
Normal depth	In hydraulics, the depth at which uniform flow will occur in an open channel. For the Aqueduct, normal depth also represents the original design water surface elevation needed convey the original design capacity in the original design of the Aqueduct.
Normal operating range	The range of water surface elevations between the normal maximum water surface elevation and normal minimum water surface elevation. Water surface elevations under normal operating conditions are typically within this range. (See Standing Operating Order (SOO) 600.22 dated May 20, 2020, for additional information about the normal operating range in the Aqueduct.)
Original design capacity	The Aqueduct's HCC when it was first built according to original design criteria and operated consistent with the original operating criteria. The original design HCC was defined for each pool in cfs.
Original design	A generic term referring to the condition resulting from the integration of the original layout and design of the physical features and geometry of the Aqueduct, the subsidence forecast at the time of design, and the intended operation criteria. The Aqueduct Design Criteria (DWR, 1965) defined the original design criteria for the State-owned portions of the Aqueduct. The 1961 Joint Use Facilities Operations and Maintenance Agreement defined the original design criteria for the San Luis Canal.
Original design water surface elevation	The water surface elevation needed to convey the original design capacity in the original design of the Aqueduct.
Pool	A distinct segment of the Aqueduct bounded by either pump stations or check structures. Each pool has a defined original design conveyance capacity, storage capacity, and water surface elevations. The sequential numbering of the Aqueduct pools is coincident with the check structure number at the downstream end of a pool.
Pool service demand	The sum of deliveries measured at turnouts within a pool, plus the deliveries conveyed downstream measured at the check structure, plus any losses of water within the pool for a given time period.

TERM	DEFINITION
Precise Survey	An annual topographic survey performed by DWR that measures elevations at established monuments along the Aqueduct. Elevations are measured at several types of locations including top of concrete liner, check structures, bridges, turnouts, and other Aqueduct facilities.
Special conditions	Field constraints (i.e., orders) operators must abide by to help with the functionality of a particular facility or to give operators additional flexibility in a constrained part of the Aqueduct.
Steady state	A condition of the Aqueduct during which hydraulics are not changing. Flows through pools, turnouts, and checks are constant and water surface levels are not changing.
Stilling well	Aqueduct facilities used to monitor water surface elevations. They are located upstream and downstream of every check structure. They measure the water depth using a pressure transducer. The depth is converted and reported as water surface elevation.
Turnout loading	The configuration of engaged turnouts (i.e., the number of flowing turnouts, their location along a pool, and their flow rate). When calculating HCC, variations in the configuration of engaged turnouts can lead to differences in HCC estimates for a particular pool.

Appendix A. 2020 SOO Water Surface Elevation Operating Range

The Absolute Maximum, Absolute Minimum, Normal Maximum, and Normal Minimum water surface elevation values presented in Table A-1 references the 2020 SOO 600.22 (with no water surface elevation adjustments for 2018-2023 subsidence). The values in this table are reported in NGVD 29 as presented in the 2020 SOO 600.22.

Table A-1. 2020 SOO water surface elevation operating range (NGVD 29)

Check No.	MP Pool Limits	Measurement Type Measurement Limits	Absolute Maximum (ft)	Absolute Minimum (ft)	Normal Maximum (ft)	Normal Minimum (ft)
DA	70.90-86.73	Stilling/encoder 225.0-215.0	225.0	213.3	224.0	217.0
CK 14	86.96-95.06	Stilling/encoder 335.5-325.5	333	328	331	329
CK 15	95.11-109.5	332.5-322.5	330.4	326	329	327
CK 16	108.56-122.07	Stilling/encoder 329.5-319.5	326	322.1	323.4	322.5
CK 17	122.13-132.95	Stilling/encoder 326.0-316.0	322.2	320.4	321.6	320.6
CK 18	133.0-143.23	Stilling/encoder 324.5-314.5	320.8	317.5	319	318
CK 19	143.29-155.64	Stilling/encoder 321.0-311.0	316.4	314.6	316.1	315.1
CK 20	155.70-164.69	Stilling/encoder 319.0-309.0	315.1	313.4	314.7	314
CK 21	164.74-172.40	Pressure transducer 317.0-307.0	314.5	311	313.7	311.3
CK 22	172.44-184.82	Pressure transducer 314.5-304.5	312.6	310.5	311.8	310.8
CK 23	184.84-197.05	Pressure transducer 311.5-301.5	309.1	305.6	306.7	306.2

