Technical Memorandum

Date: September 1, 2016

To: State Water Resources Control Board

From: R. Craig Addley, PhD, Cardno, Inc.

Subject: Effects of California WaterFix operations on Folsom and Shasta reservoirs and American River water deliveries

Overview

This memorandum discusses the effects of California WaterFix modeled operations (as presented in the Bay Delta Conservation Plan/California WaterFix RDEIR/SDEIS and Petitioners' Exhibits filed in support of the California WaterFix water right change petition) on Folsom and Shasta reservoirs and the resulting injury to American River water users that divert water from Folsom Reservoir.

The key findings of this memorandum are that:

- Modeled California WaterFix storage operations at Folsom Reservoir limit American River water users' access to water from Folsom Reservoir in dry years resulting in injury.
- Modeled California WaterFix operations do not comply with the end-of-September (EOS) storage criteria for Shasta Reservoir as specified in the National Marine Fisheries Service 2009 Biological Opinion Reasonable and Prudent Alternative (NMFS 2009 BO RPAs) - modeled storage is much lower than the storage specified in the 2009 BO to protect water temperature in winter-run salmon spawning/incubation habitat.
- Compliance with the NMFS 2009 BO RPA (also various water temperature criteria) would require increased storage in Shasta Reservoir as compared to WaterFix modeling, which would cause larger draw-downs of Folsom Reservoir than disclosed in the WaterFix modeling (if other portions of the system remain as modeled) and the result would be injury to American River water users in most years.
- The WaterFix No Action Alternative (NAA) is not a technically appropriate baseline (for absolute or comparative purposes) because it does not adequately depict Folsom Reservoir storage in the driest years and does not meet Shasta Reservoir storage requirements in the 2009 NMFS BO (also various water temperature criteria).
- Operations criteria for Folsom Reservoir that provide storage protection (with a safety factor) for both individual years and carryover storage for multiple dry year sequences are necessary to prevent injury to the American River water users and should be included in DWR's/Reclamation's water rights permit terms.

WaterFix Folsom Reservoir Operations

Modeled Folsom Reservoir operations under California WaterFix Alternative 4A H3 (or other scenarios within Boundary 1 and Boundary 2) and the No Action Alternative (NAA) impact the ability of American River water users to meet water demands in drier years (Figure 1). Figure 1 shows Folsom Reservoir operations from California WaterFix Testimony (DWR-515; Figure 14). In approximately 10% of the years, EOS storage is below a safe level required for diversion by Folsom Reservoir water purveyors (see Appendix A Folsom Reservoir Municipal Outlet Pumping Curve Delivery-Storage Relationship). Delivery shortages greater than 50 cfs (average for a month) would occur in nine of 82 years (Figure 2) and reservoir levels would be dangerously close to causing delivery restrictions in several other years (Figure 1).

Extremely low EOS storage (carryover storage for the subsequent year) in approximately 10% of the years increases the likelihood that a subsequent severe drought year with very low inflow such as 1977 or 2015 could result in disastrous water supply consequences. The California WaterFix operations would provide inadequate carryover storage in those years when EOS storage is extremely low (Figure 1). It should be noted that average storage typically decreases after September.

The WaterFix modeling of Alternative 4A H3 (or other scenarios within Boundary 1 and Boundary 2) and NAA represent modeling/operation decisions to maintain south of delta export and delta water quality in the face of estimated future climate change to the determent of upstream local M&I water supply deliveries at Folsom Reservoir. For example, the EOS storage draw-down on Folsom Reservoir presented in the WaterFix modeling is substantially greater in comparison to EOS storage draw-down in the Existing Conditions modeled in the 2008 OCAP Biological Assessment study (BA) without climate change assumptions (Figure 1). The differences in the modeling/operations assumptions have large relative impacts on the water supply security of upstream American River water users (Reclamation 2008, Chap. 10, Pg 10-63, Figure 10-92).

Using the WaterFix NAA as a baseline to parse impacts related to WaterFix alternatives does not appear to be appropriate. NAA simulates operations of Folsom Reservoir storage in 5-10% of the driest years far below current management or any future management that seems reasonable. In September 2015, one of the driest periods on record, Folsom Storage was at 170 TAF at the end-of-September. Conversely, the NAA model shows Folsom Reservoir at dead pool (90 TAF) at EOS for the driest 5% of years. This, along with concerns identified below related to the NAA operations at Shasta Reservoir, suggests that the NAA, as modeled in WaterFix, is not a technically appropriate baseline for absolute or comparative purposes.

Figure 14. Simulated End of September Folsom Storage Exhibit DWR-514

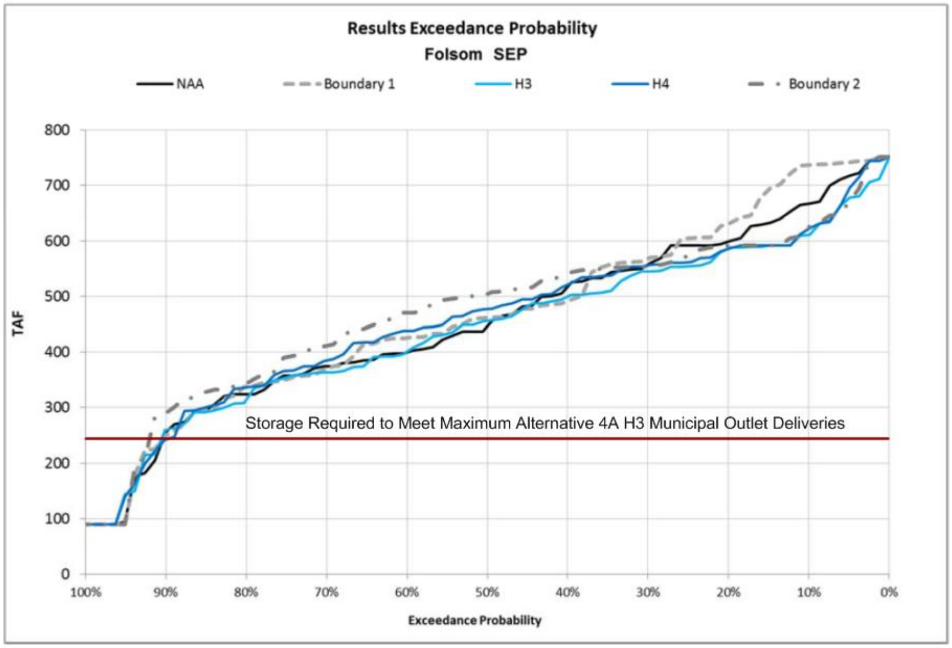


Figure 14. Simulated End of September Folsom Storage Exhibit DWR-514

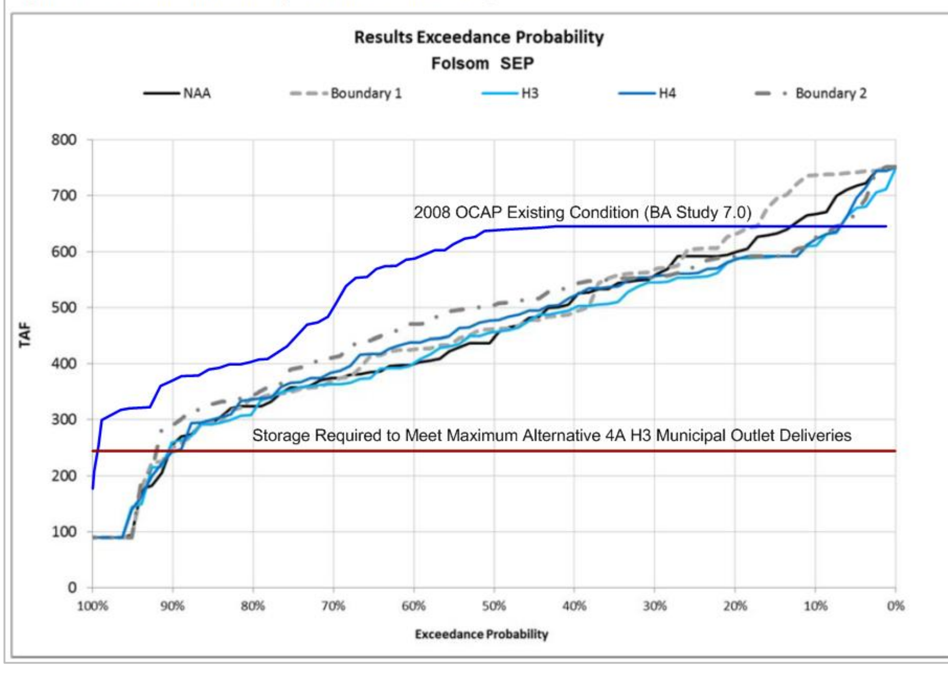


Figure 1. Simulated end-of-September Folsom Reservoir Storage from California WaterFix Testimony (DWR-515) compared to storage requirement to meet American River water user municipal outlet water demands in the WaterFix Alternative 4A H3 (top plot) and the same plot overlain with the 2008 OCAP Existing Conditions modeling (Reclamation 2008, Chap. 10, Pg 10-63, Figure 10-92) (without climate change) (bottom plot).

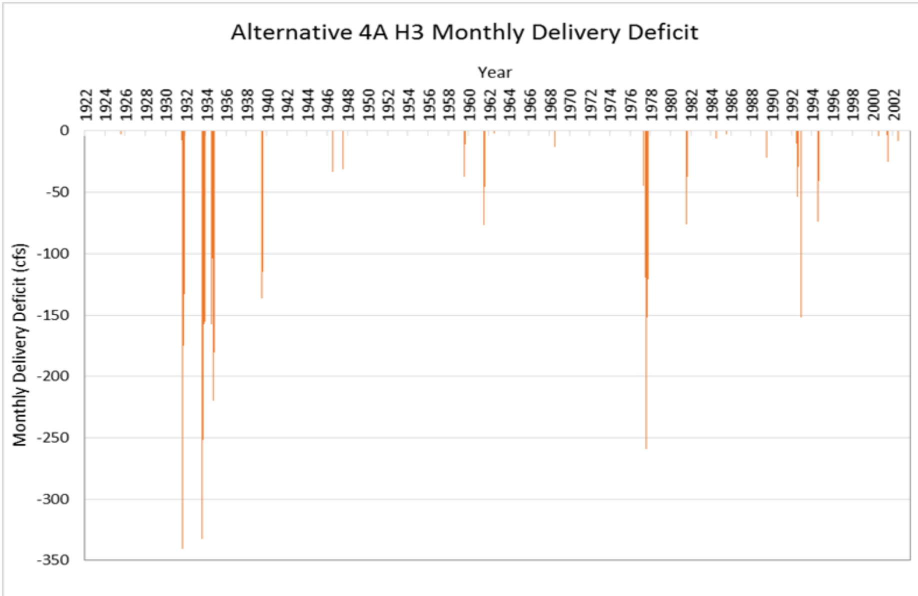


Figure 2. Effects of California Water Fix operations Alternative 4A H3 on American River water user deficits by month (based on Alternative 4A H3 modeled Folsom reservoir storage and municipal outlet deliveries).

WaterFix Shasta Reservoir Operations

Shasta Reservoir operations in Alternative 4A H3 (or other scenarios within Boundary 1 and Boundary 2) and NAA do not meet the 2009 BO or Amended 2011 BO RPA criteria designed to protect winter-run salmon in the Sacramento River downstream of Shasta Reservoir. The California WaterFix Shasta Reservoir EOS storage is on average 442 TAF below the 2009 BO RPA performance criteria[1] (Figure 3). Figure 3 shows Shasta Reservoir EOS storage as presented in the California WaterFix Testimony (DWR-515; Figure 12) compared to the 2009 BO RPA requirements and the 2008 OCAP BA modeling (Reclamation 2008, Chap. 10, Pg 10-32, Figure 10-46) and Appendix B shows that the 10-year running average of Shasta Reservoir operations does not meet the performance criteria contained in the 2009 BO RPAs.

Supplemental information provided in Appendix C illustrates that Alternative 4A H3 (or other WaterFix Alternatives or NAA) Shasta Reservoir EOS operations are not viable operations in relation to winter-run Chinook salmon temperature protection criteria and would have to be modified. For example, Appendix C demonstrates that as specified in the 2009 BO RPA (1) spring Shasta Reservoir Storage (e.g., April/May) affects water temperature downstream of Keswick Reservoir; (2) Shasta Reservoir EOS storage has an effect on water temperature downstream of Keswick Reservoir the following year (lower storage generally equates to higher water temperature); and (3) modeled Alternative 4A H3 (or other WaterFix Alternatives or NAA) water temperatures result in a large increase in water temperature compared to the WaterFix REIR/SEIS Existing Conditions scenario. In addition, the modeled Alternative

[1] 2011 Amended 2009 BO page 18 states "the following long-term performance measures shall be attained." 87% of years – minimum EOS storage of 2.2 MAF; 82% of years – minimum EOS storage of 2.2 MAF and EO April storage of 3.8 MAF in the following year; 40% of years – minimum EOS storage 3.2 MAF. Measured as a 10-yr running average.

4A H3 (or other WaterFix Alternatives or NAA) water temperatures exceed the 2009 BO criteria, State Water Resources Control Board Order WR 90-5 (SWRCB-24) and WR 91-1 criteria, Basin Plan criteria for the Central Valley Region (SWRCB-34), and the thermal tolerance of winter-run Chinook salmon egg incubation (Appendix C). Also, increasing the water temperature downstream of Shasta Reservoir under Alternative 4A H3 compared to Existing Conditions is contrary to how the reservoir is currently being managed to reduce water temperatures in the Sacramento River downstream of Keswick Dam below 56^{o}F (e.g., NMFS March 31, 2016; USBR June 27, 2016; NMFS June 28, 2016).

Because the WaterFix NAA scenario does not provide a viable operation that meets the existing Shasta Reservoir storage or water temperature requirements downstream of Shasta Reservoir (e.g., 2009 BO RPA, SWRCB Order WR 90-5 and WR 91-1 criteria, Basin Plan criteria), NAA as modeled in WaterFix is not a technically appropriate baseline for absolute or comparative purposes.

Compliance with Shasta Reservoir 2009 BO RPA Effects on Folsom Reservoir

Compliance with the 2009 BO RPA Shasta Reservoir EOS storage criteria, designed to protect winter-run Chinook salmon, requires much higher Shasta Reservoir EOS storages than modeled in the California WaterFix operations. Specifically, Shasta Reservoir EOS storage based on the 2009 BO RPA criteria would need to be, on average, 442 TAF higher (Figure 3) and, if other California WaterFix deliveries were held static (e.g., delta water quality and delta exports) as depicted in the petitioners' modeling and testimony, the primary potential operational solution to comply with 2009 BO RPA would be to greatly increase draw-down of Folsom Reservoir storage compared to modeled storage. Conservatively assuming only 50% of the approximately 442 TAF of the water needed to comply with the Shasta storage performance criteria came from Folsom Reservoir (e.g., 200 TAF), the result would have a large impact on Folsom Reservoir storage (see illustration in Figure 4). These operations would result in injury to American River water users in many years. Additionally, another >200 TAF of water would have to come from some other part of the CVP/SWP system.

Summary/Recommendations

Future operation of Folsom Reservoir as disclosed in the California WaterFix RDEIR/SDEIS and California WaterFix water right change petition exhibits represent to the best of our knowledge how the WaterFix would affect the operations of the CVP/SWP. Those operations result in extremely low EOS Folsom Reservoir storage that would cause injury to American River water user deliveries in dry years and would not include adequate carryover storage to protect against the second year of a drought sequence. The injury could be greatly exacerbated given that the California WaterFix operations disclosed at Shasta Reservoir would need to be modified (storage increased to comply with the 2009 BO RPA) and would require additional water releases from Folsom Reservoir; thereby, resulting in further injury to American River water users in many years.

Operations criteria for Folsom Reservoir that provide storage protection (with a safety factor) in both a single year and carryover for a multiple year drought sequence are necessary to prevent injury to the American River water users and should be included in DWR's/Reclamation's water right permits related to the California WaterFix Project.

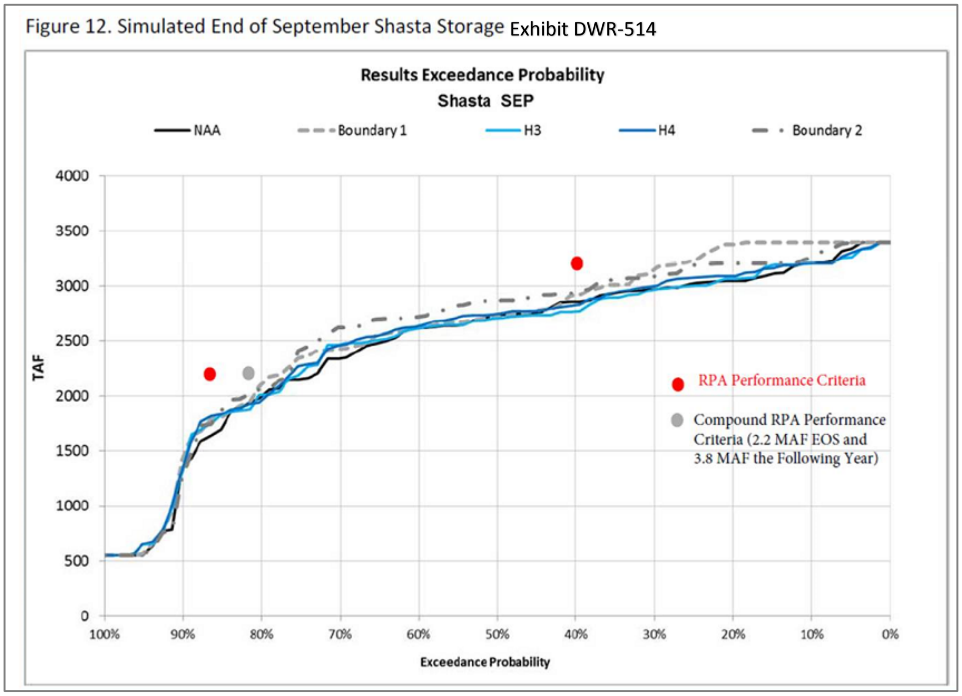


Figure 12. Simulated End of September Shasta Storage Exhibit DWR-514

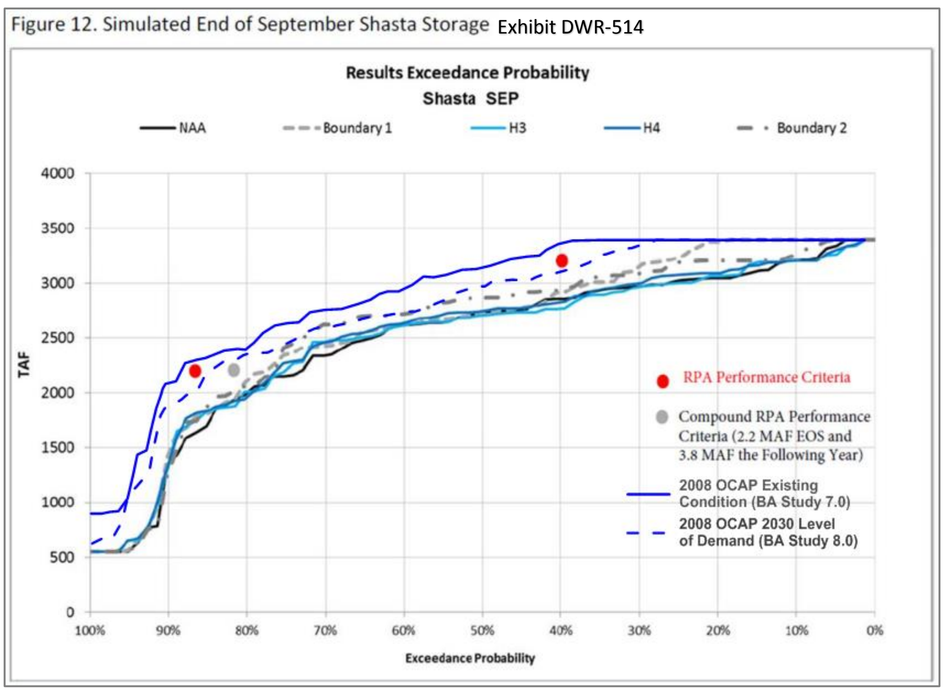
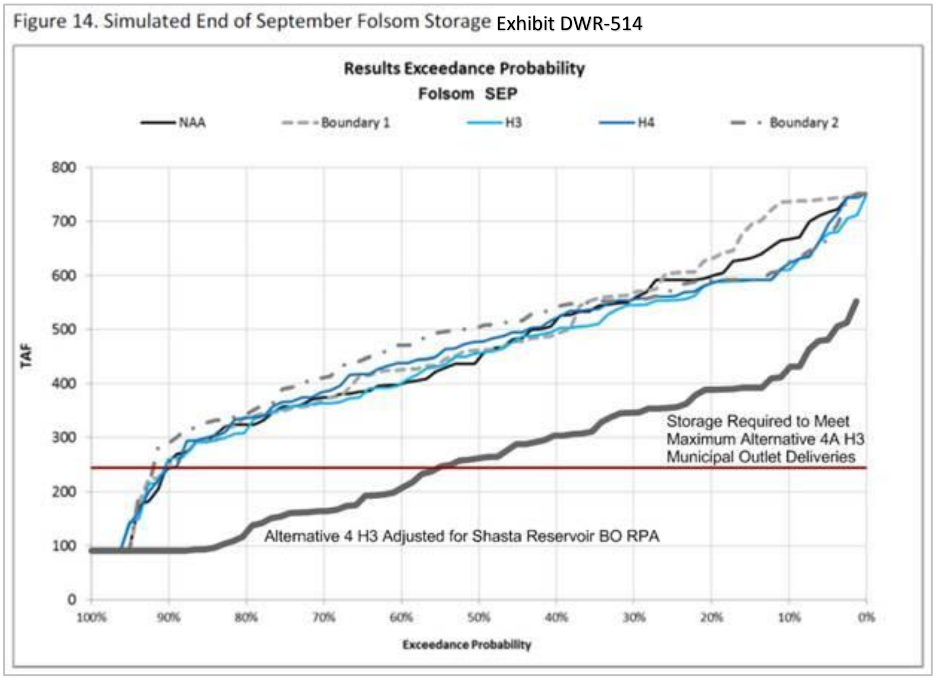


Figure 12. Simulated End of September Shasta Storage Exhibit DWR-514

Figure 3. Shasta Reservoir Storage with the NMFS 2009 BO End-of-September performance criteria (top plot) and the 2008 BA modeling (bottom plot) (note the 2009 BO RPA performance criteria appear to be derived directly from the 2008 BA modeling). The underlying graphic is from California Water Fix Testimony (DWR-515) and the 2008 OCAP BA information is from Reclamation (2008; Chap. 10, Pg 10-32, Figure 10-46).



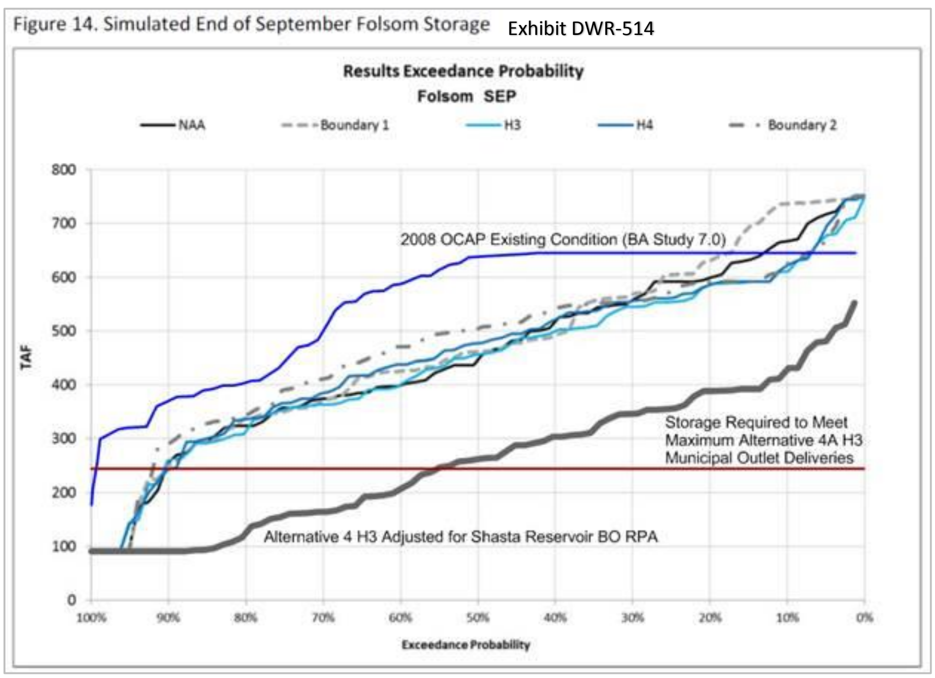


Figure 4. Folsom Reservoir elevations based on using 200 TAF of Folsom Storage (conservative estimate) to offset the 422 TAF Shasta Reservoir storage requirements to meet 2009 BO end-of-September performance criteria (heavy gray line) compared to California WaterFix Testimony (DWR-515; Figure 14) (top plot). Same plot with 2008 Biological Assessment Existing Conditions modeling (Reclamation 2008, Chap. 10, Pg 10-63, Figure 10-92) included for comparison (bottom plot).

References

Alternative 4A H3, NAA, and others. http://www.waterboards.ca.gov/waterrights/water_issues/programs/bay_delta/california_waterfix/water_right_petition.shtml. May 25, 2016. https://ftp.waterboards.ca.gov/

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Bureau of Reclamation (Reclamation). June 27, 2016. Final Sacramento River Temperature Management Plan. http://www.westcoast.fisheries.noaa.gov/publications/Central_Valley/Water%20Operations/bureau_of_reclamation_s_sacramento_river_temperature_management_plan_-_june_27__2016.pdf. Accessed August 25, 2016. Exhibit ARWA-206

National Marine Fisheries Service (NMFS). 2011. 2011 Amendments to the NMFS OCAP RPA. http://www.westcoast.fisheries.noaa.gov/publications/Central_Valley/Water%20Operations/Operations,%20Criteria%20and%20Plan/040711_ocap_opinion_2011_amendments.pdf. Accessed August 25, 2016. Exhibit SWRCB-83

National Marine Fisheries Service (NMFS). 2009. Biological and conference opinion on the long-term operations of the Central Valley Project and State Water Project. National Marine Fisheries Service Southwest Region. http://www.westcoast.fisheries.noaa.gov/publications/Central_Valley/Water%20Operations/Operations,%20Criteria%20and%20Plan/nmfs_biological_and_conference_opinion_on_the_long-term_operations_of_the_cvp_and_swp.pdf. Accessed August 25, 2016. Exhibit SWRCB-84

National Marine Fisheries Service (NMFS). March 31, 2016. Sacramento River Temperature Management Plan Letter. http://www.westcoast.fisheries.noaa.gov/publications/Central_Valley/Water%20Operations/nmfs__march_31__2016__response_to_the_bureau_of_reclamation_s_march_forecast.pdf. Accessed August 25, 2016. Exhibit ARWA-207

National Marine Fisheries Service (NMFS). June 28, 2016. Sacramento River Temperature Management Plan Letter. http://www.westcoast.fisheries.noaa.gov/publications/Central_Valley/Water%20Operations/nmfs_concurrence_on_the_bureau_of_reclamation_s_sacramento_river_temperature_management_plan-_june_28__2016.pdf. Accessed August 25, 2016. Exhibit ARWA-208

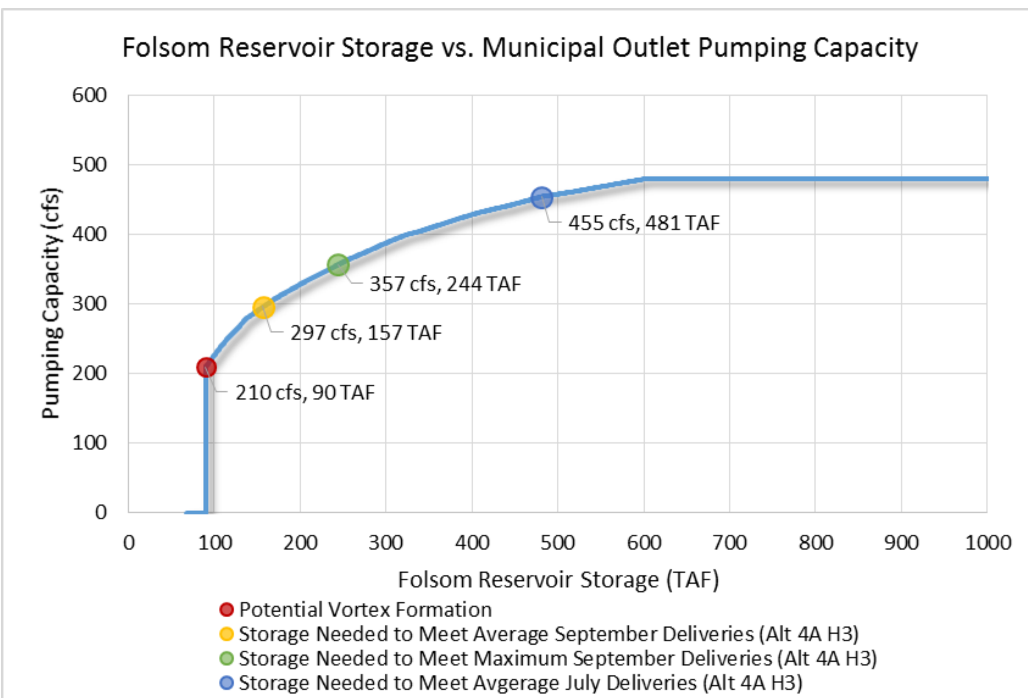
Appendix A

Folsom Reservoir Municipal Outlet Pumping Curve and Delivery-Storage Relationship

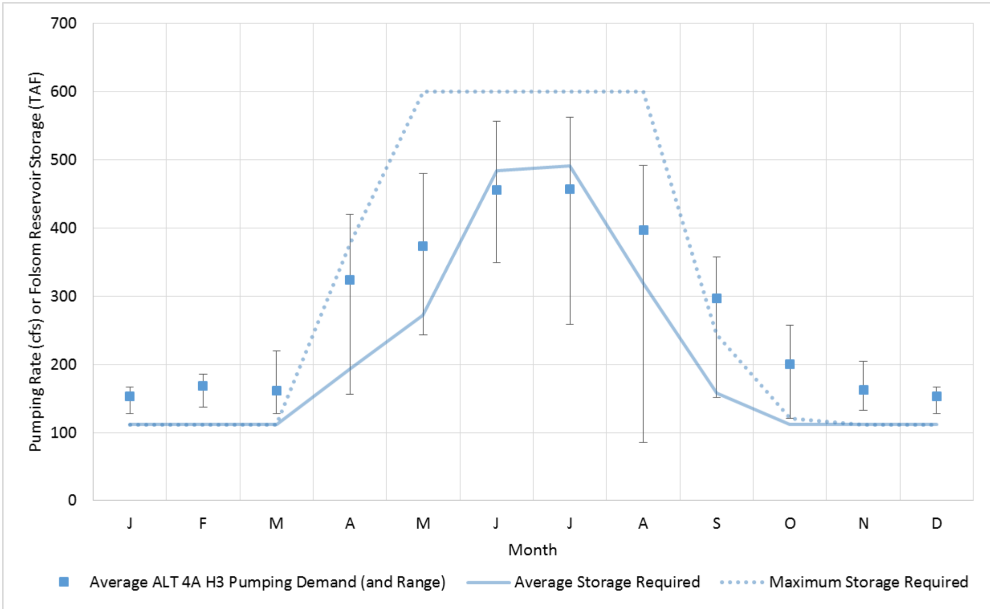
Folsom Reservoir provides water to multiple municipalities and water users through a single outlet (84-inch) at an elevation of 317 feet (centerline), which feeds a pumping station. The approximate pumping station head (storage) versus pumping capacity curve is provided in Figure 1 (ESA 1996). The pumping station has the potential to become inoperable due to vortex formation when the elevation in Folsom Reservoir is at approximately 330 ft (NGVD 29) (ESA 1996; Water Resource Engineering 2011), which is at a storage level of approximately 90 TAF (Bureau of Reclamation 2005). Folsom Reservoir elevation and corresponding storage required to meet various September and summer municipal outlet delivery rates from the California WaterFix Alternative 4A H3 modeling are shown in Table 1 and Figure 1. The average monthly municipal outlet deliveries (and range) from Alternative 4A H3 are shown in Figure 2 with the corresponding Folsom Reservoir elevations required for pumping.

Appendix A - Table 1. Summary of Folsom water surface elevations and pumping relationship (deliveries from Alternative 4 H3).

Surface Elevations (feet - mean sea level NGVD29)	Storage (acre-feet) [Based on 2005 Sediment Survey]	Pumping Relationship
415	481,466	Average July Alt 4A H3 deliveries 455 cfs
377	244,180	Maximum September Alt 4A H3 deliveries 357 cfs
356	157,031	Average September Alt 4A H3 deliveries 297 cfs
330	89,869	Vortex potential at Folsom Dam Intake, depending on volume of pumping.



Appendix A - Figure 1. Folsom Reservoir Storage vs. Pumping Capacity Based on ESA (1996) (Storage values calculated from USBR 2005 sediment survey; see Table 1 for highlighted values).



Appendix A – Figure 2. Monthly Average and Maximum Folsom Reservoir Storage vs. Pumping Required Based WaterFix Alternative 4A H3 Deliveries, the ESA (1996) Pump Curve, and Elevation – Storage Values Calculated from Bureau of Reclamation (2005).

References:

Bureau of Reclamation. 2005. Folsom Lake 2005 Sedimentation Survey. Bureau of Reclamation, Technical Service Center, Denver, CO. Exhibit ARWA-209

ESA Consultants, Inc. 1996. Increasing Water Supply Pumping Capacity at Folsom Dam. Prepared for City of Roseville Department of Environmental Utilities. Exhibit ARWA-210

Water Resources Engineering, Inc. 2011. Folsom Pumping Plant System Capacity Evaluation. Prepared for US Bureau of Reclamation Central California Area Office Folsom, California. Exhibit ARWA-211

Appendix B

Shasta Reservoir 2009 Biological Opinion Storage RPA 10 year Running Average Compliance

The 2009 Biological Opinion Storage RPA (NMFS 2011) identifies three Shasta Storage performance criteria that must be met based on a 10-year running average (Table 1). An excerpt of NMFS (June 28, 2016) that includes the NMFS approach to the 10-year running average. The 10-year running average for each of the criteria (40%, 82% and 87% of years) are shown in Figures 1, 2 and 3, respectively. Alternative 4A H3 does not meet any of the three performance measures (storage is much too low).

Appendix B - Table 1. 2009 Biological Opinion Shasta Storage Reasonable and Prudent Action Performance Criteria.

Action 1.2.1 Performance Measures.

Objective: To establish and operate to a set of performance measures for temperature compliance points and End-of-September (EOS) carryover storage, enabling Reclamation and NMFS to assess the effectiveness of this suite of actions over time. Performance measures will help to ensure that the beneficial variability of the system from changes in hydrology will be measured and maintained.

Action: The following long-term performance measures shall be attained. Reclamation shall track performance and report to NMFS at least every 5 years. If there is significant deviation from these performance measures over a 10-year period, measured as a running average, which is not explained by hydrological cycle factors (*e.g.,* extended drought), then Reclamation shall reinitiate consultation with NMFS.

Performance measures for EOS carryover storage at Shasta Reservoir:

- 87 percent of years: Minimum EOS storage of 2.2 MAF
- 82 percent of years: Minimum EOS storage of 2.2 MAF and end-of-April storage of 3.8 MAF in following year (to maintain potential to meet Balls Ferry compliance point)
- 40 percent of years: Minimum EOS storage 3.2 MAF (to maintain potential to meet Jelly's Ferry compliance point in following year)

Measured as a 10-year running average, performance measures for temperature compliance points during summer season shall be:

- Meet Clear Creek Compliance point 95 percent of time
- Meet Balls Ferry Compliance point 85 percent of time
- Meet Jelly's Ferry Compliance point 40 percent of time
- Meet Bend Bridge Compliance point 15 percent of time

Rationale: Evaluating long-term operations against a set of performance measures is the only way to determine the effectiveness of operations in preserving key aspects of life history and run time diversity. For example, maintaining suitable spawning temperatures down to Bend Bridge in years when this is feasible will help to preserve the part of winter-run distribution and run timing that relies on this habitat and spawning strategy. This will help to ensure that diversity is preserved when feasible. The percentages are taken from those presented in the CVP/SWP operations BA, effects analysis in the Opinion, and NMFS technical memo on historic Shasta operations.

Appendix B - Table 2. Excerpt for NMFS June 28, 2016 letter regarding the NMFS approach to calculating the 2009 BO RPA 10 –year running average.

from May 15 through October 31. In addition, there is a 10-year average performance measure and for temperature compliance points on the Sacramento River during the summer season:

- Meet Clear Creek compliance point 95% of time
- Meet Balls Ferry compliance point 85% of time
- Meet Jelly's Ferry compliance point 40% of time
- Meet Bend Bridge compliance point 15% of time

So far the current 6-year average (2010-2015) since issuance of the CVP/SWP operations Opinion is below this performance metric (see Table 1):

- Clear Creek was met 66% of the time
- Balls Ferry was met 50% of the time
- Jellys Ferry was met 50% of the time
- Bend Bridge was met 0% of the time

Also there is a 10-year average performance measures associated with meeting EOS carryover storage at Shasta Reservoir in order to maintain the potential to meet the various temperature compliance points:

- 87% of years: Minimum EOS storage of 2.2 million acre-feet (MAF)
- 82% of years: Minimum EOS storage of 2.2 MAF and End of April (EOA) storage of 3.8 MAF in following year (to maintain potential to meet Balls Ferry compliance point)
- 40% of years: Minimum EOS storage of 3.2 MAF (to maintain potential to meet Jelly's Ferry compliance point in following year)

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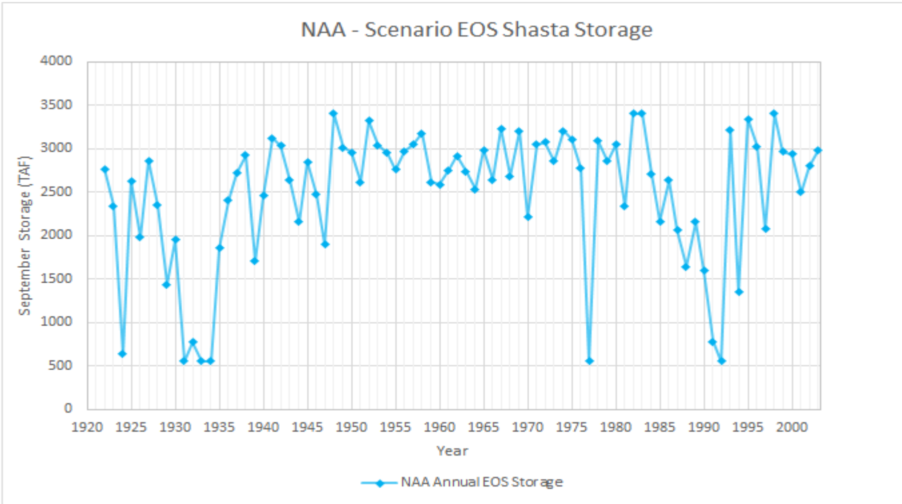
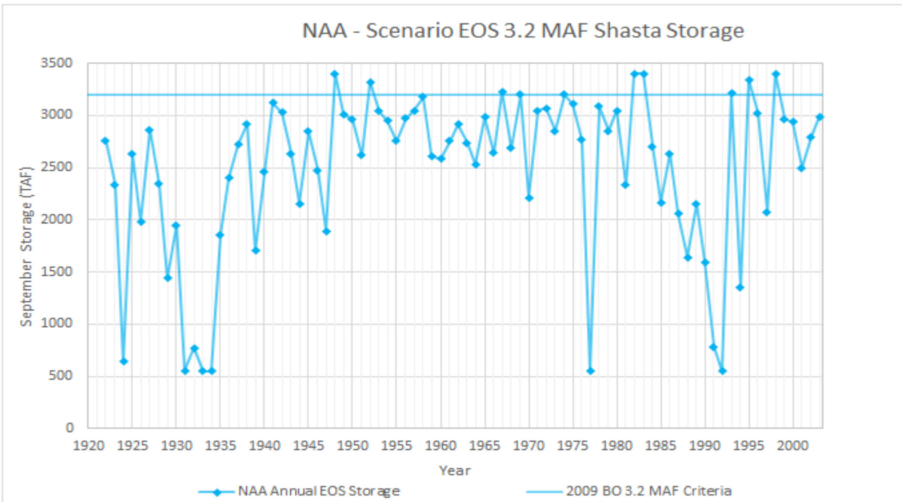
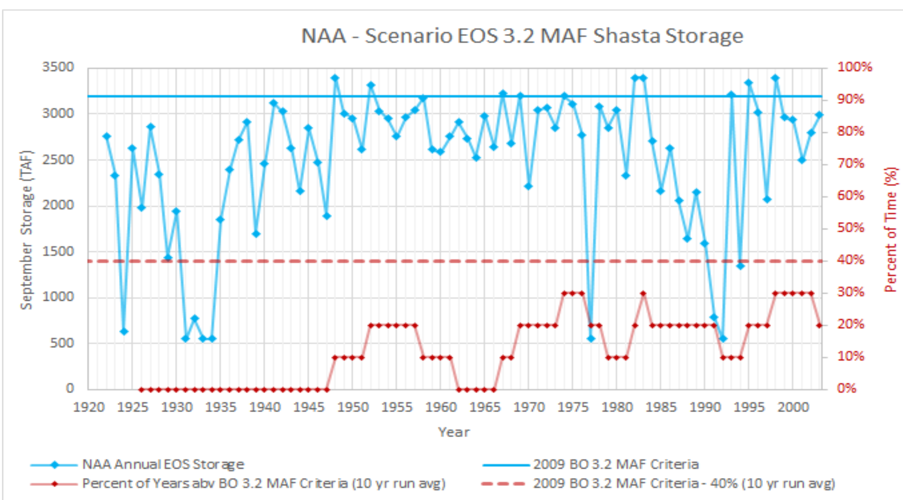
The current 6-year average also falls short of this performance metric:

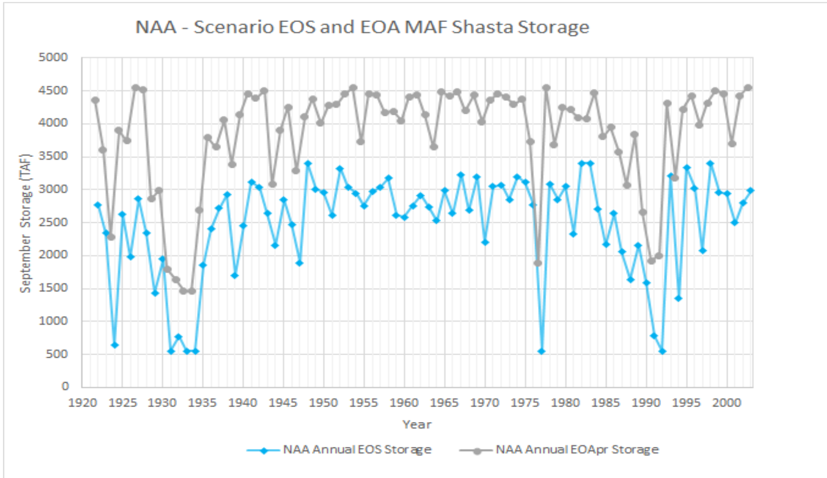
- 50% of Years: Minimum 2.2 MAF
- 50% of Years: Minimum 2.2 MAF and EOA 3.8 MAF
- 33% of Years: Minimum 3.2 MAF