Check No.	MP Pool Limits	Measurement Type Measurement Limits	Absolute Maximum (ft)	Absolute Minimum (ft)	Normal Maximum (ft)	Normal Minimum (ft)
CK 24	197.07-207.94	Pressure transducer 309.0-299.0	305.5	303.4	304.9	303.9
CK 25	207.96-217.79	Pressure transducer 307.0-297.0	303.8	301.9	303.4	302.4
CK 26	217.81-224.92	Pressure transducer 305.0-295.0	303.7	300	301.4	300.4
CK 27	224.94-231.73	Pressure transducer 303.5-293.5	302.8	298.6	299.9	298.9
CK 28	231.75-238.11	Pressure transducer 302.0-292.0	298.6	295.6	297.7	296.7
CK 29	238.13-244.54	Pressure transducer 300.5-290.5	297.8	295.2	296.7	296
BV	244.56-250.99	Pressure transducer 299.0-289.0	296.2	293.1	294.6	293.6
CK 31	251.01-256.14	Pressure transducer 502.0-492.0	500.3	498.8	*	*
CK 32	256.18-261.72	Pressure transducer 502.0-492.0	499.5	497.5	499.3	497.8
CK 33	261.77-267.36	Pressure transducer 498.0-488.0	497.7	495.2	497.2	495.7
CK 34	267.43-271.27	Pressure transducer 496.5-486.5	495.9	493.4	495.4	493.9
WR	271.33-278.13	Pressure transducer 494.5-484.5	494.5	492.0	494.0	492.5
WG	278.13-280.36	Pressure transducer 729.0-719.0	726.5	722.0	725.3	722.5
CK 37	280.37-283.95	Pressure transducer 1244.5-1234.5	1243.5	1241.4	1242.9	*
CK 38	284.01-287.09	Pressure transducer 1243.5-1233.5	1241.0	1240.5	*	*

Check No.	MP Pool Limits	Measurement Type Measurement Limits	Absolute Maximum (ft)	Absolute Minimum (ft)	Normal Maximum (ft)	Normal Minimum (ft)
CK 39	287.14-290.21	Pressure transducer 1242.5-1232.5	1239.0	1238.9	*	*
ED	290.23-293.45	Pressure transducer 1241.5-1231.5	1239.5	1237.5	*	*
DA	70.90-86.73	Stilling/encoder 225.0-215.0	225.0	213.3	224.0	217.0

* Buena Vista (BV); Check (CK); Dos Amigos (DA)

Appendix B. Subsidence corrections and vertical datums conversions for 2020 SOO WSE operating range

Table B-1 shows the corrections made to the 2020 SOO 600.22 water surface elevations at each check to account for 2018-2023 subsidence. Table B- also shows the conversions made from the NGVD 29 Datum to the NAVD 88 Datum.

Table B-1. Corrections made to the 2020 SOO water surface elevation operating range (NAVD 88)