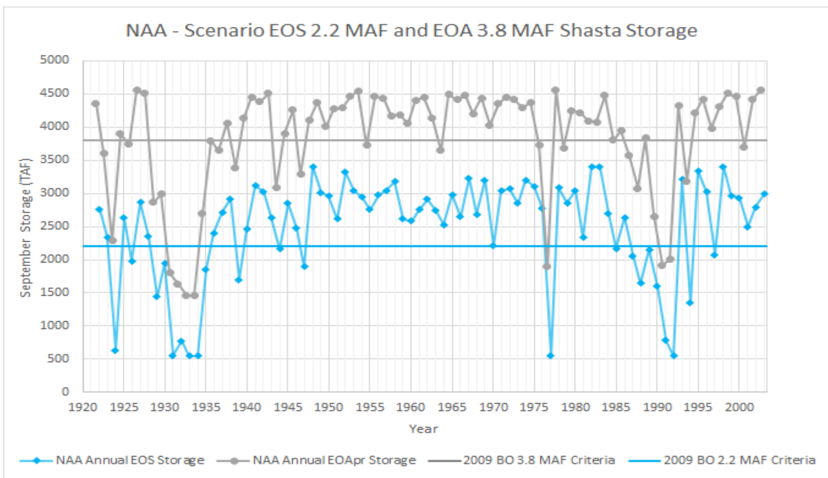


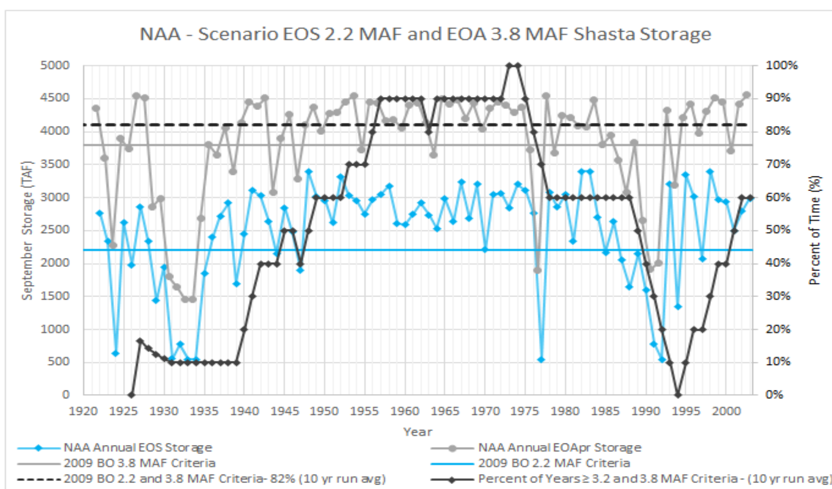




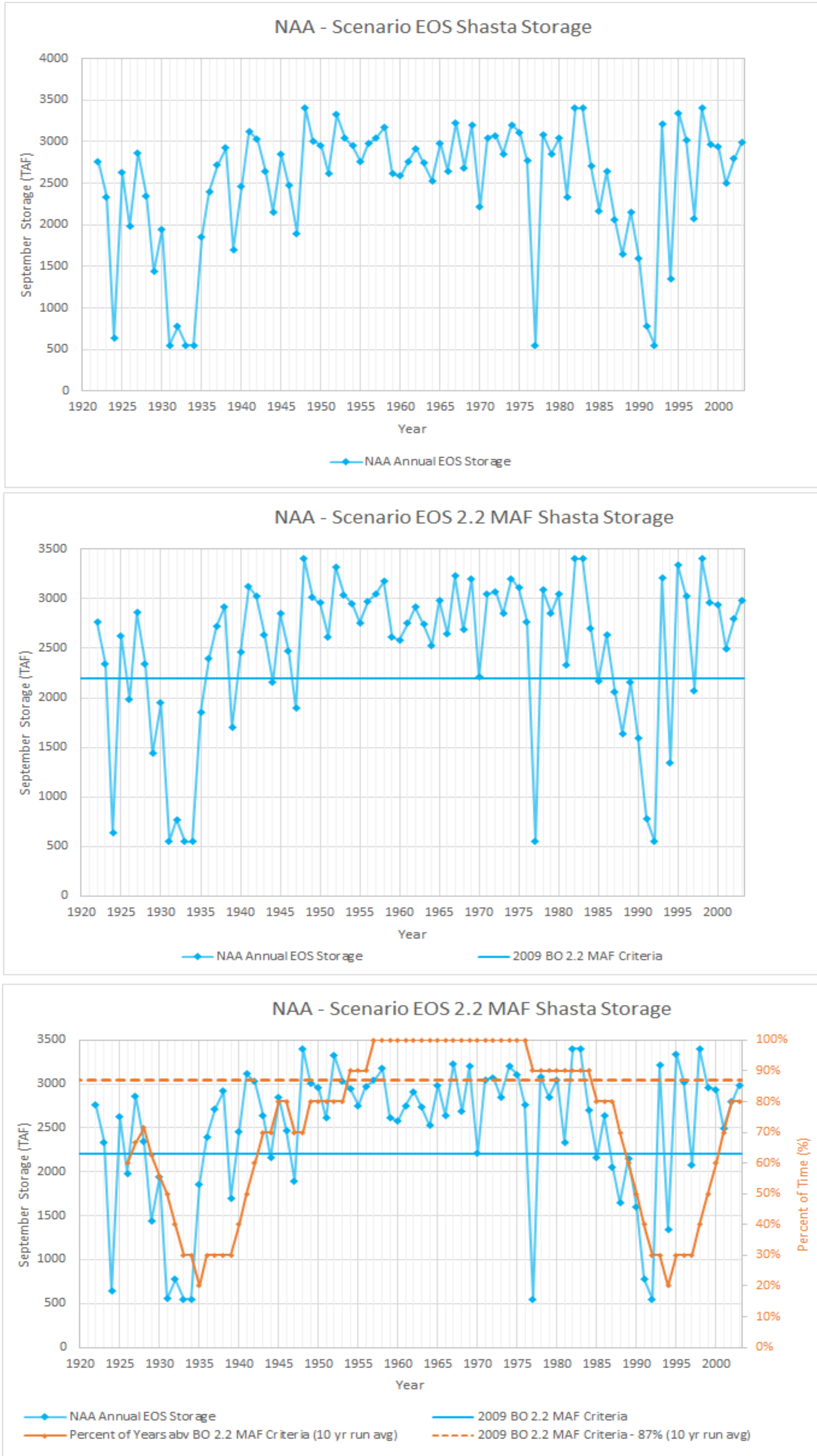
Appendix B – Figure 1. Shasta Reservoir EOS Storage Compared to the 2009 BO RPA 40% Performance Criteria for Minimum EOS Storage of 3.2 MAF. Time Series of Alternative 4A H3 EOS Storage (top), 3.2 MAF Line (middle), and the 10-year Running Average Criteria (bottom plot, right axis) (Criteria are Never Met).







Appendix B – Figure 2. Shasta Reservoir EOS and EO April Storage Compared to the 2009 BO RPA 82% Performance Criteria for Minimum EOS Storage of 2.2 MAF and Subsequent EO April Storage of 3.8 MAF. Time Series of Alternative 4A H3 EOS storage and EO April Storage (top), 2.2 MAF and 3.8 MAF Line (middle), and the 10-year Running Average Criteria (bottom plot, right axis) (Criteria are Not Met in Most Years).



Appendix B – Figure 3. Shasta Reservoir EOS Storage Compared to the 2009 BO RPA 87% Performance Criteria for Minimum EOS Storage of 2.2 MAF. Time Series of Alternative 4A H3 EOS storage (top), 2.2 MAF Line (middle), and the 10-year Running Average Criteria (bottom plot, right axis) (Criteria are Not Met in >50% of Years).

Appendix C
Shasta Reservoir and Sacramento River Water Temperature

Relationship between Shasta Reservoir Storage and Sacramento River Temperatures

Historical data show that, in the years when Shasta Reservoir does not completely fill, an empirical relationship exists between EOS storage, spring storage, and the water temperature that can be attained downstream of Keswick Reservoir during the summer (higher storage equals colder temperatures). The relationship between Shasta Reservoir May and September storage and Sacramento River temperatures is presented in Appendix C – Figure 1. The empirical relationships were generated using daily Shasta storage and Sacramento River temperature data from 5 stations below Keswick Dam (KWK, CCR, BSF, BND and RDB) downloaded from CDEC for 1995 through 2015. Using linear interpolation between the known station river miles, the number of miles of river below Keswick at or below 56 °F was calculated on a daily and average monthly basis. Years in which Shasta Reservoir completely filled in May (indicating high winter inflows) were removed from the analysis. The remaining years (below normal, dry and critically dry years) show a clear relationship between Shasta storage and the average number of miles of Sacramento River that are at or below 56 °F. Lower May and September Shasta storage directly correlates with fewer miles of river at or below 56 °F.

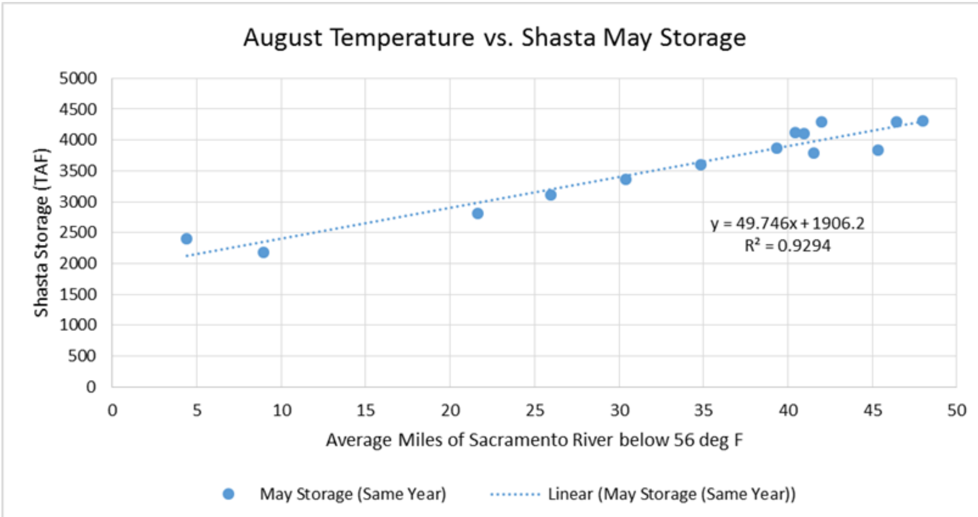
Sacramento River Water Temperatures at Keswick by Water Year Type

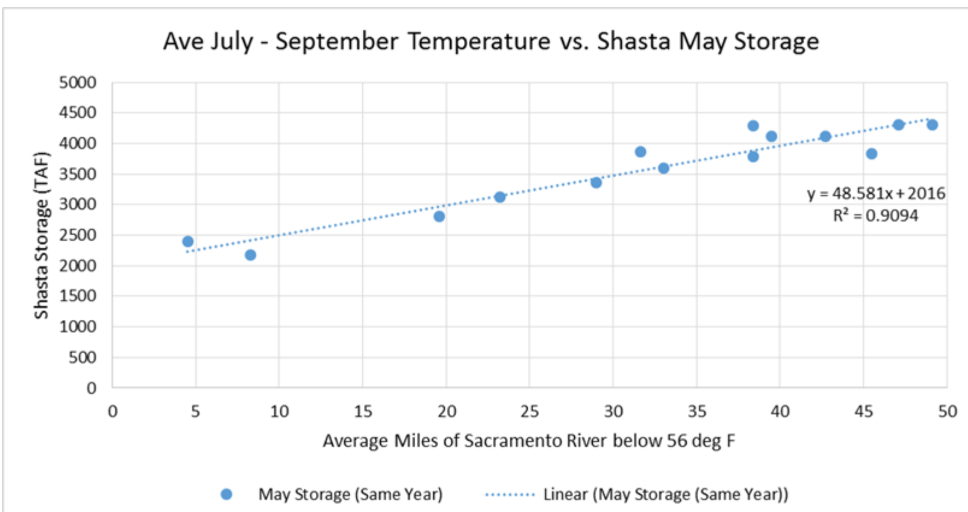
The WaterFix Alternative 4A H3 Scenario greatly increases water temperature in the Sacramento River compared to Existing Conditions. The temperature increases exceed the temperature suitability for winter-run Chinook salmon. Average water temperatures by hydrologic year type in the Sacramento River at Keswick were obtained from RDEIR/SDEIS Appendix B – Supplemental Modeling for New Alternatives, page B-376 and graphed in Appendix C –Figure 3 for California WaterFix Alternative 4A H3 and Existing Condition scenarios. Along with the average monthly temperatures by year type, the graph also shows the 56°F criterion set by the 2009 NMFS BO and the 2016 NMFS target temperature of 53°F at Keswick Dam set to meet the 7-day average daily maximum temperature of 55°F at the Above Clear Creek gage (CCR) based on NMFS guidance in 2016.

The plots show that Alternative 4A H3 has consistently higher temperatures than the Existing Condition scenario for almost all months and year types, although the difference is particularly pronounced for dry and critically dry years (Alternative 4A H3 is warmer by 5 to 7 °F for critically dry years, depending on the month). The Alternative 4A H3 scenario does not meet the 2016 NMFS target temperature of 56°F downstream of Keswick Dam for either July, August, or September for drier year types.

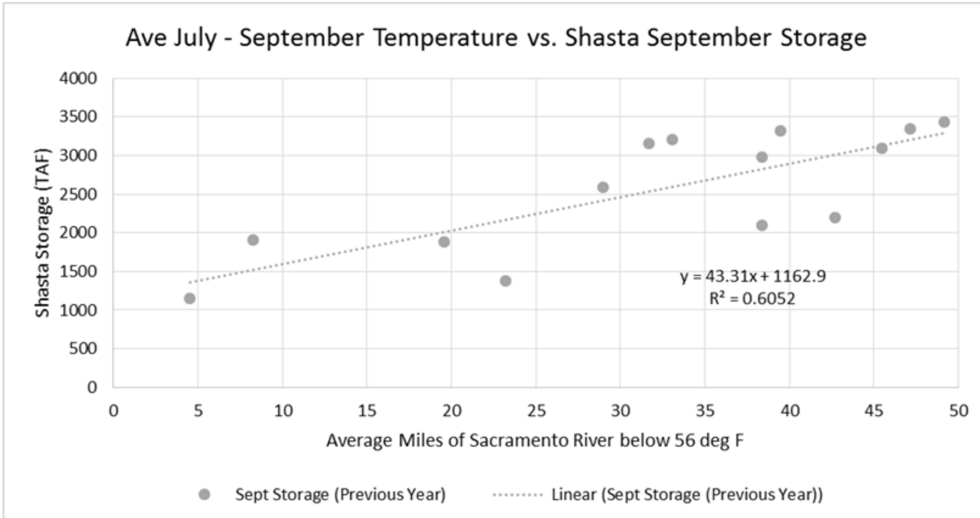
Sacramento River Water Temperature at Various Locations by Month

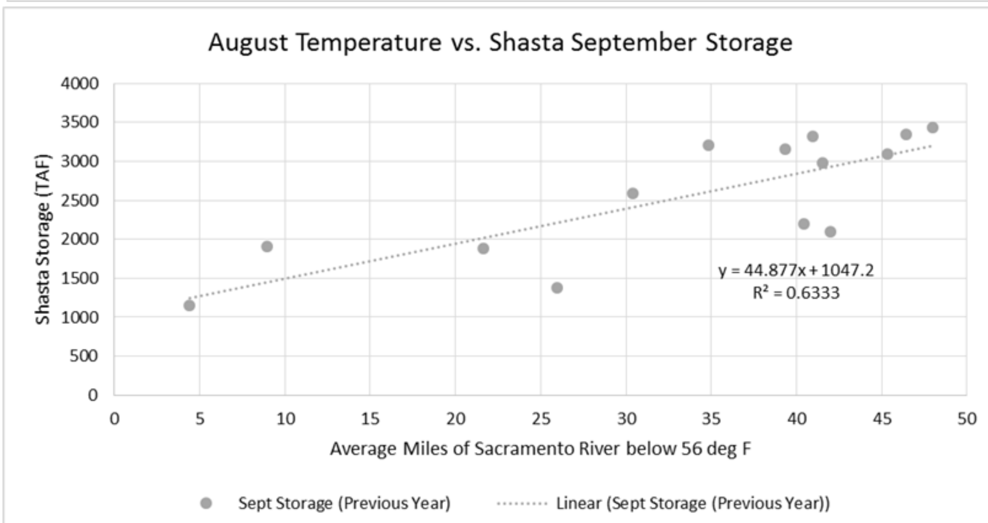
The WaterFix Alternative 4A H3 scenario does not comply with the 2009 BO RPA Action 1.2.1 temperature performance measure percentages (Appendix B – Table 1) over the 82 year period of the simulation. An analysis of Sacramento River temperatures at Keswick, Clear Creek, Balls Ferry, Jelly's Ferry and Bend Bridge for the Alternative 4A H3 scenario was carried out to determine the percentage of days during which the 56 °F target was met (temperatures equal to or less than 56 °F) for the months of June through September. The percentage of days when the temperature criteria was met compared to the 2009 BO RPA criteria is plotted in Appendix C – Figure 4. While we did not plot the 10-year running average, it is physically impossible for the 10-year running average to be in compliance through the 82 year period when the 82 year average is not in compliance as in Appendix C – Figure 4.





Appendix C – Figure 1. Historical Shasta May Storage (1995-2015* see text) vs. Sacramento River Temperature. (Top) Average miles of Sacramento River at or below 56°F for August vs. Shasta May storage. (Bottom) Average miles of Sacramento River at or below 56°F for July through August periods vs. Shasta May storage.





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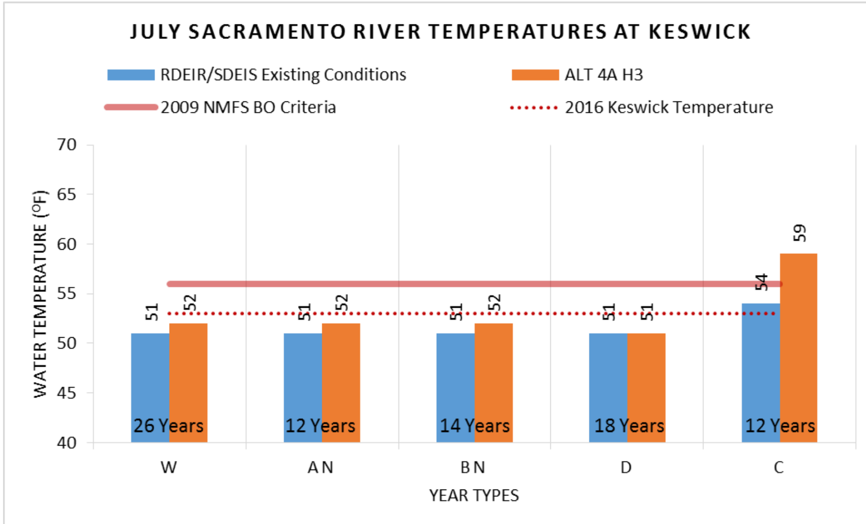
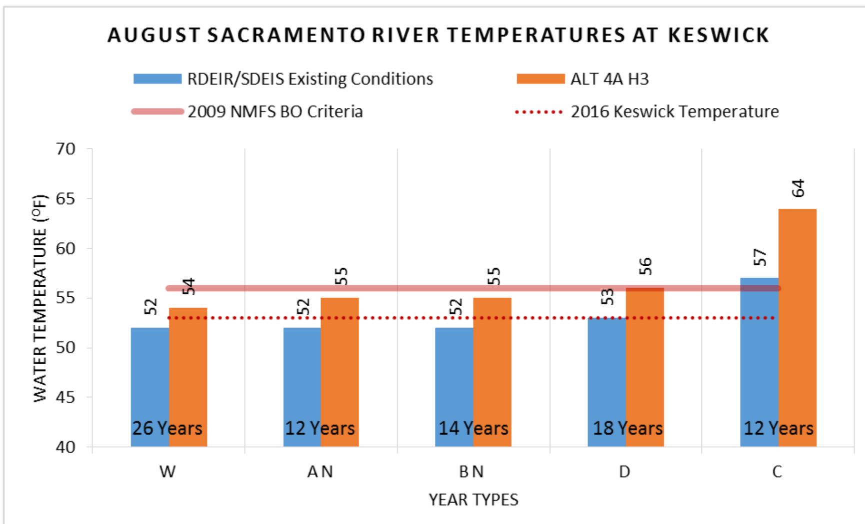
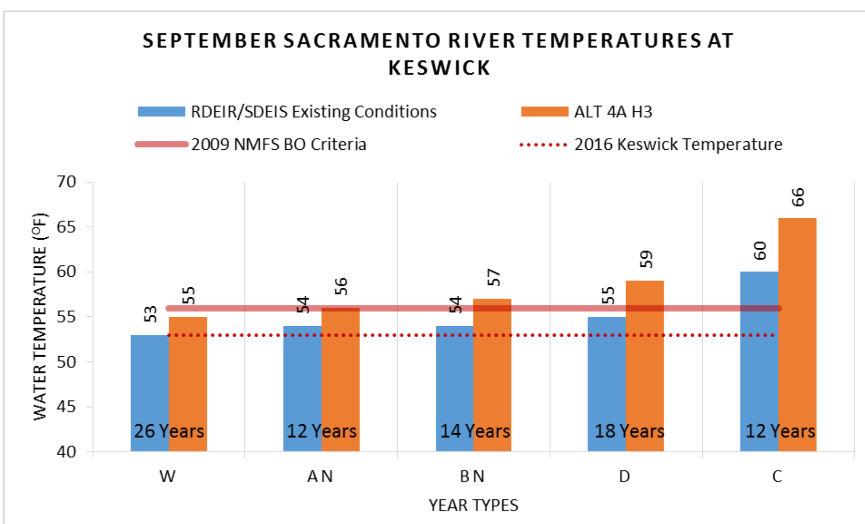
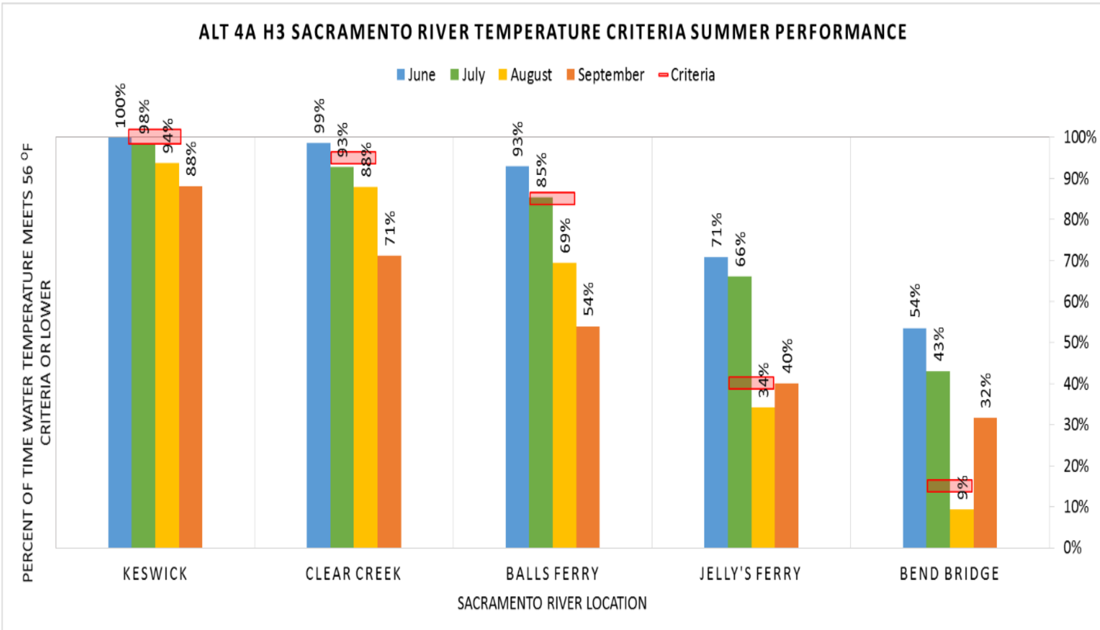






Appendix C - Figure 2. July, August, and September Monthly Average Water Temperatures at Keswick by Water Year Type for California WaterFix Existing Condition and Alternative 4A H3 Scenarios.