Check No.	MP Pool Limits	2020 SOO Normal Minimum (NGVD 29)	2020 SOO Normal Maximum (NGVD 29)	2018 - 2023 Correction for Subsidence	2020 SOO Normal Minimum (NGVD 29) Corrected for Subsidence between 2018 - 2023	2020 SOO Normal Maximum (NGVD 29) Corrected for Subsidence between 2018 - 2023	Vertical Datum Conversion Factor (NGVD 29 to NAVD 88)	2020 SOO Normal Minimum (NAVD 88) Corrected for Subsidence between 2018 - 2023	2020 SOO Normal Maximum (NAVD 88) Corrected for Subsidence between 2018 - 2023
DA	70.90-86.73	217	224				2.8		
14	86.96-95.06	329	331	0.14	329.14	331.14	2.9	332.04	334.04
15	95.11-109.5	327	329	0.06	327.06	329.06	2.8	329.86	331.86
16	108.56-122.07	322.5	323.4	-0.6	321.9	322.8	2.9	324.8	325.7
17	122.13-132.95	320.6	321.6	-0.78	319.82	320.82	2.8	322.62	323.62
18	133.0-143.23	318	319	-0.52	317.48	318.48	2.9	320.38	321.38
19	143.29-155.64	315.1	316.1	-0.46	314.64	315.64	2.9	317.54	318.54
20	155.70-164.69	314	314.7	-0.93	313.07	313.77	2.9	315.97	316.67
21	164.74-172.40	311.3	313.7	0.17	311.47	313.87	3	314.47	316.87
22	172.44-184.82	310.8	311.8	0.07	310.87	311.87	3	313.87	314.87
23	184.84-197.05	306.2	306.7	-0.46	305.74	306.24	3	308.74	309.24
24	197.07-207.94	303.9	304.9	-0.56	303.34	304.34	3	306.34	307.34
25	207.96-217.79	302.4	303.4	-0.4	302	303	3	305	306
26	217.81-224.92	300.4	301.4	-0.47	299.93	300.93	3	302.93	303.93
27	224.94-231.73	298.9	299.9	-0.28	298.62	299.62	3	301.62	302.62
28	231.75-238.11	296.7	297.7	-0.17	296.53	297.53	3	299.53	300.53
29	238.13-244.54	296	296.7	-0.02	295.98	296.68	3	298.98	299.68
BV	244.56-250.99	293.6	294.6		293.6	294.6	3	296.6	297.6
31	251.01-256.14	498.8	500.3	-0.12	498.68	500.18	3	501.68	503.18
32	256.18-261.72	497.8	499.3	-0.12	497.68	499.18	3	500.68	502.18
33	261.77-267.36	495.7	497.2	-0.31	495.39	496.89	3	498.39	499.89
34	267.43-271.27	493.9	495.4	-0.43	493.47	494.97	3	496.47	497.97
WR	271.33-278.13	492.5	494		492.5	494	3	495.5	497
WG	278.13-280.36	722.5	725.3		722.5	725.3	3	725.5	728.3
37	280.37-283.95	1241.4	1242.9	0.01	1241.41	1242.91	3	1244.41	1245.91
38	284.01-287.09	1240.5	1241	0.03	1240.53	1241.03	3	1243.53	1244.03
39	287.14-290.21	1238.9	1239	0.04	1238.94	1239.04	3	1241.94	1242.04
ED	290.23-293.45	1237.5	1239.5		1237.5	1239.5	3	1240.5	1242.5

* Buena Vista (BV); Dos Amigos (DA)

Appendix C. Pool-by-Pool Method for Calculating Hydraulic Conveyance Capacity

The pool-by-pool method was used to calculate the hydraulic conveyance capacity (HCC) estimates presented in the 2017 CASS Report and the 2019 CASS Supplemental Report. The pool-by-pool method has limitations that prevent it from fully capturing an actual systemwide scenario, because it does not observe conservation of mass. Large flow changes are assumed within the modeling between some of the pools that are physically infeasible.

Regardless of its simplifying assumptions and limitations, the pool-by-pool method of evaluating HCC can be useful for estimating the localized pool capacities (although it has limitations when considering the impacts from downstream pools). This information can be useful when evaluating some localized operations such as water transfers and for evaluating CalSim operations. The analysis requires an iterative process by which steady-state HEC-RAS simulations are executed until one converges on a solution that satisfies all of the intended criteria. Each iteration follows the following procedure:

1. Set an elevation boundary condition at the most downstream end of each model reach.[6] If the elevations of the concrete liner permit, this will typically be the average between the SOO normal minimum and normal maximum water surface elevations.
2. Run the HEC-RAS model with flowrates set at the most upstream cross section of each pool equal to the original design capacity for that pool. For modeling purposes, as part of the pool-by-pool analysis, gates remain fully open and flow changes between pools occur instantaneously at the most upstream cross section of each pool.
3. Check the model results for operational criteria violations. Typically, this consists of checking whether the model exceeds specified lined freeboard criteria. During this step, the resulting water surface elevation profile is compared against the top of liner elevations and locations are noted where the calculated water surface elevation does not match the desired freeboard criteria.
4. Adjust flow inputs to the model and rerun simulations until all intended operating criteria are satisfied, starting with the most downstream location where target criteria are not satisfied, by reducing flow in the most downstream pool at which the unsatisfied criteria had occurred. Iteratively reduce the flow in the pool by trial and error and re-run the model until the intended criteria are satisfied.

[6] In this context, “model reaches” refers to the various stretches of the Aqueduct between pumping plants. For example, the “model reach” between Dos Amigos Pumping Plant and Buena Vista Pumping Plant, which includes Pool 14 through Pool 30.

5. Progressively move upstream to the next location where criteria were not satisfied and adjust inputs and rerun simulations until all criteria are satisfied by repeating step 4. Once all criteria have been satisfied, the flowrate is noted for each pool.

If a particular pool has limited capacity (less than original design capacity), that limited capacity will typically be transferred to downstream pools when performing a pool-by-pool HCC assessment. This is because there is generally no way to introduce significant amounts of additional flow downstream of the limited capacity pool. This is referred to as the effective HCC, which is the HCC of a pool that is limited indirectly by the capacity of other system elements, such as upstream or downstream pools, rather than only by the concrete liner profile within the pool.

For example, pool 20 conveys water to pool 21. If pool 20 has a limited capacity of 5,600 cfs (and an original design capacity of 8,350 cfs), the limited capacity is carried downstream. Therefore, even if pool 21 has a greater standalone capacity of 7,000 cfs, the effective HCC of pool 21 (when analyzed using the pool-by-pool method) would be reported as 5,600 cfs. These results show that the capacity of upstream pools limit downstream pools and assumes there would be no other way to get water to the lower pool.

An alternative way to calculate pool-by-pool capacities is to assume that additional water could be delivered to downstream pools. Some turnouts also function as turn-ins, so there could be alternative ways to get water to downstream pools. To evaluate the discrete HCC of these pools—the HCC of a pool calculated without the indirect impacts from upstream or downstream pool conveyance limitations—the alternative “non-linear pool-by-pool analysis” assumes that pool capacities are not limited by the capacity of upstream pools. However, the non-linear pool-by-pool analyses, when performed with a model in series, often results in lower capacities for subsided pools. This is because flowrates in pools downstream of pools compromised by subsidence are increased (assuming downstream pools can have higher flowrates than upstream pools), which raises downstream pool water surface elevations and creates greater backwater resistance for upstream pool flows.

Another way to perform this evaluation and avoid impacts from downstream pools is to isolate the geometry and evaluate each pool separately. This analysis can help define the capacity of individual pools without the impacts from the rest of the system. However, it neglects the potential impacts from stages in downstream pools.

The pool-by-pool method may produce misleading results for a few pools. For example, the pool-by-pool analysis tends to overestimate the operable HCC of pool 19 because it does not directly take into consideration the limitations of pool 20 and the turnouts in pool 19 which lead to a lower effective HCC.

Other factors indirectly related to subsidence, besides lined freeboard, also can impact the hydraulic conveyance capacity of a pool. For example, as noted above, the capacity of an upstream pool can influence the effective hydraulic conveyance capacity of a downstream pool. Analyses showed that the capacity of a downstream pool can also limit the capacity of an upstream pool. Thus, more comprehensive approach for estimating hydraulic conveyance capacity is needed to reflect field operations and systemwide hydraulic constraints more closely.

Appendix D. Overview of Original Aqueduct Design

This appendix provides an overview of the approach the original designers of the Aqueduct took to establish the original intended flow rate (“original design capacity” described below), how that flow rate was used to size the canal pools, and how an additional factor of safety was incorporated through the addition of lined and unlined freeboard. This information provides additional context that may be useful in understanding how to calculate the long-term, sustainable, maximum steady flow rate HCC; as opposed to an instantaneous peak flow rate that cannot be sustained “long-term” (i.e., more than a few hours). As described below, the Aqueduct’s original design capacity considers demand at a monthly timescale and includes flexibility in the form of a peaking factor.