Appendix C – Figure 4. Sacramento River Temperature Summer Performance at Various Locations for Alternative 4A H3 (1922-2003) Compared to the 2009 BO Percentages (red bar).

RECLAMATION
Managing Water in the West

Folsom Lake
2005 Sedimentation Survey

U.S. Department of the Interior
Bureau of Reclamation
Technical Service Center
Denver, Colorado

January 2007
ARWA-202

REPORT DOCUMENTATION PAGE

Form Approved
OMB No. 0704-0188

1. REPORT DATE
January 2007

2. REPORT TYPE

3. DATES COVERED *(From – To)*

4. TITLE AND SUBTITLE

Folsom Lake
2005 Sedimentation Survey

5a. CONTRACT NUMBER

5b. GRANT NUMBER

5c. PROGRAM ELEMENT NUMBER

6. AUTHOR(S)

Ronald L. Ferrari

5d. PROJECT NUMBER

5e. TASK NUMBER

5f. WORK UNIT NUMBER

7. PERFORMING ORGANIZATION NAME(S) AND ADDRESS(ES)
Bureau of Reclamation, Technical Service Center, Denver CO 80225-0007

8. PERFORMING ORGANIZATION REPORT NUMBER

9. SPONSORING/MONITORING AGENCY NAME(S) AND ADDRESS(ES)
Bureau of Reclamation, Technical Service Center, Denver CO 80225-0007

10. SPONSOR/MONITOR'S ACRONYM(S)

11. SPONSOR/MONITOR'S REPORT NUMBER(S)

12. DISTRIBUTION/AVAILABILITY STATEMENT

13. SUPPLEMENTARY NOTES
Hard copy available at Bureau of Reclamation Technical Service Center, Denver, Colorado

14. ABSTRACT

The Bureau of Reclamation (Reclamation) surveyed Folsom Lake in the fall of 2005 via an interagency agreement with the U.S. Army Corps of Engineers (Corps of Engineers). The extensive surveys developed topographic imagery to study Folsom Dam, the wing and auxiliary dams, and the eight dikes that form Folsom Lake. This report presents the computed present storage-elevation relationship (area-capacity tables) developed from the 2005 surveys. Unless noted, all elevations in this report are tied to the project vertical datum in feet that was determined to be 2.34 feet less than the North American Vertical Datum of 1988 (NAVD88-05).

The underwater survey was conducted from September 9 through 22, 2005 between lake elevations 437 and 441 feet (project datum). The underwater survey used sonic depth recording equipment interfaced with a global positioning system (GPS) that gave continuous sounding positions throughout the underwater portions of the reservoir covered by the survey vessel. The above-water topography was developed by aerial photography under contract with Reclamation. The main body of the reservoir was flown on October 20, 2005 near reservoir elevation 430.2 (NAVD88-05) and high accuracy data was flown on 10/31/2005 near reservoir elevation 427.1 (NAVD88-05). The 2005 topographic maps were developed from the combined data sets. All digital topographic images for these studies were tied to vertical datum NAVD88-05. For purpose of computing updated area and capacity tables, the topographic elevations were shifted 2.34 feet to match the project vertical elevations. The bathymetric survey measured a minimum elevation of 190. Lake contour surface areas were developed from elevation 490.0 and below.

As of September 2005, at water surface elevation 466.0, the surface area was 11,140 acres with a total capacity of 966,823 acre-feet. Since initial closure in 1955, about 43,407 acre-feet of volume loss was measured by the 2005 survey.

15. SUBJECT TERMS
reservoir area and capacity/ sedimentation/ reservoir surveys/ sonar/ sediment distribution/ contour area/ reservoir area/ sedimentation survey/ global positioning system/ lake/ multibeam/ bathymetry

16. SECURITY CLASSIFICATION OF:			17. LIMITATION OF ABSTRACT	18. NUMBER OF PAGES	19a. NAME OF RESPONSIBLE PERSON
a. REPORT	b. ABSTRACT	a. THIS PAGE			19b. TELEPHONE NUMBER *(Include area code)*

Folsom Lake
2005 Sedimentation Survey

prepared by

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Bureau of Reclamation
Technical Service Center
Sedimentation and River Hydraulics Group
Denver, Colorado

January 2007
ARWA-202

Mission Statements

The mission of the Department of the Interior is to protect and provide access to our Nation's natural and cultural heritage and honor our trust responsibilities to Indian Tribes and our commitments to island communities.

The mission of the Bureau of Reclamation is to manage, develop, and protect water and related resources in an environmentally and economically sound manner.

Acknowledgments

Reclamation's Sedimentation and River Hydraulics Group (Sedimentation Group) of the Technical Service Center (TSC) prepared and published this report. Corps of Engineers provided funding for conducting the detailed surveys of the Folsom Lake study area and for the analysis presented in this report. Terri Reaves, Chief of Reclamation's Mid-Pacific Region Surveys and Mapping Branch coordinated the collection and analysis of these extensive data sets. Ronald Ferrari of the Sedimentation Group, with assistance from staff of the Surveys and Mapping Branch, conducted the underwater data collection. The Surveys and Mapping Branch completed the topographic mapping of the merged data for these surveys. Ron Ferrari completed this report and the data processing needed to generate the reservoir only topography and area-capacity tables. Stacey Smith of the Mid-Pacific Region assisted in the computation of the operational values. This report includes comments, from December 2006 review, from the Corps of Engineers and Mid-Pacific Region. Kent Collins of the Sedimentation Group performed the technical peer review of this documentation.

Reclamation Report

This report was produced by the Bureau of Reclamation's Sedimentation and River Hydraulics Group (Mail Code 86-68540), PO Box 25007, Denver, Colorado 80225-0007. **http://www.usbr.gov/pmts/sediment/**

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Folsom Lake 2005 Sedimentation Survey

Introduction

Folsom Dam and reservoir are located on the American River approximately two miles upstream from the city of Folsom and 20 miles northeast of the city of Sacramento, California, figure 1. The dam is located in Sacramento County with portions of the lake also in El Dorado and Placer Counties. The reservoir provides a water supply for irrigation, domestic, municipal, industrial, and power production purposes. The reservoir also provides flood protection and recreation. Releases provide water quality control in the Sacramento-San Joaquin delta, maintain fish runs in the American River below the dam, and help maintain navigation along the lower reaches of the Sacramento River. Folsom Dam was constructed by the Corps of Engineers in the late 1940's and transferred to Reclamation upon completion in a 1956 Memorandum of Understanding. A 1981 Memorandum of Understanding made the Corps of Engineers responsible for any studies necessary to determine the structural adequacy of the dam.

Figure 1 - Folsom Lake location map.

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Folsom Lake was created by the closure of Folsom Dam on February 25, 1955. Folsom Dam is a concrete structure flanked by two earth wing dams, Mormon Island Auxiliary Dam, and eight dikes with combined crest lengths of around 5 miles. The concrete portion is a 1,400 foot straight gravity structure with a maximum structural height[1] of 340.0 feet, a crest elevation of 480.5[2], and parapet wall elevation of 484.0 feet. The left wing dam is a zoned earthen embankment with a crest length of 2,100 feet and maximum height of 145 feet at crest elevation 480.5. The right wing dam is a zoned earthen embankment with a crest length of 6,700 feet and maximum height of 145 feet at crest elevation 480.5. Mormon Island Auxiliary Dam is located east of the main dam and is a zoned earthen embankment with a crest length of 4,820 feet and maximum height of 110 feet at crest elevation 480.5. There are eight dikes located around the reservoir rim with crest elevations of 480.5 and ranging in maximum height from 15 to 105 feet and length from 740 to 2,060 feet.

The spillway is a gate-controlled overflow spillway divided by piers into eight equal sections and is located in the center gravity section of the dam at crest elevation 418.0. Flow is controlled by 42-foot wide radial gates that are 50-foot high in the service spillway area and 53-foot high in the emergency spillway area. The service spillway consists of the western-most five gates that discharge into a stilling basin in the original river channel below. The eastern most three gates make up the emergency spillway and discharge into a flip bucket that projects the discharge. The discharge capacity is 567,000 cubic feet per second (cfs); however, the levee system that protects the city of Sacramento downstream only allows a safe channel capacity of 115,000 cfs.

The outlet works consists of eight 5-foot wide by 9-foot high gated sluice outlets located in the overflow spillway section of the dam. Four river outlet conduits are at invert elevation 280.0 and four are at invert elevation 210.0. The maximum discharge capacity, into the spillway stilling basin, is 27,800 cfs at reservoir elevation 466.0.

Three 15.5-foot-diameter penstocks are located in the right non-overflow section of the concrete dam that carry water approximately 500 feet downstream to three generating units at the Folsom Power plant. The powerplant is the primary source of normal releases into the American River.

The drainage area above Folsom Dam is approximately 1,861 square miles and 1,020 square miles are considered sediment contributing. The total drainage area

---

[1] The definition of such terms as "crest length," "structural height," etc. may be found in manuals such as Reclamation's *Design of Small Dams and* ASCE's *Nomenclature for Hydraulics.*

[2] Elevations in feet. All elevations based on the original project datum established during construction that was found to be 2.34 feet lower than NAVD88-2005 (NAVD88-05). Unless noted, all listed elevations in feet and in project vertical datum.

value is from the USGS water resource data (USGS, 1990). The non-sediment contributing area includes the normal surface area of Folsom Lake and drainage areas above the numerous dams located within the drainage basin above Folsom Dam. The drainage basin elevations range from approximately 10,400 feet at the headwaters to normal reservoir surface elevation 466.0. The reservoir is around 28.0 miles in length and around 0.6 miles in width.

Summary and Conclusions

This Reclamation report presents the 2005 results of the survey of Folsom Lake. The primary objectives of the surveys were to gather data needed to:

- develop lake topography
- develop detailed topography of the entrapment structures
- compute area-capacity relationships
- estimate storage depletion, by sediment deposition, since dam closure

Reclamation was directed to survey Folsom Lake in the fall of 2005 by an interagency agreement with the Corps of Engineers. The extensive surveys developed detailed topography to be used for studies of Folsom Dam, Mormon Island Auxiliary Dam, the wing dams, and the eight dikes that form Folsom Lake. Aerial surveys covered upstream and downstream of the entrapment structures and extended up the north and south arms of the American River. Combined aerial and bathymetric survey data were used to develop the 2005 area and capacity tables of Folsom Lake formed by these entrapment structures.

The underwater (bathymetric) survey was conducted from September 9 through 22, 2005 between lake elevations 437 and 441 feet (project datum). The bathymetric survey used sonic depth recording equipment interfaced with a real-time kinematic (RTK) global positioning system (GPS) capable of determining sounding locations within the reservoir. The system continuously recorded depth and horizontal coordinates of the survey boat as it navigated along grid lines covering Folsom Lake. The positioning system provided information to allow the boat operator to maintain a course along these grid lines. The reservoir's water surface elevations (project datum), recorded by the Reclamation reservoir gauge during the time of collection, were used to convert the sonic depth measurements to reservoir bottom elevations. These gauge elevations are tied to the project vertical datum that was found to be 2.34 feet lower than NAVD88-05. All area and capacity computations within this report are tied to the project vertical datum.

The above-water topography was developed by aerial photography under contract with Reclamation. The main body of the reservoir was flown on October 20, 2005 near reservoir elevation 430.2 (NAVD88-05) and high accuracy data was flown on October 31, 2005 near reservoir elevation 427.1 (NAVD88-05). All digital topographic images for these surveys were tied to vertical datum NAVD88-05.

The 2005 Folsom Lake topography for this report is a combination of the 2005 aerial and the shifted underwater data sets with elevations tied to NAVD88-05. For purpose of computing updated area and capacity tables, these topographic elevations were reduced 2.34 feet for the measured surface areas to match the project datum elevations. Since all past and present reservoir operations are tied to the project vertical datum, all elevations and resulting values were shifted to match the project datum elevations. Unless noted, all elevations in this report are tied to the project vertical datum in feet.

In September 2005, Reclamation and Corps of Engineers, under the direction of the National Geodetic Survey, established an extensive geodetic control network for the entire project area in North American Datum of 1983 (NAD83) and NAVD88-05. This control network was established prior to all the photogrammetric work with the horizontal control in California state plane, zone 2, in NAD83. This control network was established after the bathymetric survey was conducted and was used to adjust the processed bathymetric data to match the aerial survey's horizontal and vertical datums.

A computer graphics program generated the 2005 reservoir surface areas at predetermined contour intervals from these combined data sets. The 2005 area and capacity tables were generated by a computer program that used the measured contour surface areas and a curve-fitting technique to compute area and capacity at prescribed project datum elevation increments (Bureau of Reclamation, 1985).

Tables 1 and 2 contain summaries of the Folsom Lake and watershed characteristics for the 2005 survey. The 2005 survey determined that the reservoir has a total storage capacity of 966,823 acre-feet and a surface area of 11,140 acres at joint use reservoir water surface elevation 466.0. Since initial closure in 1955, about 43,407 acre-feet of volume loss was measured by the 2005 survey.

Control Survey Data Information

Reclamation's Mid-Pacific Regional Surveys and Mapping Branch provided the control network information used by the bathymetric survey crew. The base station was set over marker "WD48," located on the east wing dam, and was used throughout the duration of the survey, figure 2. The data collection was in California state plane coordinates, zone 2, NAD83 and the vertical was tied to National Geodetic Vertical Datum of 1929 (NGVD29).

Figure 2- RTK GPS base station, WD48.

In September of 2005, Reclamation and Corps of Engineers, under the direction of the National Geodetic Survey, surveyed an extensive geodetic control network for the entire project area in NAD83 and NAVD88-05. This control network was used for all of the photogrammetric work that followed, but was established after the bathymetric survey was conducted. The results of the 2005 geodetic control survey required a shift of the bathymetric data in the vertical and a slight shift in the horizontal coordinates to match the aerial survey data's horizontal (NAV83) and vertical datums (NAVD88-05). The bathymetric data was tied to base station "WD48" and the following shifts were applied.

	NAD83/NGVD29 (project elevation)	NAD83/NAVD88-05	Difference
North	2,019,300.82	2,019,301.29	(+) 0.47
West	6,804,655.01	6,804,654.40	(-) 0.61
Elevation:	481.04	483.38	(+) 2.34

Reservoir Operations

Folsom Dam operates to provide regulated diversion and downstream flows from the American River. The September 2005 capacity table shows 1,074,207 acre-feet of total storage below the maximum water surface elevation 475.4 feet. The 2005 survey measured a minimum lake bottom near elevation 190. Since all past and present operations are tied to the project vertical datum, all elevations and resulting values in NAVD88-05 were shifted to match. The following values are from the September 2005 capacity table:

- 107,384 acre-feet of surcharge between elevation 466.0 and 475.4 feet.
- 398,724 acre-feet of joint use between elevation 425.8 and 466.0 feet.
- 484,706 acre-foot of active storage between elevation 327.0 and 425.8 feet.
- 83,387 acre-foot of inactive storage between elevation 205.5 and 327.0 feet.
- 6 acre-foot of dead storage below 205.5 feet.

Folsom Lake computed annual inflow and reservoir stage available records are listed by water year on table 1 for the operation period 1955 through 2005. The inflow values were computed by the Mid-Pacific Regional office and show annual fluctuation with a computed average inflow of 2,787,400 acre-feet per year. The maximum reservoir elevation was 467.2 recorded during water year 1963 with a minimum elevation of 347.6 recorded during water year 1978.

Hydrographic Survey Equipment and Method

The hydrographic survey equipment was mounted in the cabin of a 24-foot trihull aluminum vessel equipped with twin in-board motors, figure 3. The hydrographic system included a GPS receiver with a built-in radio, a depth sounder, a helmsman display for navigation, a computer, and hydrographic system software for collecting the underwater data. An on-board generator supplied power to all the equipment. The shore equipment included a second GPS receiver with an external radio. The GPS receiver and antenna were mounted on a survey tripod over a known datum point and a 12-volt battery provided the power for the shore unit.

The Sedimentation and River Hydraulics Group uses RTK GPS with the major benefit being precise heights measured in real time to monitor water surface elevation changes. The basic output from a RTK receiver are precise 3D coordinates in latitude, longitude, and height with accuracies on the order of 2 centimeters horizontally and 3 centimeters vertically. The output is on the GPS datum of WGS-84 that the hydrographic collection software converted into California’s state plane, zone 2, coordinates in NAD83. The RTK GPS system

Figure 3 - Survey vessel with mounted instrumentation on Jackson Lake in Wyoming.

employs two receivers that track the same satellites simultaneously just like with differential GPS.

In 2001, the Sedimentation and River Hydraulics Group began utilizing an integrated multibeam hydrographic survey system. The system consists of a single transducer mounted on the center bow or forward portion of the boat. From the single transducer a fan array of narrow beams generate a detailed cross section of bottom geometry as the survey vessel passes over the areas to be mapped. The system transmits 80 separate 1-1/2 degree slant beams resulting in a 120-degree swath from the transducer. The 200 kHz high-resolution multibeam echosounder system measured the relative water depth across the wide swath perpendicular to the vessel's track. Figure 4 illuminates the swath of the sea floor that is about 3.5 times as wide as the water depth below the transducer.

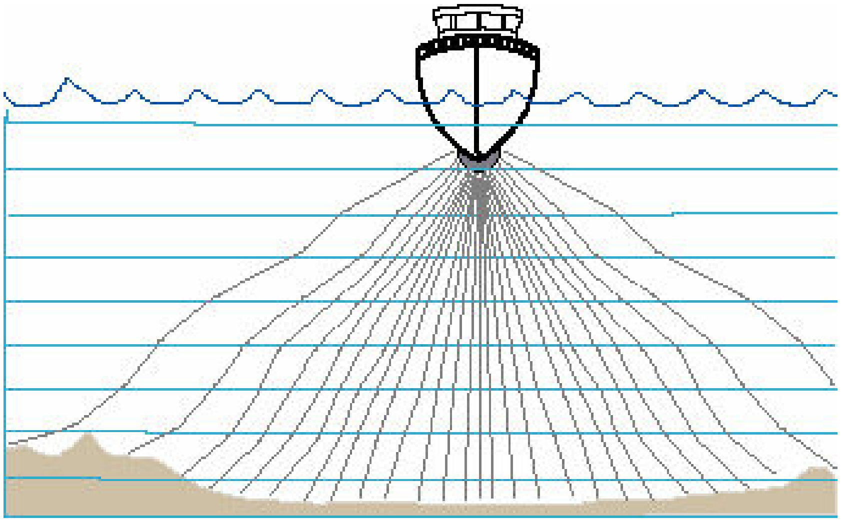

Figure 4 - Multibeam collection system.

The multibeam system is composed of several instruments that are all in constant communication with a central on-board notebook computer. The components include the RTK GPS for positioning; a motion reference unit to measure the heave, pitch, and roll of the survey vessel; a gyro to measure the yaw or vessel attitude; and a velocity meter to measure the speed of sound of the reservoir water column. With the proper calibration, the data processing software utilizes all the incoming information to provide an accurate detailed x, y, z data set of the lake bottom.

The Folsom Lake bathymetric survey collection was conducted from September 9 through September 22 of 2005 between water surface elevation 437 and 441 (project datum). The survey was run using the multibeam instrumentation described above. The survey system software continuously recorded reservoir depths and horizontal coordinates as the survey vessel moved across close-spaced grid lines covering the reservoir area. Most of the transects (grid lines) were run along the original river alignment of the reservoir where the multibeam swaths overlapped each other. In the shallower depths, around thirty feet and less, the swaths did not overlap. The multibeam system could have provided full bottom coverage not covered by these swaths, but time, budget, and access prevented this in the shallow water portions of the reservoir. Due to the cost and sensitivity of the multibeam transducer, the collection crew generally avoids collection in depths shallower than 10 feet. The loss of these additional data points did not have a significant impact on the area computations since they occurred in shallower areas of the reservoir where the bottom topography was generally flatter in nature.

The 2005 collection of bathymetric data did not include single beam data in the shallow, less than 10 foot, areas of the reservoir. It was anticipated that the lake would be low enough during aerial collection to obtain enough overlap between the aerial and multibeam data sets, but this did not occur throughout the reservoir. Besides the shallow flat areas of the reservoirs some of the additional areas not covered included the upstream arms and coves of the reservoir. For these missed areas the contours between the surveyed data were interpolated using contouring software. These contours should not be considered reliable and would not meet most accuracy standards. To preserve the integrity of the data sets, interpolated points were not added to the shallow water areas. Also, these areas were small relative to the total reservoir area and would not have had a significant effect on the overall surface area computations.

The first part of the analysis started with the processing of all the collected raw profile data files of the bottom. This included application of all necessary correction information that was collected, such as vessel location and the roll, pitch, and yaw effects on the survey vessel. Other corrections included application of the field measured sound velocity of the reservoir water column and then conversion of all corrected depth data to elevations. All elevations in the final processing were tied to the Reclamation measured water surface gauge elevations at the time of collection. During map processing, these bottom elevations were converted to NAVD88-05 datum by adding 2.34 feet. The geodetic survey also measured a slight shift in the horizontal coordinates (+) 0.47 feet north and (-) 0.61 feet west that was also applied.

Due to the massive amount of multibeam data collected, procedures within the collection and analysis software logically filtered data points without adversely affecting the results were utilized. Filtering mainly occurred in the flatter areas of the reservoir where the additional survey points were not necessary to map the bottom details of the reservoir. Quality control and assurance of the data sets were accomplished by conducting field calibration as required by the multibeam system and by collecting velocity profile data for the areas being surveyed. The processing of the multibeam data was conducted by Reclamation's Sedimentation Group. The processed data in an x,y,z format was forwarded to the Surveys and Mapping Branch for topographic development.

Reservoir Area and Capacity

Topography Development

Survey and Mapping Branch Processing

The entire topography of 2005 Folsom Lake and the surrounding area were developed by the Reclamation's Mid-Pacific Region Surveys and Mapping Branch. This included overseeing the contracting of the aerial collection and quality control. The aerial mapping included the upstream arms of the north and south forks of the American River and the high accuracy mapping of the dams and dikes that enclose the reservoir. Using standard photogrammetric processes the film diapositives were used for aerial triangulation and subsequent 3D data collection. Using AutoCAD, breaklines along with random and regularly gridded points were also compiled to create surface models for contour generation. For the bathymetric data, the processed x,y,z data points were shifted to match the aerial horizontal and vertical control datums which were in NAD83 and NAVD88-05 respectively. The bathymetry contours were developed using a hardclip boundary around the underwater data that was developed from the aerial data, contour elevation 430.0 (NAVD88-05). Due to the large data sets, the final contours were broken up into seven blocks or drawing files. The area blocks also included digital terrain model (DTM) files that contained the surface data in the form of breaklines along with the random and regular gridded points. These data files were forwarded to the Sedimentation Group for area and capacity computations and sediment inflow calculations. All data was tied to California state plane, zone 2, in NAD83 and the elevations tied to NAVD88-05. Additional files included the full orthorectified photos of the project area. Additional information on these coverage files and methods of processing are listed in the metadata file located in the appendix.

Sedimentation Group Processing

The Sedimentation Group uses ARCGIS (ESRI, 2006) to process data for developing reservoir topography and computing the resulting surface areas and volumes. To accomplish this, the 2005 Folsom Lake study area's processed x,y,z data points were combined into one data set. This included the nearly nine million bathymetric data points that were shifted to match the aerial data datums. These bathymetric points were combined with the aerial data points, located within the DTM files, for developing the study area contours.

The first step was to enclose all the combined data points within a hardclip polygon so that during contour development all interpretations would remain within the study area. The contours within this hardclip were developed from the combined aerial and underwater data sets using the triangular irregular network (TIN) surface-modeling package within ARCGIS. A TIN is a set of adjacent non-overlapping triangles computed from irregularly spaced points with x,y coordinates and z values. TIN was designed to deal with continuous data such as elevations. The TIN software uses a method known as Delaunay's criteria for triangulation where triangles are formed among all data points within the polygon clip. The method requires that a circle drawn through the three nodes of a triangle will contain no other point, meaning that sample points are connected to their nearest neighbors to form triangles using all collected data. This method preserves all collected survey points. Elevation contours were interpolated along the triangle elements using the surface contouring option within ARCGIS.

The aerial data of the dam and dikes is a very detailed set of points of the entire structures that are located beyond or downstream of the actual reservoir area. For the purpose of computing the surface area and capacity of the reservoir area only, a hardclip was developed to enclose the data points within the reservoir area only. This was accomplished by using the elevation 500.0 (NAVD88-05) contour developed from the entire study area survey points as described above. This contour is above the top of the dam, but was chosen for developing the updated reservoir surface areas since there have been discussions of possibly raising the existing structures and resulting reservoir levels. Using ARCGIS edit tools, the elevation 500.0 (NAVD88-05) contour was enclosed along the existing dike and dams by overlaying this contour onto the orthorectified photos. Once this polygon was developed and enclosed, elevation 500.0 (NAVD88-05) was assigned for the purpose of developing the reservoir TIN and resulting contours.

Within this reservoir area elevation 500.0 (NAVD88-05) hardclip, a TIN was developed for the Folsom Lake reservoir area. From this TIN, the 2005 surface areas and volumes were computed at one-foot increments from elevation 500.0 (NAVD88-05) and below. The contour data presented on these maps are tied to the vertical datum of NAVD88-05. All surface area and volume computations within this report were tied to the Folsom Lake project datum by shifting the elevations 2.34 feet lower than NAVD88-05. The vertical shift is necessary since past and present reservoir operations are tied to the project vertical datum.

Development of the 2005 Contours

Reclamation's Survey and Mapping Branch in Sacramento developed the 2005 contours of the Folsom Lake study area by combining the 2005 aerial and underwater data sets. These contours are presented on 204 detailed maps as illustrated on the index map, figure 5. The contours presented on these maps are tied to NAVD88-05 and are 2.34 feet higher than the project vertical datum and the horizontal coordinates are on the California State Plane, zone 2, in NAD83. Examples of maps are illustrated on figures 6 through 11. These maps are of the whole study area that extends upstream and downstream of the entrapment structures forming Folsom Lake. The metadata file in the appendix provides additional information on the creation of these maps.

Development of the 2005 Folsom Lake Surface Areas

The 2005 surface areas for Folsom Lake were computed at 1-foot increments from the TIN that covered the Folsom Lake area only. This TIN was developed within a hardclip area that included the existing elevation 500.0 (NAVD88-05) contour that was modified to run along the present alignment of the entrapment structures. These calculations were performed using the ARCGIS surface area and volume command that computes areas at user-specified elevations directly from the TIN and takes into consideration all regions of equal elevation. For the purpose of computing the 2005 area and capacity tables for this report, the measured surface area elevations in NAVD88-05 where shifted down by 2.34 feet to match the project elevations.

2005 Storage Capacity

The storage-elevation relationships based on the measured surface areas were developed using the area-capacity computer program ACAP (Bureau of Reclamation, 1985). For the purpose of this study, the measured 2005 survey areas at 3-foot increments from elevation 190.0.0 through 490.0 were used to compute the new area and capacity tables and were used as the control parameters for computing the 2005 Folsom Lake capacity. The ACAP program can compute the area and capacity at elevation increments 0.01- to 1.0-foot by linear interpolation between the given contour surface areas. The program begins by testing the initial capacity equation over successive intervals to ensure that the equation fits within an allowable error limit. The error limit was set at 0.000001 for Folsom Lake. The capacity equation is then used over the full range of

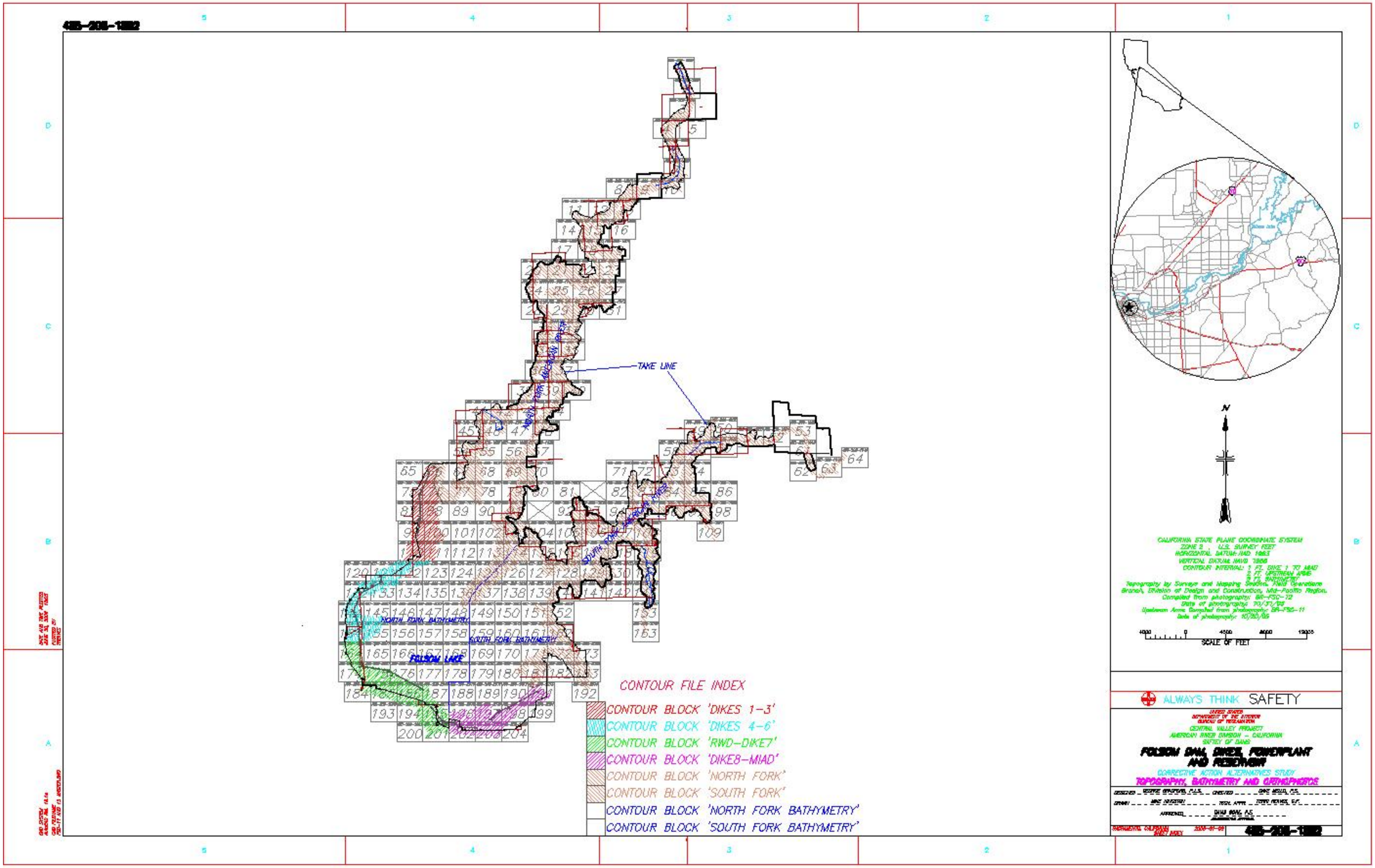

Figure 5 - Map index for the 2005 Folsom Lake survey.

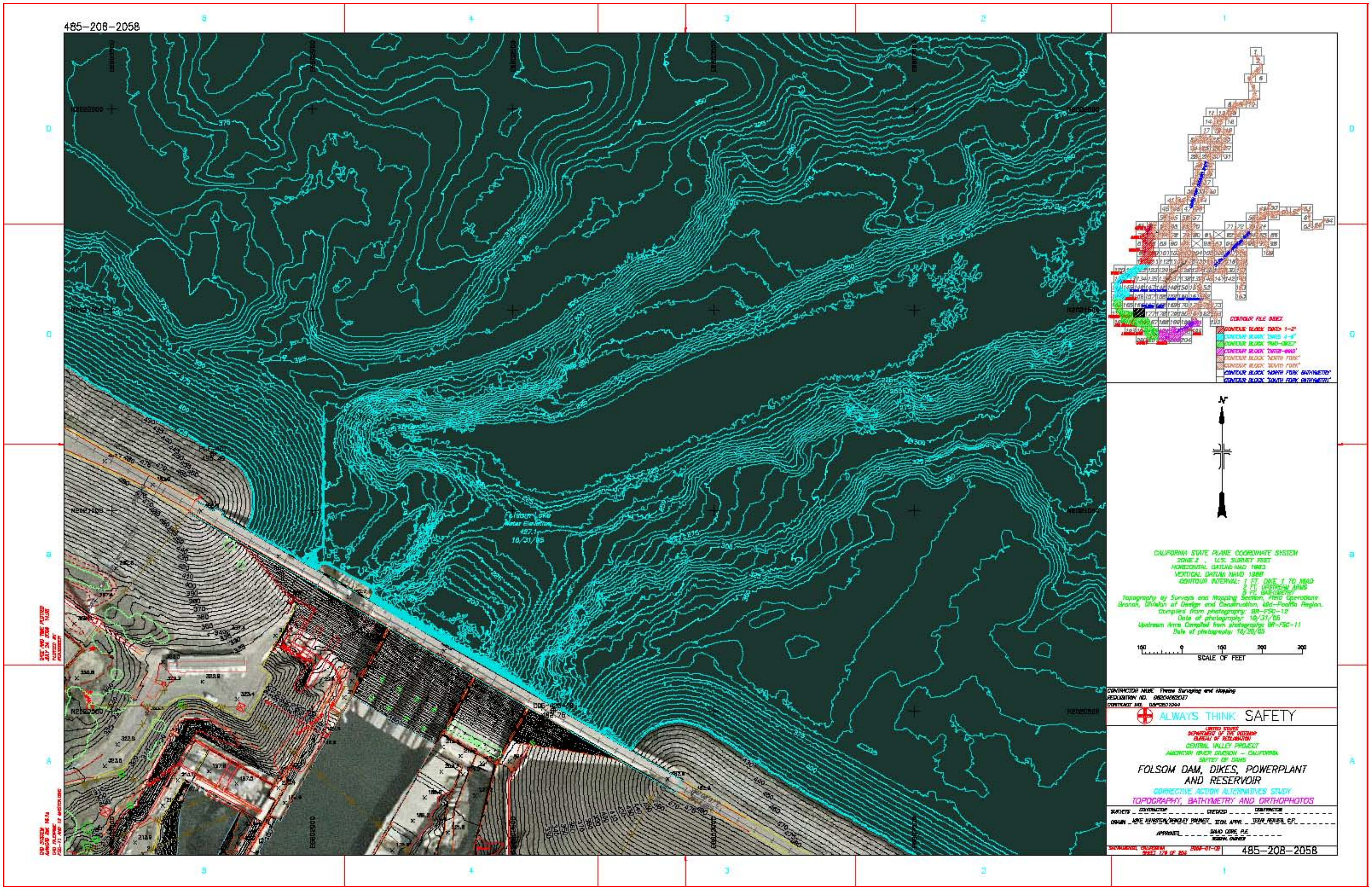

Figure 6 - Folsom Dam and lake topography, drawing 485-208-2058.

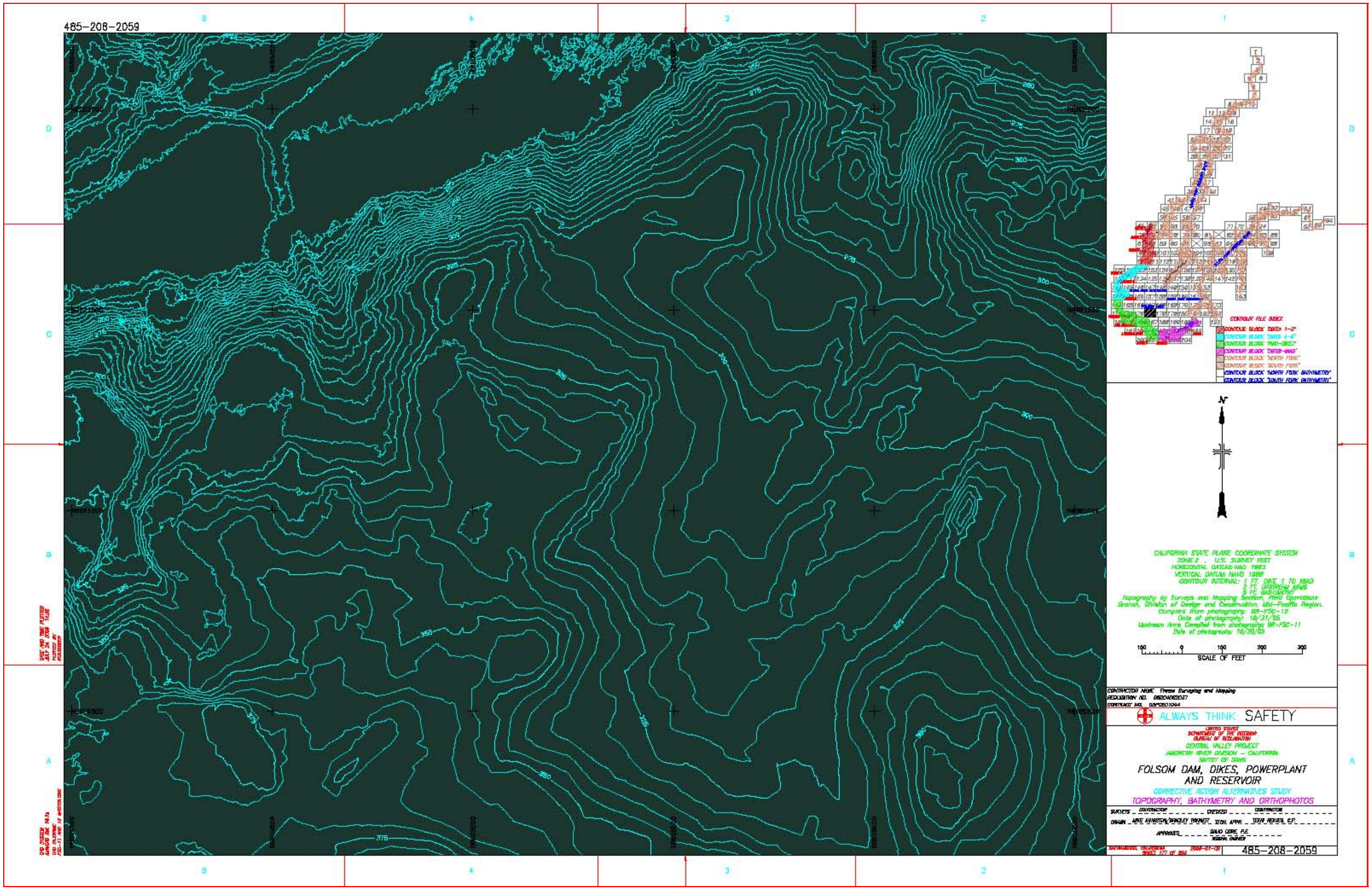

Figure 7 - Folsom Lake topography, drawing 485-208-2059.

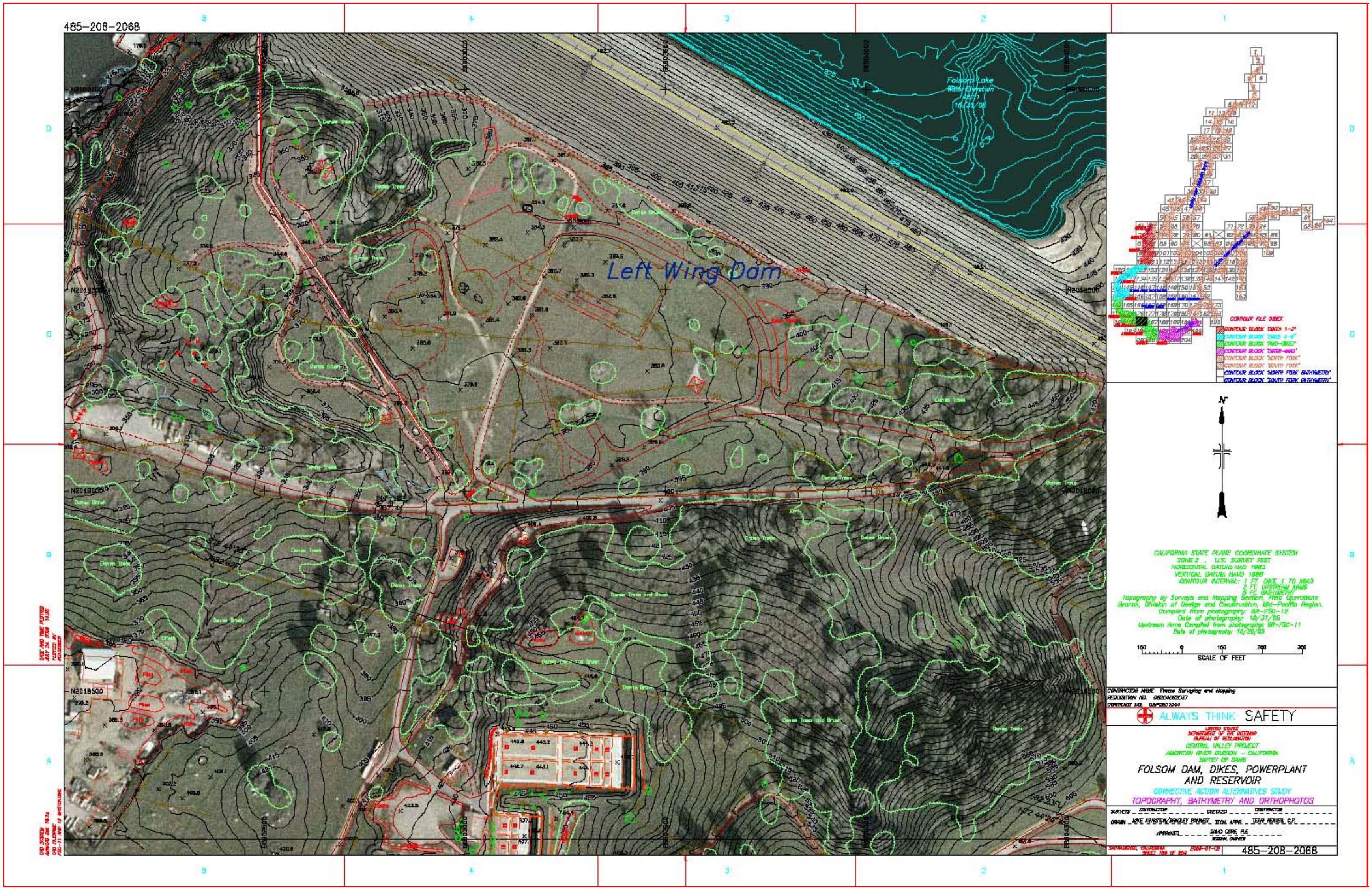

Figure 8 - Folsom Lake left wing dam and topography, drawing 485-208-2088.