Original Design Capacity Calculation

Original design capacity, as used herein, refers to the Aqueduct’s original hydraulic conveyance capacity when it was first built according to design criteria and operated according to original operating criteria. The 1965 Aqueduct Design Criteria (DWR, 1965) indicate that the flow rate used to size the canal comprised two inputs: the Area Service Demand and a Peaking Factor. Area Service Demand refers to the water demanded from a specific Aqueduct facility, including deliveries and losses. The largest portion of an Area Service Demand is the net delivery volume.

To compute original design capacity by pool, an analogous pool-specific variable, pool service demand, is needed. For any given pool for a given time period, the pool service demand is the sum of the deliveries measured at turnouts within that pool, the deliveries conveyed downstream measured at the check structure, and any losses of water within the pool.

A monthly Peaking Factor (as defined in DWR, 1965) is added to the pool service demand to account for variable demand patterns through the year. The following Equation 1 for original design capacity was developed while evaluating the 2023 HCC based on the description provided in Section 2.1.1.6 of the Aqueduct Design Criteria (DWR, 1965).

Equation 1: Original Design Capacity

$$Q_D = Q_{max} + PF$$

Where:

Q_D = Original design capacity (cfs)

Q_{max} = The flow rate required for the month with the most demand if the demand was met with continuous flow during that month (cfs)

PF = Peaking Factor defined as $PF = 0.2\ (Q_{max} - Q_{avg})$

Q_{avg} = The flow rate required if the entire annual demand was delivered with continuous flow divided evenly throughout the year (cfs)

For example, consider a hypothetical pool with a pool service demand of 3 million acre-feet (af) per year. If distributed evenly over 12 months, this results in an average of 250,000 af per month, or an average continuous flow rate of approximately 4,200 cfs. Suppose that the month with the highest demand along this pool requires 20 percent of the total annual delivery. This is equal to 600,000 af or a required flow rate of approximately 10,083 cfs. Using Equation 1, the original design capacity for this hypothetical canal pool is 11,260 cfs.

QD = 10,083 cfs + 0.2 (10,083 cfs – 4,200 cfs) = 11,260 cfs

Sizing the Original Canal Sections

After determining the original design capacity, Aqueduct designers established the depth, width, and side slopes of the canal trapezoidal sections necessary to provide the intended flow rate under the intended original operating criteria. A portion of the analyses focused on identifying the ideal, most hydraulically efficient canal depth-to-width ratio for a trapezoidal channel to convey the original design capacity. Based on these analyses, a range of depth-to-width ratios between 0.6 and 0.9 was recommended, varying based on the side slopes of the trapezoidal channel. Subsequently, the geotechnical properties of the foundation soil were used to determine the final channel cross section dimensions. Different approaches were used for the hydraulic design of the canal for the SLFD (Reclamation design) and SJFD (DWR design).

Numerous tests by Reclamation in the Delta-Mendota and Friant-Kern canals informed the selection of a hydraulic design approach for the San Luis Canal. Based on information discovered during these tests, Reclamation decided to design the San Luis Canal using Manning's equation. When applying Manning's equation, Reclamation designers used a roughness coefficient of n = 0.016 for the canal, and additional provisions were included to account for energy losses at bridges.

In the design of SJFD facilities, DWR used the Colebrook-White equation to design the Aqueduct (DWR, 1965). Tests performed prior to the design of the Aqueduct indicated that for larger channels, such as the Aqueduct, the roughness coefficient in Manning's equation must be increased to match the results generated by the Colebrook-White equation. This does not mean that the actual roughness of the canal increases, but rather that, according to these tests, a roughness higher than the typical value or range corresponding to the liner material is needed to match test results (DWR, 1965).

Once the Aqueduct slope and cross section dimension (bottom width and side slopes) were established, Equation 2 and/or Equation 3 were used to determine the normal water surface elevation. This normal water surface elevation is referred to herein as the original design water surface elevation.