Figure 9 - Folsom Lake topography, drawing 485-208-2089.

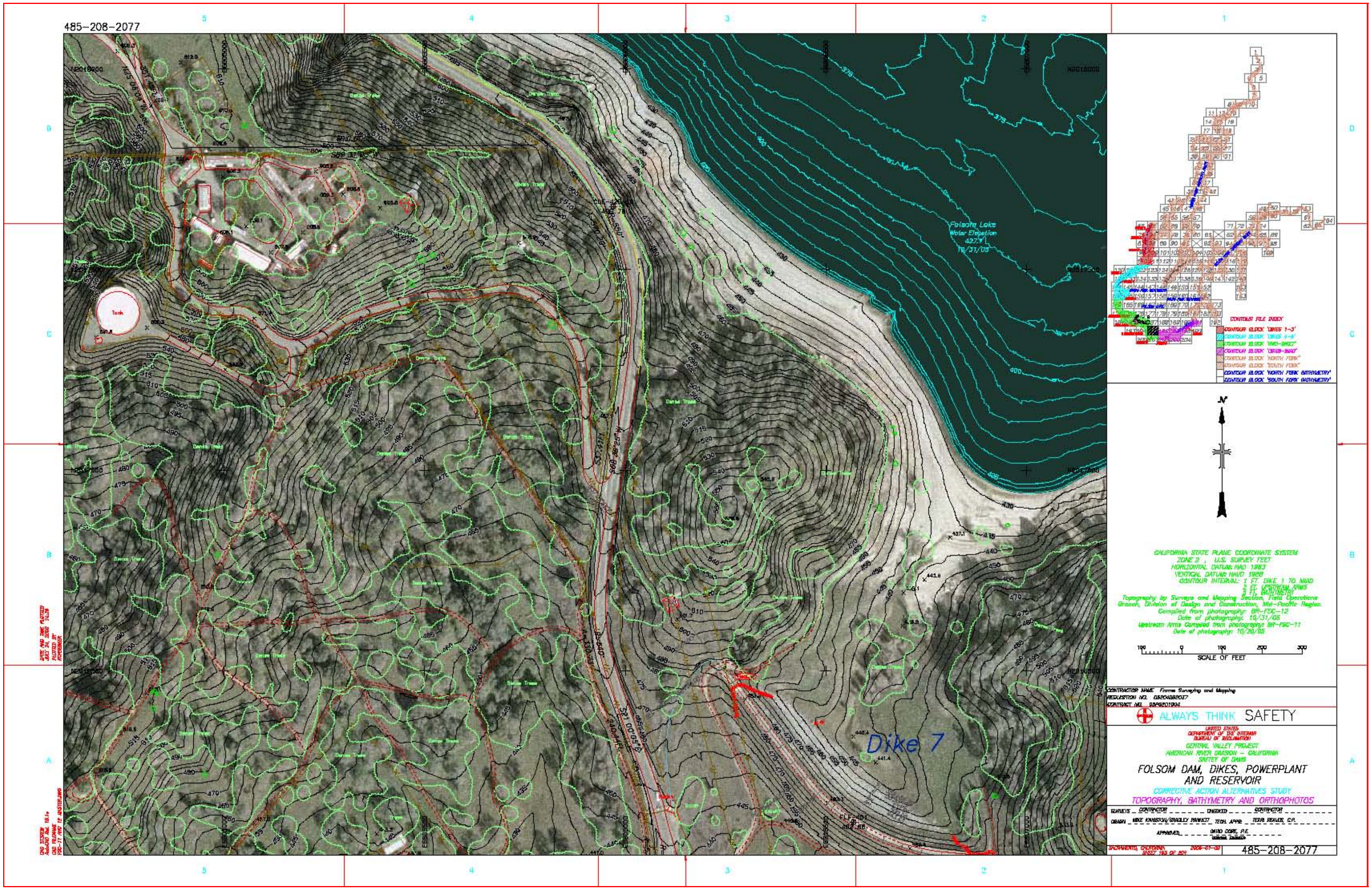

Figure 10 - Folsom Lake and Dike 7 topography, drawing 485-208-2077.

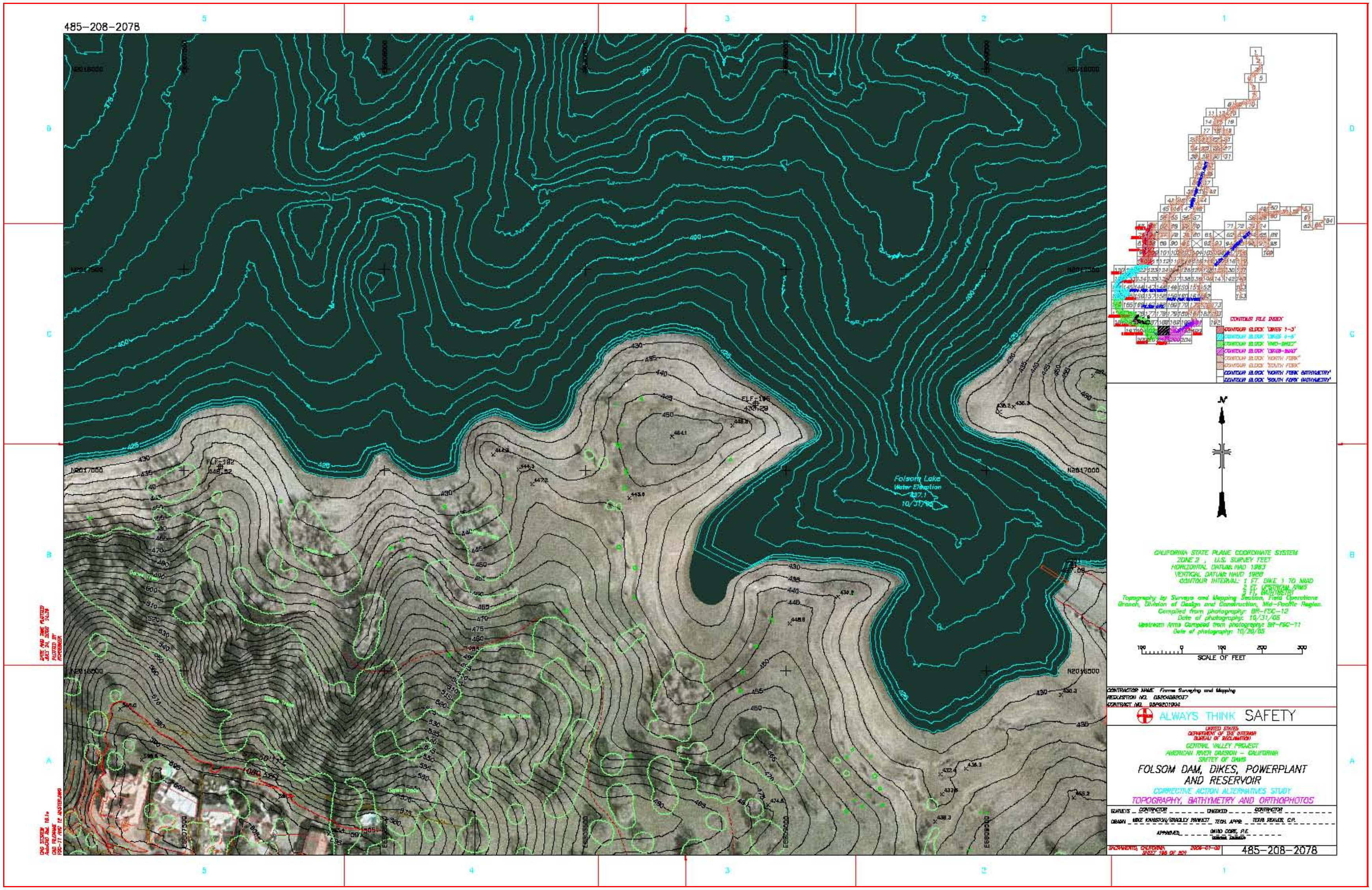

Figure 11 - Folsom Lake topography, drawing 485-208-2078.

intervals fitting within this allowable error limit. For the first interval at which the initial allowable error limit is exceeded, a new capacity equation (integrated from basic area curve over that interval) is utilized until it exceeds the error limit. Thus, the capacity curve is defined by a series of curves, each fitting a certain region of data. Differentiating the capacity equations, which are of second order polynomial form, derives final area equations:

$$y = a_1 + a_2x + a_3x^2$$

where: y = capacity
x = elevation above a reference base
a_1 = intercept
a_2 and a_3 = coefficients

Results of the Folsom Lake area and capacity computations are listed in a separate set of 2005 area and capacity tables and have been published for the 0.01, 0.1 and 1-foot elevation increments (Bureau of Reclamation 2006). A description of the computations and coefficients output from the ACAP program is included with these tables. The 1955, 1991, and 2005 area-capacity curves are listed on table 2 and plotted on figure 12. As of September 2005, at top of joint use elevation 466.0, the surface area was 11,140 acres with a total capacity of 966,823 acre-feet.

2005 Reservoir Analyses

Results of the 2005 Folsom Lake area and capacity computations are listed in table 1 and columns 8 and 9 of table 2. Columns 2 and 3 of table 2 list the 1955 or original area and capacity values and column 4 and 5 list the 1991 surface and area and capacity results for Folsom Lake. Column 6 and 10 of table 2 list the capacity differences between the original and 1991 and 2005 survey results. Figure 12 is a plot of the Folsom Lake surface area and capacity values for the three surveys and illustrates the differences between the surveys. The comparisons show that the total reservoir capacity in 2005 is 45,871 acre-feet less in volume from the original volume at maximum reservoir elevation 475.4.

Research into the original area and capacity data found 20-foot contour surface areas were used to compute the original volumes. For elevations 400 and below, the 20-foot contours were developed from U.S. Geologic Survey (USGS) river survey data collected in 1935-36. The 20-foot contours above elevation 400 came from 1940’s USGS quadrangle maps of the reservoir area. During the original planning of Folsom Lake, the 100 year estimated loss of total capacity of the reservoir below elevation 466.0 was 5.7 percent, a total of 58,000 acre-feet, or an annual average loss of 580 acre-feet. There was not any information on factors used to compute this sediment inflow value. The 2005 investigation found that the total drainage area into Folsom Lake is around 1,861 square miles and with several of the upstream reservoirs capturing sediment, it was computed that 1,020 square miles of the drainage area contributes sediment inflow into Folsom Lake. There are several reservoirs operating as diversion structures that were assumed had no effect on sediment retention. It is assumed the original 100 year estimate took into account the upper reservoir effects, but to what degree is not known.

The 2005 survey computed a loss of 43,407 acre-feet of storage during the first 50.5 years of operation below joint use reservoir elevation 466.0. It is unknown how much of this loss is due to differences in the detail of the surveys. The 1991 survey was a combination of an aerial survey conducted during low reservoir content (elevation 366), and a single beam bathymetric survey conducted at reservoir elevation 418. Parallel cross sections were run 200-feet apart to fill in the deeper reservoir area not covered by the aerial survey. The survey computed an average annual loss of 921.7 acre-feet, below elevation 466.0, over the first 36.1 years of operation by comparing the original recomputed volume with the 1991 computed volume. The 2005 survey computed an average annual loss of 703.6 acre-feet, below elevation 466.0, for the 14.4 years of operation since the 1991 survey. Even though the period is small between these surveys, the average annual loss of 703.6 acre-feet is a better representation for computing future losses since both surveys were of better detail than the original. There are many factors in the drainage basins that affect the annual sediment inflows, but it is recommended that the 703.6 acre-feet value be used as a basis for future prediction of reservoir losses.

It is the general conclusion that the difference between the original and 2005 surveys is due partially to sediment inflow, but the differences in the detail between the two surveys is also a factor. The 2005 survey is of greater detail and provides an accurate representation of the present reservoir volume as of September 2005.

RESERVOIR SEDIMENT
DATA SUMMARY

Folsom Lake
NAME OF RESERVOIR

1
DATA SHEET NO.

D	1. OWNER: Bureau of Reclamation [1]	2. STREAM: American River	3. STATE: California
A	4. SEC 24 TWP. 10 N RANGE 7 E	5. NEAREST P.O. Folsom	6. COUNTY: Sacramento
M	7. LAT 38 ° 42 ' 29 " LONG 121 ° 09 ' 22 "	8. TOP OF DAM ELEVATION: 480.5 [2]	9. SPILLWAY CREST EL. 418.0 [3]

RESERVOIR	10. STORAGE ALLOCATION	11. ELEVATION TOP OF POOL	12. ORIGINAL SURFACE AREA, AC-FT	13. ORIGINAL CAPACITY, AC-FT	14. GROSS STORAGE ACRE-FEET	15 DATE STORAGE BEGAN
	a. SURCHARGE	475.4 [4]	11,931	109,848	1,120,078	
	b. FLOOD CONTROL					2/25/55
	c. POWER					
	d. JOINT USE	466.0	11,440	411,211	1,010,230	16 DATE NORMAL OPERATIONS BEGAN
	e. CONSERVATION	425.8	8,946	508,972	599,019	
	f. INACTIVE	327.0	2,035	89,933	90,047	
	g. DEAD	205.5	20	114	114	12/6/1955[5]
	17. LENGTH OF RESERVOIR 28 [6] MILES		AVG. WIDTH OF RESERVOIR 0.64 MILES			

B	18. TOTAL DRAINAGE AREA 1,861 SQUARE MILES		22. MEAN ANNUAL PRECIPITATION 22.4 [7] INCHES
A	19. NET SEDIMENT CONTRIBUTING AREA 1,020 [8] SQUARE MILES		23. MEAN ANNUAL RUNOFF 28.2 INCHES
S	20. LENGTH MILES	AVG. WIDTH MILES	24. MEAN ANNUAL RUNOFF 2,787,400 [9] ACRE-FEET
IN	21. MAX. ELEVATION	MIN. ELEVATION	25. ANNUAL TEMP, MEAN 61 °F RANGE 17 °F to 112 °F [7]

SURVEY DATA

26. DATE OF SURVEY	27. PER. YRS	28. PER. YRS	29. TYPE OF SURVEY	30. NO. OF RANGES OR INTERVALS	31. SURFACE AREA, AC.	32. CAPACITY ACRE - FEET	33. C/ RATIO AF/AF
2/25/55			Contour (D)	20-ft	11,440 [4]	1,010,230 [4]	0.36
4/15/91 [10]	36.1	36.1	Contour (D)	5-ft	11,183 [11]	976,955 [11]	0.35
9/21/05	14.4	50.5	Contour (D)	3-ft	11,140 [11]	966,823 [11]	0.35

26. DATE OF SURVEY	34. PERIOD ANNUAL PRECIPITATION	35. PERIOD WATER INFLOW, ACRE-FEET			36 WATER INFLOW TO DATE, AF	
		a. MEAN ANN.	b. MAX. ANN.	c. TOTAL	a. MEAN ANN.	b. TOTAL
4/15/91	22.7 [7]	2,795,100 [9]	6,541,200	101,568,200	2,745,100	101,568,200
9/21/05		2,899,400	5,414,300	40,591,900	2,787,400	142,160,100

26. DATE OF SURVEY	37. PERIOD CAPACITY LOSS, ACRE-FEET			38. TOTAL SEDIMENT DEPOSITS TO DATE, AF		
	a. TOTAL	b. AVG. ANN.	c. /MI.^{2}-YR.	a. TOTAL	b. AVG. ANN.	c. /MI.^{2}-YR.
4/15/91	33,275 [12]	921.7 [12]	0.90 [12]	33,275 [12]	921.7 [12]	0.90 [12]
09/21/05	10,132 [12]	703.6 [12]	0.69 [12]	43,407 [12]	859.5 [12]	0.84 [12]

26. DATE OF SURVEY	39. AVG. DRY WT. (#/FT^3)	40. SED. DEP. TONS/MI.^{2}-YR		41. STORAGE LOSS, PCT.		42 SEDIMENT INFLOW, PPM	
		a. PERIOD	b. TOTAL TO DATE	a. AVG. ANNUAL	b. TOTAL TO DATE	a. PER.	b. TOT.
4/15/91				0.091	3.29		
9/21/05				0.085	4.30		

26. DATE OF SURVEY	43. DEPTH DESIGNATION RANGE BY RESERVOIR ELEVATION										
		190.0-300.0	300.0-327.0	327.0-350.0	350.0-370.0	370.0-390.0	390.0-410.0	410.0-430.0	430.0-450.0	450.0-466.0	
	PERCENT OF TOTAL SEDIMENT LOCATED WITHIN DEPTH DESIGNATION										
9/21/05		14.6	6.4	5.9	2.5	6.1	15.4	19.9	16.6	12.6	

26. DATE OF SURVEY	44. REACH DESIGNATION PERCENT OF TOTAL ORIGINAL LENGTH OF RESERVOIR													
	0-10	10-20	20-30	30-40	50-60	60-70	70-80	80-90	90-100	100-105	105-111	110-115	115-120	120-125
	PERCENT OF TOTAL SEDIMENT LOCATED WITHIN REACH DESIGNATION													

Table 1 - Reservoir sediment data summary (1 of 2).

45. RANGE IN RESERVOIR OPERATION [13]

YEAR	MAX. ELEV.	MIN. ELEV.	INFLOW, AF	YEAR	MAX. ELEV.	MIN. ELEV.	INFLOW, AF
				1955	408.0	244.5	1,204,600
1956	466.9	353.2	4,781,300	1957	466.6	400.0	2,296,600
1958	466.6	398.7	4,205,400	1959	456.2	387.3	1,315,400
1960	466.9	378.2	1,760,700	1961	458.9	391.6	1,180,500
1962	460.7	394.5	2,171,000	1963	467.2	406.0	3,386,600
1964	464.6	403.0	1,914,300	1965	466.3	410.2	4,421,200
1966	457.7	416.3	1,516,500	1967	465.3	419.6	3,987,000
1968	446.2	418.8	1,844,600	1969	455.7	406.5	4,548,800
1970	457.1	420.1	3,380,000	1971	463.9	416.3	3,040,400
1972	464.4	421.8	2,067,900	1973	464.6	424.0	3,093,100
1974	462.8	424.4	4,407,800	1975	461.3	425.2	2,785,700
1976	443.7	403.1	1,142,300	1977	404.1	349.6	356,000
1978	460.0	347.6	2,963,100	1979	464.5	428.3	2,276,600
1980	454.8	424.7	3,971,800	1981	461.3	425.9	1,411,800
1982	465.0	419.1	6,112,800	1983	465.1	427.7	6,541,200
1984	464.5	426.8	4,159,100	1985	455.5	424.5	1,796,700
1986	465.7	413.1	4,573,400	1987	440.9	405.1	1,102,900
1988	415.0	369.1	962,400	1989	463.3	358.9	2,167,000
1990	425.8	359.0	1,345,000	1991	443.8	352.0	1,376,700
1992	440.0	360.3	1,086,500	1993	465.5	355.6	3,259,600
1994	424.0	370.9	1,041,500	1995	465.5	364.3	5,414,300
1996	464.4	393.7	3,799,600	1997	455.6	394.4	4,886,800
1998	463.6	405.4	4,437,400	1999	463.6	416.0	3,431,000
2000	448.9	416.3	2,722,500	2001	440.3	398.2	1,244,200
2002	452.8	378.9	1,865,200	2003	465.1	407.6	2,404,600
2004	440.7	399.6	1,794,700	2005	465.4	391.4	3,204,000

46. ELEVATION - AREA - CAPACITY - DATA FOR **2005 CAPACITY**

ELEVATION	AREA	CAPACITY	ELEVATION	AREA	CAPACITY	ELEVATION	AREA	CAPACITY
2005	**SURVEY**		190.0	0	0	195.0	0	1
200.0	0	3	205.0	1	6	210.0	2	11
215.0	13	55	220.0	45	180	225.0	97	543
230.0	136	1,128	235.0	190	1,931	240.0	249	3,037
245.0	300	4,410	250.0	354	6,049	255.0	420	7,976
260.0	489	10,253	265.0	560	12,869	270.0	630	15,844
275.0	710	19,191	280.0	785	22,932	285.0	864	27,051
290.0	952	30,585	295.0	1,062	36,611	300.0	1,184	42,226
305.0	1,304	48,449	310.0	1,426	55,271	315.0	1,554	62,715
320.0	1,691	70,823	325.0	1,841	79,646	327.0	1,907	83,393
330.0	2,015	89,273	335.0	2,216	99,838	340.0	2,428	111,443
345.0	2,646	124,123	350.0	2,886	137,943	355.0	3,168	153,060
360.0	3,519	169,747	365.0	3,914	188,313	370.0	4,346	208,980
375.0	4,726	231,679	380.0	5,104	256,241	385.0	5,466	282,681
390.0	5,835	310,921	395.0	6,202	341,024	400.0	6,576	372,964
405.0	6,979	406,833	410.0	7,381	442,754	415.0	7,743	480,578
420.0	8,078	520,142	425.0	8,407	561,341	427.0	8,612	578,359
430.0	8,874	604,610	435.0	9,239	649,918	440.0	9,574	696,955
445.0	9,894	745,632	450.0	10,208	795,889	455.0	10,515	847,700
460.0	10,801	900,996	465.0	11,084	955,710	466.0	11,140	966,823
470.0	11,380	1,011,844	475.0	11,687	1,069,528	475.4	11,710	1,074,207
480.0	11,973	1,128,679	485.0	12,233	1,189,200	490.0	12,484	1,250,992

47. REMARKS AND REFERENCES

[1] Constructed by the Corps of Engineers. Upon completion transferred to Reclamation to operate as part of the Central Valley Project.
[2] Top of parapet wall at elevation 484.0. All elevations in feet based on original project datum that is 2.34 feet lower than NAVD88 (2005).
[3] Spillway crest elevation 418.0. Elevation top of drum gates: five at 468.0 and three at 471.0 feet.
[4] Area and capacity calculated by BOR program ACAP for sediment computation purposes. Areas below elevation 400.0 based on 1935-36 USGS survey resulting in 20-foot contour map. Areas above elevation 400.0 based on USGS quadrangle maps dated 1940's.
[5] Full power production with all generators began on December 6, 1955.
[6] Total length includes North Fork American River (16.3 miles) and South Fork American River (11.7 miles) at elevation 466.
[7] Climate records, Bureau of Reclamation Project Data Book, 1981. Values for Central Valley Project.
[8] Total drainage area from USGS water year records. Loss of contributing areas due to closing of North Fork in 1941 (343 mi^2), French Meadows in 1964 (34.2 mi^2), Hell Hole in 1965 (79.8 mi^2), Loon Lake in 1963 (6.0 mi^2), Caples Lake in 1924 (13.5 mi^2), Silver Lake in in 1876 (15.2 mi^2), Union Valley in 1962 (66.9 mi^2), Ice House in 1959 (23.6 mi^2), Slab Creek in 1967 (229.4 mi^2), Lake Edison in in 1961 (13.0 mi^2), and Folsom Lake surface area (17.9 mi^2).
[9] Computed inflows for water years 1955 through 2005. For water year 1955, inflow values for March through September 1955.
[10] 1991 bathymetric survey performed by Reclamation, 4/15/91 through 4/21/91 at elevation 418. Aerial on 10/10/90 at elevation 366.0
[11] Surface area and capacity at joint use elevation 466.0.
[12] Computed capacity loss, by comparing differences of surveys, is affected by the detail and methods of surveys. Original data from 20-foot contour intervals. 1991 and 2005 data from more detailed aerial and underwater data.
[13] Water year data provided by Reclamation's Mid Pacific Regional Office.

48. AGENCY MAKING SURVEY Bureau of Reclamation
49. AGENCY SUPPLYING DATA Bureau of Reclamation DATE September 2006

Table 1 - Reservoir sediment data summary (page 2 of 2).

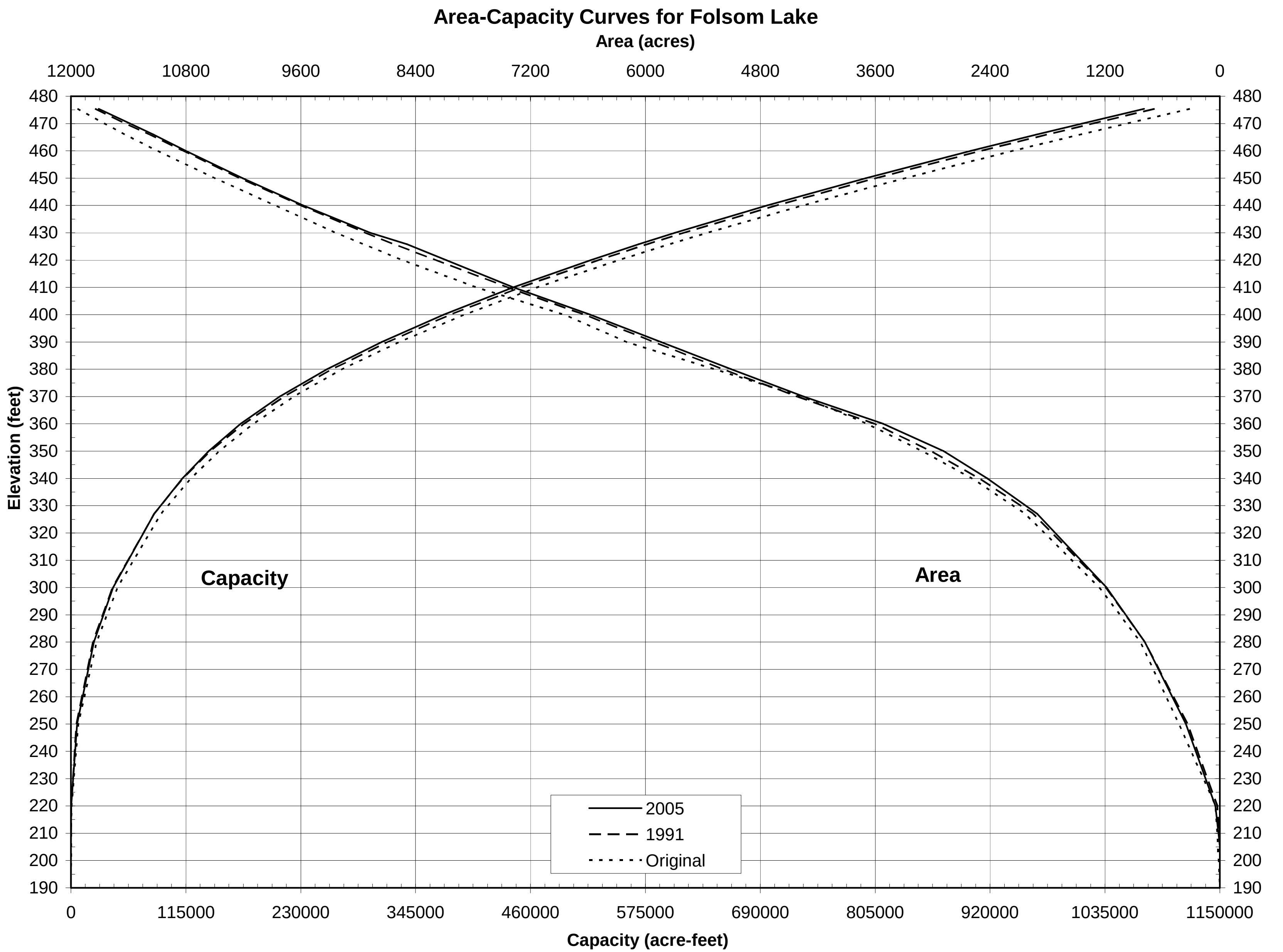

Figure 12 - Folsom Lake area and capacity plots.

1	2	3	4	5	6	7	8	9	10	11	12	13	14
					1991				2005				
					Computed	1991			Total	2005	Sediment	Percent	
	Original	Original	1991	1991	Sediment	Percent	2005	2005	Sediment	Percent	Volume	Computed	Percent
Elevation	Survey	Capacity	Survey	Survey	Volume	Computed	Survey	Survey	Volume	Computed	1991-2005	Sediment	Reservoir
Feet	Acres	Ac-Ft	Acres	Ac-Ft	Ac-Ft	Sediment	Acres	Ac-Ft	Ac-Ft	Sediment	Ac-Ft	1991-2005	Depth
475.4	11931	1120078	11749	1084778	35300		11710	1074207	45871		10571		100.0
470	11650	1056410	11432	1022185	34225		11380	1011844	44566		10341		98.1
466	11440	1010230	11183	976955	33275	100.0	11140	966823	43407	100.0	10132	100.0	96.7
460	11100	942610	10829	910928	31682	95.2	10801	900966	41644	95.9	9962	98.3	94.6
450	10500	834610	10240	805535	29075	87.4	10208	795889	38721	89.2	9646	95.2	91.1
440	9870	732760	9604	706360	26400	79.3	9574	696955	35805	82.5	9405	92.8	87.6
430	9240	637210	8932	613650	23560	70.8	8874	604610	32600	75.1	9040	89.2	84.1
425.8	8946	599019	8629	576772	22247	66.9	8489	568099	30920	71.2	8673	85.6	82.6
420	8540	548310	8191	527987	20323	61.1	8078	520142	28168	64.9	7845	77.4	80.6
410	7770	466760	7444	449815	16945	50.9	7381	442754	24006	55.3	7061	69.7	77.1
400	6850	393660	6638	379410	14250	42.8	6576	372964	20696	47.7	6446	63.6	73.6
390	6200	328410	5915	316613	11797	35.5	5835	310921	17489	40.3	5692	56.2	70.1
380	5280	271010	5196	261095	9915	29.8	5104	256241	14769	34.0	4854	47.9	66.6
370	4370	222760	4413	212978	9782	29.4	4346	208980	13780	31.7	3998	39.5	63.1
360	3690	182460	3609	173043	9417	28.3	3519	169747	12713	29.3	3296	32.5	59.6
350	3110	148460	3019	139493	8967	26.9	2886	137943	10517	24.2	1550	15.3	56.1
340	2590	119960	2514	111945	8015	24.1	2428	111443	8517	19.6	502	5.0	52.6
327	2035	90047	1950	83071	6976	21.0	1907	83393	6654	15.3	-322	-3.2	48.0
300	1260	46310	1192	41465	4845	14.6	1184	42226	4084	9.4	-761	-7.5	38.5
280	830	25560	784	22040	3520	10.6	785	22932	2628	6.1	-892	-8.8	31.5
250	430	7260	338	5450	1810	5.4	354	6049	1211	2.8	-599	-5.9	21.0
220	45	585	26	130	455	1.4	45	180	405	0.9	-50	-0.5	10.5
205.5	20	114	1	3	111	0.3	1	6	108	0.2	-3	0.0	5.4
190	0	0	0	0	0	0.0	0	0	0	0.0	0	0.0	0.0
1	Elevation of reservoir water surface.												
2	Original reservoir surface areas.												
3	Original reservoir capacity computed using ACAP.												
4	1991 measured reservoir surface area.												
5	1991 reservoir capacity computed using ACAP.												
6	1991 computed sediment volume, column (3) - column (5).												
7	1991 measured sediment in percentage of total sediment, 33,275 acre-feet, by elevation.												
8	2005 measured reservoir surface area.												
9	2005 reservoir capacity computed using ACAP.												
10	2005 measured sediment volume = column (3) - column (9).												
11	2005 measured sediment in percentage of total sediment, 43,407 acre-feet, by elevation.												
12	Measured sediment volume from 1991 to 2005, column (5) - column (9).												
13	Measured sediment in percentage my elevation from 1991 to 2005. Total sediment volume of 10,132 acre-feet.												
14	Depth of reservoir expressed in percentage of total depth (285.4), from maximum water surface.												

Table 2 - Summary of 2005 survey results.

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Corps of Engineers, January 2002. *Engineer and Design* - ***Hydrographic Surveying***, EM 1110-2-1003, Department of the Army, Washington DC, (www.usace.army.mil/inet/usace-docs/eng-manuals/em1110-2-1003/toc.htm).

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Appendix

February 21, 2007
Folsom Dam Raise
Topography and Imagery

Identification_Information:
Citation:
Citation_Information:
Originator:
U.S. Bureau of Reclamation.
Publication_Date:
May 2006
Title:

Folsom Dam Raise - Topography and Imagery

Master Data files:

BR FSC 11 and 12 Master Topo Folsom Dam.dwg

Geospatial_Data_Presentation_Form:
Raster and Vector Data
Online_Linkage:
AM Teamworks and/or REDS
Overview:

Topography, Bathymetry and Imagery was developed to study the Dikes, the Wing Dams, Folsom Dam, and Mormon Island Auxiliary Dam along with the up-stream arms of the American River.

The project consists of 1 Sheet Index @ 1:48000 on drawing number 485-208-1882 and 204 mapping sheets @ 1:1200 on drawing numbers 485-208-1883 through 485-208-2086.

Orthophotos for above water sheets in related sheet tiles are in .sid format with .sdw files for geo-referencing.

Project Datum:
California Coordinate System, CCS, Zone 2, U.S. Survey Feet
Horizontal Datum:
NAD 83
Vertical Datum:
NAVD 88 (2005)

Process and Methodology Description:

Ground Control:
The Bureau of Reclamation and the Army Corps of Engineers, under the direction of the National Geodetic Survey, performed an extensive geodetic control network in NAD83/NAVD88 encompassing the project. Aerial control was then established for the photogrammetric work.

Aerial Photography:
Under contract with BOR, American Aerial Surveys, Ione, CA provided color vertical aerial photography as follows; BR-FSC-11@ 1:6000 flown for the Upstream Arms of the American River on 10-20-05 with a reservoir elevation of 430.2'. BR-FSC-12 @ 1:3600 flown for high accuracy mapping from Dike 1 to MIAD on 10-31-05 with a reservoir elevation of 427.1'. Aerial obliques of all structures were taken 12-05-05.
Bathymetry:
Bathymetric Data was collected in October 2005 using Real Time Kinetic Global Positioning System with a Multiple Beam Sonar Collection System by the Bureau of Reclamation Sedimentation and River Hydraulics Group from Denver, CO. The average reservoir elevation during data collection was 440' giving reliable readings below approximately 425'.

The bathymetric data was collected prior to the completion of the Geodetic Network and was post processed to the final survey adjustment. To allow efficient contour generation, the data files were filtered. A Triangulated Irregular Network (TIN) was computed with the insertion of the photogrammetric water surfaces as a breakline around the perimeter.

Because the sensor could not collect data in shallow water, some of the upstream arm data was interpolated between the photogrammetric water surface information and the bathymetric data points which created anomalous contour data and should be considered unreliable. These areas will not meet any accuracy standards. This data was not edited or clipped out to keep vector work concatenated for area capacity and surface area calculations.

Photogrammetry:

Using standard photogrammetric processes, film diapositives were used for aerial triangulation and subsequent 3D data collection. Breaklines and random and regularly gridded points were compiled to create surface models for subsequent contour generation.

1 foot contours were produced from Dike 1 to Morman Island Auxiliary Dam, 2 foot contours cover the upstream arms and 5 foot contours reflect the bathymetry, all at a horizontal scale of 1:1200/1"=100'.

Field surveys were performed to check final data and the results meet or exceed standards on above water data.

Orthophotos:

Full orthorectification was performed using the photogrammetric surface information. Tif images were compressed to create .sid files with associated world files corresponding to the sheet layout and index. Any surface information outside of the contoured corridors were only collected for gross rectification for surrounding areas and said data and imagery will not meet mapping standards.

Satellite Imagery:
p0576North.sid and .sdw
p0576South.sid and .sdw

Date Flown: January 2004

Pixel Size: 2 feet

Prepared By: IntraSearch Inc.
MapMart
12424 East Weaver Place
Suite 100
Centennial, Colorado 80111
303-759-5050
303-759-0400 - fax
www.intrasearch.com
www.mapmart.com
info@intrasearch.com
Map Mart Project Number: 2004-p-0576
Digital Take Line Information:

Information for a digital take line (NAD27) was extracted from (22) USACE drawings AM-1-13-490 dated January 1952 and (6) BOR drawings 485-208-254 through 485-208-259 dated October 1958. A simple transformation was performed to move this data into the current horizontal datum of NAD83.

This boundary data is limited by the condition of the records and was oftentimes not clearly visible, had obvious errors and omissions and was produced using survey and drafting methods suitable only for that particular timeframe. Some data manipulation was necessary to resolve minor issues of precision and newer technology. Major issues with no obvious resolution are noted.

Some field work was performed to check this data and without significant field verification we believe the take line data is now within a +/-10' error ellipse but have no solutions for major discrepancies. The take line data should be considered informational only until a full boundary survey is performed.

Horizontal Alignment Information:

Information for the construction of a digital horizontal alignment was taken from BOR's Spec 896 book. The horizontal alignment data is limited by the condition of the records and was oftentimes not clearly visible, had obvious errors and omissions and was produced using survey and drafting methods suitable only for that particular timeframe. Some data manipulation was necessary to resolve minor issues of precision and newer technology.

Data Management:

Appropriate data files are referenced into the master AutoCAD file:

BR FSC 11 and 12 Master Topo Folsom Dam.dwg

Mapping Features_2d.dwg

Contour Data referenced into master AutoCAD file per contour blocks:

Contour Blk Dikes 1-3.dwg
Contour Blk Dikes 4-6.dwg
Contour Blk RWD-Dike7.dwg
Contour Blk Dikes8-MIAD.dwg
Contour Blk North and South Forks.dwg
Contour Blk North Fork Bathymetry.dwg
Contour Blk South Fork Bathymetry.dwg

Surface Data in the form of DTM breaklines and random and regularly gridded points are available:

DTM Dikes 1-3.dwg
DTM Dikes 4-6.dwg
DTM RWD-Dike7.dwg
DTM Dikes8-MIAD.dwg
DTM North and South Forks.dwg
DTM North Fork Bathymetry.dwg
DTM South Fork Bathymetry.dwg

Additional Data Available:

Mapping Features_3d.dwg
Folsom Take Line 83.dwg
Waterlines (2d and 3d).dwg

Data Limitations:

Terrain information outside the main contour corridors will not meet standards for vertical accuracy and is intended only to support gross orthorectification of surrounding areas.

Data Duplication:

USBR Surveys and Photogrammetry; Art Aguirre 978-5333 or AM Teamworks and/or REDS.

Time_Period_Information:
Calendar_Date: Date of Aerial Photography October 20 and 31, 2005
Date of Bathymetry August 2005
Currentness_Reference:
Current to the listed Calendar Date provided. See Data Quality for process steps.
Status:
Progress: Only current to the date specified.
Maintenance_and_Update_Frequency: Updated as determined by USBR.
Spatial_Domain:
Bounding_Coordinates:
West_Bounding_Coordinate: -121.10.00
East_Bounding_Coordinate: -121.00.00
North_Bounding_Coordinate: 38.46.00
South_Bounding_Coordinate: 38.37.30
Access_Constraints: None.
Use_Constraints:
If used, please indicate that the database source was the U.S. Bureau of Reclamation.
Point_of_Contact:

Contact_Information:
Contact_Person_Primary:
Contact_Person: Terri Reaves
Contact_Organization: U.S. Bureau of Reclamation, Design and Construction
Contact_Position: Regional Chief of Surveys and Mapping
Contact_Voice_Telephone: (916) 978-5306
Contact_Facsimile_Telephone: (916) 978-5345
Contact_Electronic_Mail_Address: treaves@mp.usbr.gov
Hours_of_Service: 7:00 AM - 4:00 PM M-F
Contact_Instructions: Email your request to the above email address
Data_Set_Credit: Bureau of Reclamation
Division of Design and Construction, Surveys and Mapping Branch
2800 Cottage Way
Sacramento, CA 95825
916-978-5306
POC Terri Reaves

Geodetic Survey and Ground Control

Frame Surveys and Mapping
17029 Lambert Rd.
Ione, CA 95640
209-274-6500
POC Curtis Holmes

Aerial Photography

American Aerial Surveys
17029 Lambert Rd.
Ione, CA 95640
209-274-6500
POC Curtis Holmes

Bathymetry

USBR - Reservoir Area and Upstream Arms - 5 foot contours
Sedimentation and River Hydraulics Group
Denver, Co
POC Ronald Ferrari
303-445-2551

Photogrammetry

USBR - Dike 1 through MIAD - 1 foot contours
Surveys and Mapping Branch
Sacramento, CA
POC Terri Reaves
916-978-5306

American Aerial Surveys /Spectrum Mapping - North and South Forks - 2 foot contours
17029 Lambert Rd.
Ione, CA 95640
209-274-6500
POC Curtis Holmes

Orthophotos

Tri-State Surveying, Ltd., Inc.
1925 East Prater Way
Sparks, NV 89434-8938
775-358-9491

POC Mitch Bartorelli

Security_Information:
Security_Classification_System: None
Security_Classification: None
Security_Handling_Description: None
Native_Data_Set_Environment:
AutoCAD 2005
Cross_Reference:
Citation_Information:
Title: None

Data_Quality_Information:
Digital databases created within the USBR were reviewed using existing quality standards.
Logical_Consistency_Report:
Data meets accuracy and quality standards within USBR, the National Standard for Spatial Data Accuracy (NSSDA) and the National Map Accuracy Standards (NMAS) for 1:1200 with 1, 2 and 5 foot contours.
Completeness_Report:
This database represents the most current and up to date mapping for the Folsom Project up to the date of photography, October 2005.
Horizontal_Positional_Accuracy:
All horizontal positions meet or exceed NSSDA.
Vertical_Positional_Accuracy:
All vertical positions meet or exceed NSSDA.
Lineage:
Source_Information:
Source_Scale_Denominator: 1:3600 photography and 1:6000 photography.
Type_of_Source_Media:

Aerial Photography:
American Aerial Surveys, Inc - Ione, California
Zeiss RMK Top 15
Zeiss Pleogon a3/4
Camera Serial No. :141307
Lens Serial No. : 141329
Calibrated Focal Length 154.060

Spatial_Reference_Information:
Horizontal_Coordinate_System_Definition: NAD83
Grid_Coordinate_System: California Coordinate System - Zone 2
Lambert:
Scale_Factor_at_Central_Meridian: .999914672977
Longitude_of_Central_Meridian: -122.00.00
Latitude_of_Projection_Origin: 37.40.00
False_Easting: 6561666.667'
False_Northing: 1640416.667'
Geodetic_Model:
Horizontal_Datum_Name: North American Datum of 1983
Ellipsoid_Name: Geodetic Reference System 1980
Semi-major_Axis: 20925604.4742'
Denominator_of_Flattening_Ratio: 298.2572221008827

Distribution_Information:
Distributor:

Contact_Information:
Contact_Organization_Primary:
Contact_Organization:
U.S. Bureau of Reclamation, Mid-Pacific Region, Design and Construction
Contact_Person: Terri Reaves
Contact_Position: Regional Chief of Surveys and Mapping
Contact_Voice_Telephone: 916-978-5306
Contact_Facsimile_Telephone: 916-978-5345
Contact_Electronic_Mail_Address: treaves@mp.usbr.gov
Hours_of_Service: 7:00 AM - 4:00 PM
Contact_Instructions: Call or email for method of data transfer
Distribution_Liability:
This data set was designed for 1:1200 mapping with 1 foot contour coverage on Dike1 to Mormon Island Auxiliary Dam, 2 foot contour coverage on up-stream arms inside of USBR take line and 5 foot bathymetric contour coverage. Mapping was developed from the source documents following standard procedures of compilation, draft editing, map and orthophoto generation. Use of this data at scales and contour intervals which are more detailed than the source is not recommended.