Equation 2: Manning's Equation

$$V = \frac{1.486}{n} R^{2/3} S^{1/2}$$

Where:
V = velocity (ft/s)
n = Manning's roughness coefficient [0.016 in the original design of the San Luis Canal]
R = hydraulic radius (ft)
S = canal slope (ft/ft)

Equation 3: Colebrook-White Equation

$$V = -\sqrt{32gRS} * log\left[\frac{k}{14.8R} + \frac{1.255\upsilon}{R\sqrt{32gRS}}\right]$$

Where:
V = velocity (ft/s)
g = gravitational constant (32.2 ft/s^2)
R = hydraulic radius (ft)
S = canal slope (ft/ft)
k = equivalent sand-grain roughness [0.005 in the original design of the SJFD Aqueduct]
υ = kinematic viscosity (ft^2/s)

The elevations calculated using these equations are only a portion of the total canal depth (DWR, 1965). A subsequent assessment was implemented to calculate the height of the appropriate lined and unlined freeboard.

Original Design Freeboard

The chart shown in Figure D-1 was used as part of the original design to calculate the minimum appropriate freeboard. The source equations or analyses used to derive these relationships are not presented in the Aqueduct Design Criteria (DWR, 1965). However, the document does include some additional context.

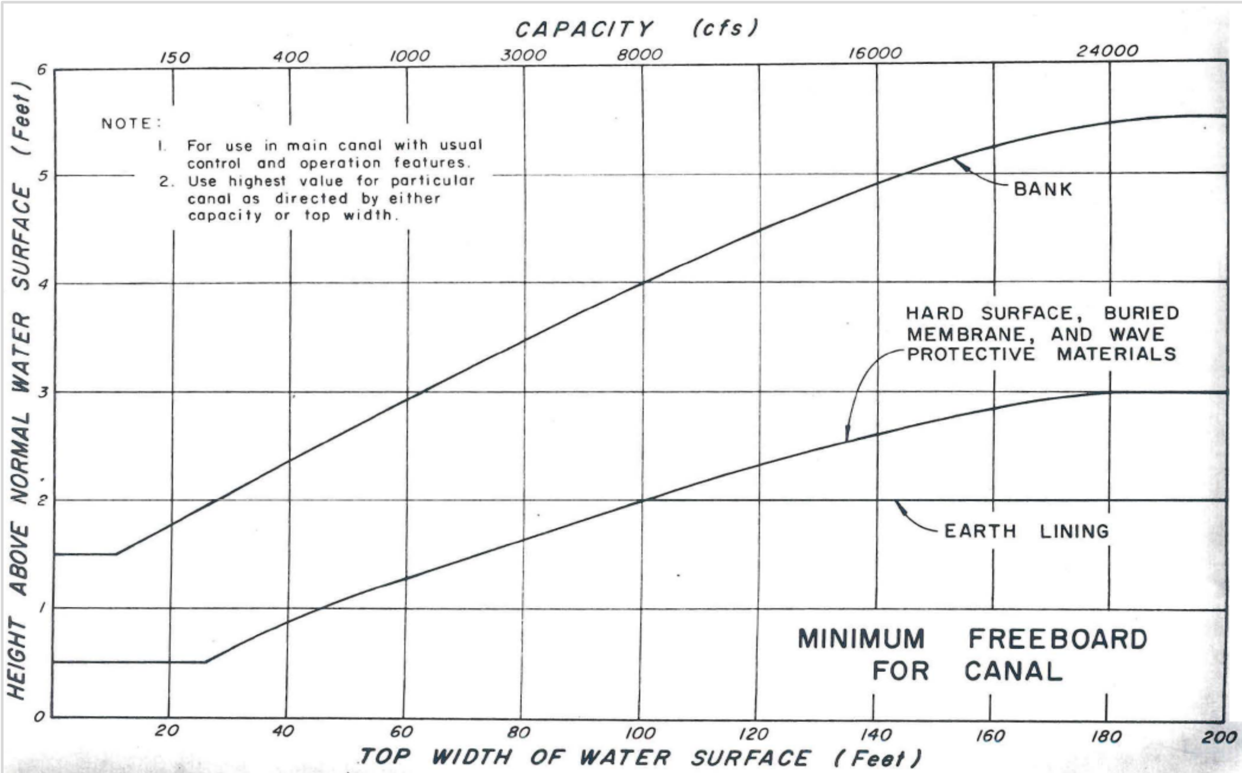

Figure D-1. Minimum freeboard value for Aqueduct canal with normal control and operation features

Source: DWR, Aqueduct Design Criteria, Figure 2.3-3