Metadata_Reference_Information:
Metadata_Date: May 2006
Metadata_Review_Date: May 2006
Metadata_Contact:
Contact_Information:
Contact_Person_Primary:
Contact_Person: Terri Reaves
Contact_Organization: U.S. Bureau of Reclamation, Mid-Pacific Region
Contact_Position: Chief of Surveys and Mapping
Contact_Address:
Address_Type: mailing address
Address:
US Bureau of Reclamation
Terri Reaves - MP222
2800 Cottage Way W2916
City: Sacramento
State_or_Province: CA
Postal_Code: 95825
Country: USA
Contact_Voice_Telephone: 916-978-5306
Contact_Facsimile_Telephone: 926-978-5345
Contact_Electronic_Mail_Address: treaves@mp.usbr.gov
Hours_of_Service: 7:00 AM - 4:00 PM
Contact_Instructions: Call or email
Metadata_Standard_Name: FGDC Content Standards for Digital Geospatial Metadata
Metadata_Standard_Version: FGDC-STD-001-1998
Metadata_Time_Convention: local time
Metadata_Access_Constraints: None
Metadata_Use_Constraints:
If used, please indicate that the data source was the U.S. Bureau of Reclamation, Mid-Pacific Region.

[redacted]

Increasing Water Supply Pumping Capacity at Folsom Dam

Prepared for:

the City of Roseville
Department of Environmental Utilities

in cooperation with

San Juan Water District, City of Folsom, and
Sacramento Area Flood Control Agency

Prepared by:

ESA Consultants Inc.
201 San Antonio Circle, Suite 102
Mountain View, California 94040

Prepared in Association with:

Murray, Burns and Kienlen
1616 29th Street, Suite 300
Sacramento, California 95816

and

SAI Engineers, Inc.
1290 Oakmead Parkway, Suite 301
Sunnyvale, CA 94086

ESA Project 043.9501

January 1996

Increasing
Water Supply Pumping Capacity
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Prepared for:

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Sacramento Area Flood Control Agency

REGISTERED PROFESSIONAL ENGINEER
WILL B
BETCHART
No C33939
EXP 12/31/98
CIVIL
STATE OF CALIFORNIA

Will B Betchart

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ESA Project 043.9501

ESA Consultants

January 1996

ESA Consultants Inc.
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January 31, 1996
043.9501

Mr. Joseph D. Countryman, President
Murray, Burns and Kienlen
1616 29th Street, Suite 300
Sacramento, CA 95816

Re: Increasing Water Supply Pumping Capacity at Folsom Dam; Final Report

Dear Mr. Countryman:

The final report for the referenced project is attached. Per our scope of work, it documents hydraulic analyses and presents a conceptual design and cost estimate for increasing water supply delivery capacity from the Folsom Project.

Installing two additional pumps in available positions in the existing Folsom Pumping Plant can achieve the immediate objective of a total system flow rate of 400 cubic feet per second (cfs) when the lake is at Elevation 392. Per the criteria established prior to our study, this is sufficient to implement Roseville's option to increase their peak flow entitlement to 150 cfs. The pumps have also been selected to provide a substantial increase in system capacity at low reservoir elevations (high pumping heads). The agencies served by the Folsom Pumping Plant and the U.S. Bureau of Reclamation (USBR) have expressed general agreement with the above findings. Specific written comments on our draft report were received from the USBR and from the City of Folsom's consultant, Robert W. Miles. These comments have been included in Appendix B and will be considered as design proceeds.

ESA Consultants Inc. has been pleased to collaborate with Murray, Burns and Kienlen on this project. We look forward to working with you and our clients to refine the design concept and to implement the resulting facility improvement plan.

Sincerely,

ESA Consultants Inc.

Will B. Betchart

Will B. Betchart, P.E.
Project Manger

Attachment

INCREASING WATER SUPPLY PUMPING CAPACITY FROM FOLSOM

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1. SUMMARY

The engineering study documented in this report has examined alternatives for increasing the Folsom Project water supply delivery capacity to 400 cfs from its present nominal capacity of 315 cfs. Such an increase is necessary for Roseville to exercise its present contractual option to receive a peak flow of 150 cfs -- which is an 85 cfs increase over its present maximum. Initially, the reservoir levels which pertain to these capacities were undefined. Thus, parts of the previous study and this study addressed that topic.

The present study examined a number of hydraulic changes that might contribute to or be necessitated by Roseville's increase. The results of these analyses focused attention on the pumping plant and the opportunity to increase capacity by installing additional pumps in two pump locations that are part of the initial pumping plant layout, but do not presently have pumps installed. Pumps were identified that will fit in the available positions and will achieve the project objectives.

Specifically, installation of two pumps with the capabilities identified will raise the project delivery capability to 400 cfs when the Folsom Lake water surface is at approximately Elevation (El.) 392. Figure 1-1 presents the system capacity curve (versus lake level) that the two additional pumps are estimated to provide when operated in combination with the existing pumps.

Furthermore, if delivery of 400 cfs at lower lake levels is required in the future, similar pumps can be installed at other pump positions replacing the more modest pumps already present. Such a program, extending the approach developed herein, could provide a system capacity of 400 cfs even at very low lake levels (approaching minimum pool).

A major consideration in developing a conceptual design is deciding between three distinct methods for driving the new pumps, namely

- Single speed motors
- Two speed motors
- Variable speed motors.

Analyses demonstrated that two markedly different project needs create a dilemma. Low head pumps suit the project well most of the time because lake levels are generally above El. 380. However, when lake levels are lower, the project must still have the capability to deliver substantial flows. To do so, the project pumps must have high head capability. However, using high head pumps when reservoir levels are high can waste large amounts of energy. The conclusions resulting from our analyses are the following:

- Additional single speed pumps are simply unworkable for the Folsom Project, given presently installed pumps and the variety of pumping requirements encountered. The project needs a substantial increase in high-head capacity. But a single speed pump for high head situations is too wasteful of energy at the low-head conditions that generally exist.

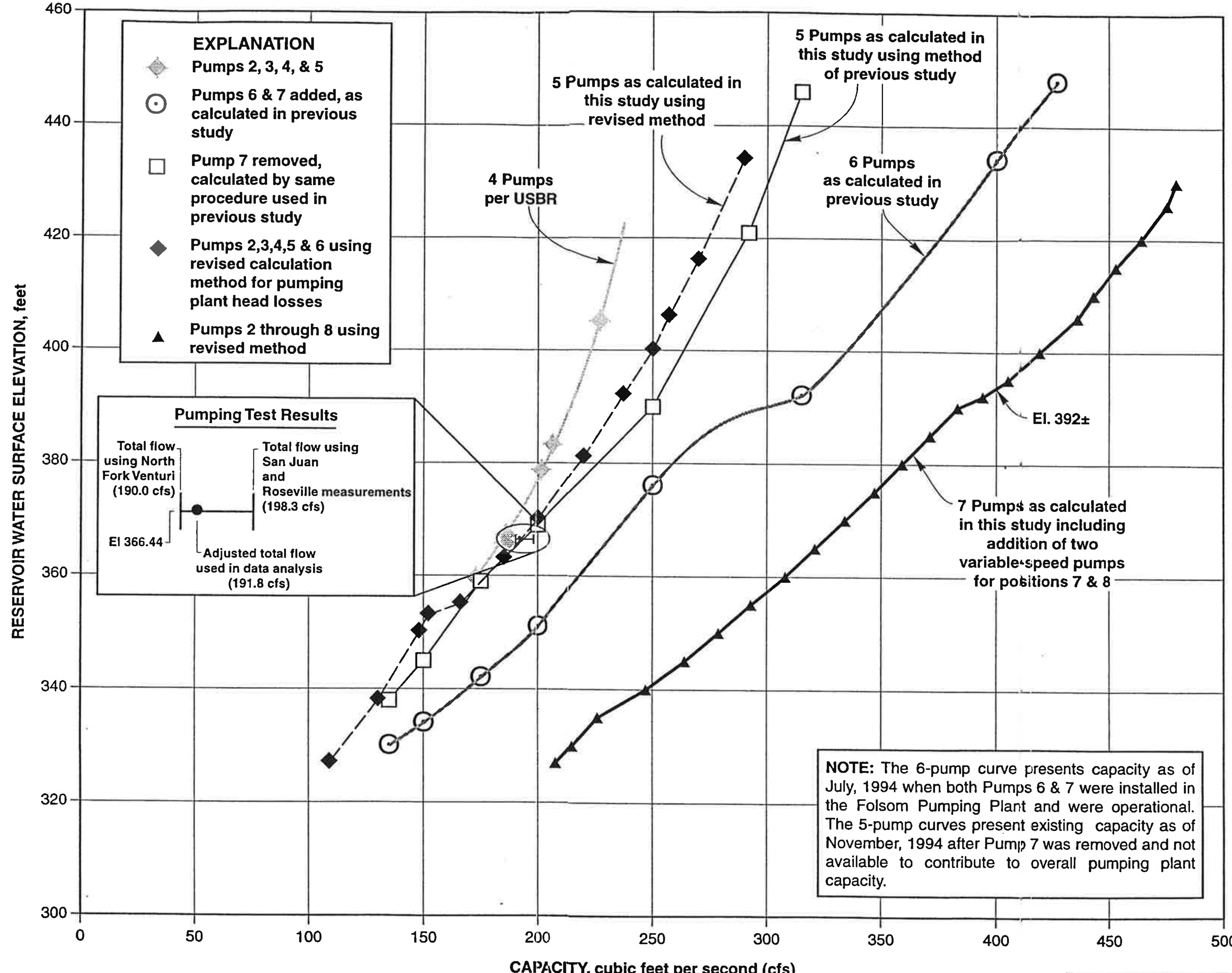
- Two speed pumps seemed to offer an approach for providing both high head and moderate to low head capacity enhancements. However, the substantial savings expected in equipment cost did not materialize. When the lower speed of the two speed pump is larger than half the high speed, two windings are required in the motor. This substantially raises the cost of a two speed motor. Decreasing to half speed, although more economical, is not satisfactory either. The decrease in pump capability is simply too dramatic.

- Variable speed pumps offer the best solution to deal with the dilemma. The increase in cost over two speed motors is modest, and they provide tremendous flexibility to deliver a variety of flows at differentpumping heads without wasteful throttling.

Accordingly, installation of two additional pumps with variable speed motors is recommended. Pumps, motors and variable frequency drives have been researched and a practical system for installation in the existing pumping plant has been identified. Details are presented in Section 5. Other issues that were addressed in the work and the resulting conclusions are:

- The 60 inch valve located in the intake line does not need to be replaced based on increasing system flows to 400 cfs. However, this valve could become a significant system restriction at higher flows. There is a more important question, however, relating to this valve's capability for emergency closure (e.g., if there were a line break between the dam and the pumping plant as the result of an earthquake). The valve was rated for a 75 psi working pressure when it was new in 1952. If an emergency closure event occurred when Folsom Reservoir were full (El. 466), the valve would be faced with a maximum working pressure of 70 psi. Failure to achieve emergency closure would be unacceptable. Such failure could mean extreme drawdown of the reservoir to achieve closure and a prolonged outage of water supplies during reservoir drawdown and pipeline repair. Accordingly, ESA recommends that the water agencies request or otherwise initiate a detailed review of the emergency closure capability of both the 60" intake valve and the 42" (Natoma) intake valve (which has the same importance). Details are discussed in Section 6.
- The potential for vortex development at the intake was reviewed. Based on available literature, the potential for vortex problems does not appear to be significant with either the existing pumping capacity or that proposed with addition of two new pumps. If additional capacity were added beyond that proposed above, vortexes might develop at very low reservoir levels--approaching minimum pool. It is recommended that further consideration of vortex issues be delayed until such subsequent expansions are proposed.
- Other potential flow restrictions in the water delivery system were considered -- such as the North Fork line venturi meter. At a system flow of 400 cfs, such contractions and expansions are not a significant concern.

In summary, a substantial improvement of system capacity can be achieved by adding two pumps with variable speed motors. The estimated construction cost for this improvement is $ 1.9 million (including a 15 percent allowance for contingencies). Inclusion of allowances for engineering, construction supervision and Bureau of Reclamation reviews raises the total to $ 2.3 million. The above estimates reflect mid-1995 price levels.



ESA Consultants
Mountain View, California

FOLSOM DAM - WATER SUPPLY PUMPING CAPACITY
PUMPING CAPACITY WITHOUT AND WITH TWO ADDITIONAL PUMPS

Checked By Peter V. Jacke	Date 1/26/96	Project No.	Figure No.
Approved By B	Date 1/31/96	043.9501	1-1

ARWA-202

2. BACKGROUND

The city of Roseville receives its raw water supply from the United States Bureau of Reclamation (USBR) water supply facilities at Folsom Dam. The relative locations of Roseville and Folsom Lake are shown in the vicinity map presented in Figure 2-1. Roseville is presently authorized to receive up to 65 cubic feet per second (cfs) of flow. Roseville's contract with USBR contains an option to increase this peak flow rate to 150 cfs. Roseville wishes to implement this increase because of its projected water demand. In pursuing that objective, Roseville is cooperating with the other water supply agencies served by the Folsom Project (San Juan Water District, Folsom State Prison, and the City of Folsom) and with other interested agencies including Placer County Water Agency, Sacramento Area Flood Control Agency, and the USBR. Various relevant water supply facilities are highlighted in the location map presented in Figure 2-2. A schematic diagram of the project water distribution facilities is presented Figure 2-3.

A first phase of study was performed previously to develop and present information on the physical/technical features and limits of the Folsom Dam water supply facilities and to characterize the needed facility improvements to implement the flow increase. That study produced a report (ESA Consultants, November 1994) documenting Folsom Project water supply topics including:

- Peak flow entitlements and needs.
- History of Folsom Dam water supply facilities development and improvement.
- Basis for project design flows.
- Water supply system delivery capacity at various reservoir levels.
- Prospective facility improvements.

One issue that the previous study specifically set out to address was the perception by some that water flow rates were limited because they cannot be allowed to exceed a velocity of 10 feet per second (Spink, 1992). The consequence of such a limit would be that flows through the initial leg of the water supply system (i.e., through the dam) would be constrained. However, no technical basis for such a velocity constraint was found. Rather it is believed to be a rule-of-thumb used by designers to select pipe sizes that would then be found to have acceptably low head losses. When a pipe is already in place, however, higher velocities (and head losses) are tolerable (to some extent and under appropriate conditions) before replacing or supplementing the pipe.

In the case of the 84-inch diameter pipe extending through Folsom Dam and to the pump station, implementing Roseville's option for a peak flow rate of 150 cfs would require a total flow rate of 400 cfs (to serve all Folsom Project users) and a velocity of 10.4 feet per second-- only slightly exceeding the rule-of-thumb. The head loss due to slightly exceeding 10 feet per second in this short length of pipe is trivial (approximately 0.35 feet). Even the total head loss to the pumping plant at 400 cfs is relatively minor, amounting to less than 5 feet. Furthermore, this velocity would occur only rarely-- for a few days or weeks in midsummer while all the water agencies were experiencing their peak demands. Thus, it was concluded that head loss is not a limitation.

The initial work also concluded that there is no danger of cavitation or coating erosion at this modest velocity (assuming the coating was properly applied). Indeed, the authors believe that somewhat higher velocities are acceptable, as long as the associated head losses can be accommodated. This means that there is no need for a new or bigger "hole in the dam" to implement Roseville's peak flow increase to 150 cfs. Such an improvement may ultimately be required as demands continue to grow, but it is not necessary at this stage.

Thus, the overall conclusion of the initial work was that Roseville's option for increasing its peak flow can be implemented through appropriate modifications to pumping capabilities at the Folsom Project Pumping Plant. It was further stated that these modifications could be accomplished within the existing pumping plant without major changes to the suction or discharge piping. That is, the existing pump locations should prove adequate.

The purpose of the present work is to follow through on that conclusion by developing a conceptual design and a cost estimate for the needed improvements. This purpose was given more specific meaning in terms of the following project objective:

> To implement Roseville's option for increasing its peak flow entitlement from 65 cfs to 150 cfs by using the existing pumping plant and major piping and achieving total Folsom Dam water supply delivery capacity (including Roseville, San Juan, Folsom, and the prison) of 400 cfs at the lowest practical reservoir water surface elevation (as close to the minimum operating pool, El. 327, as is reasonably possible).

Tasks identified for pursuit of this objective included the following:

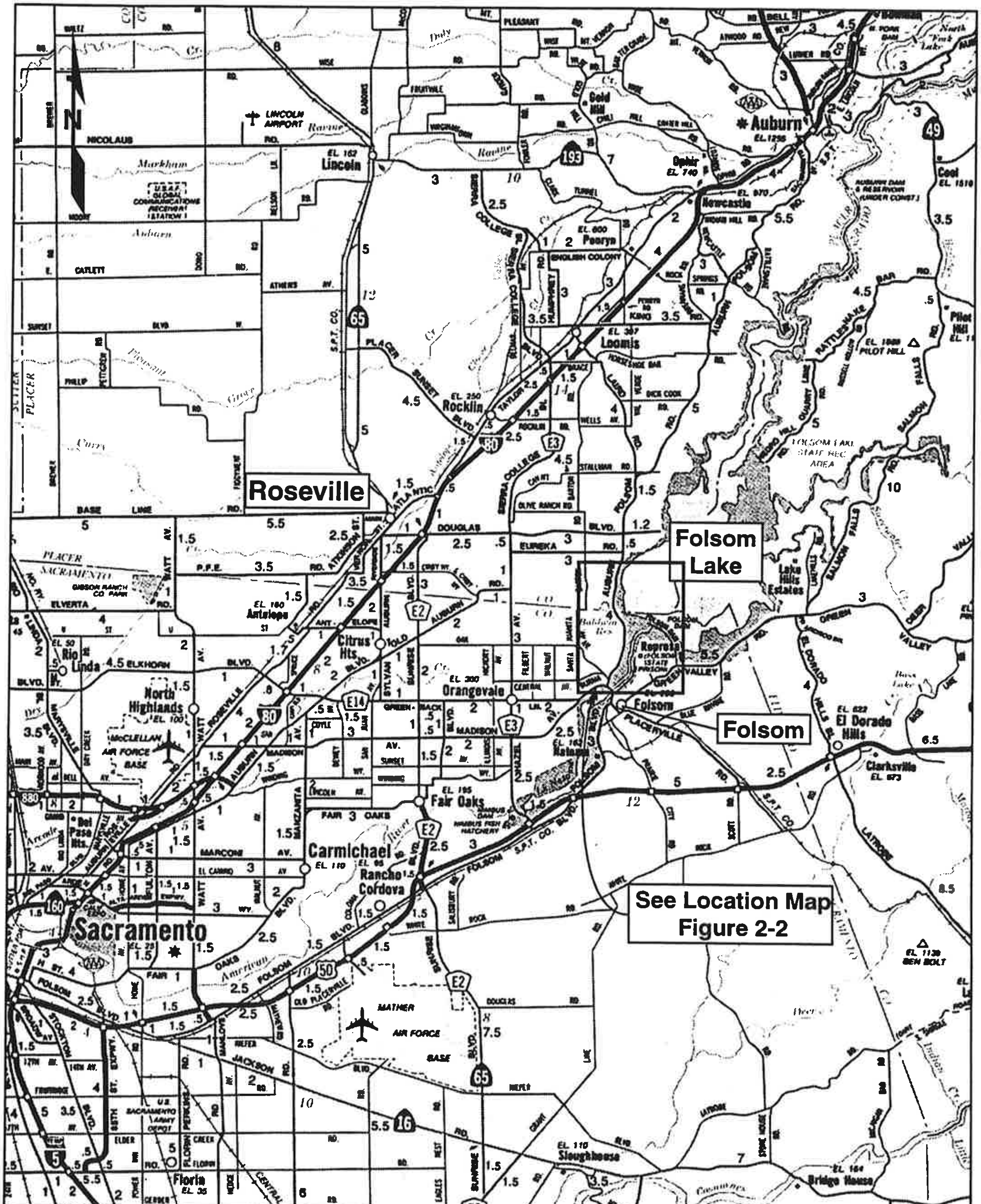
- Performance of a system pumping test to verify or refine the head loss calculations performed during the initial study.
- Research of records for additional specifics regarding aspects of the water supply facilities, particularly regarding electrical aspects.
- Performance of additional calculations on hydraulic arrangements to narrow in on the changes that need to be implemented.
- Development of a facility plan itemizing specific changes to be implemented.
- Provision of a cost estimate for accomplishing the identified improvements.

The following sections of this report document the analyses performed and the resulting facility improvements recommended. ESA wishes to acknowledge and express appreciation for the cooperation and the contributions of many agencies and individuals with whom discussions occurred during this study. Personnel from the Bureau and Corps were very helpful and provided material that was difficult to locate. Bill Joye, Bill Sanford and Bob Beingessner of the Bureau's Folsom office were particularly helpful. The various water supply agencies that use the Folsom Dam water supply facilities (San Juan Water, City of Folsom and Placer County Water Agency, in addition to Roseville) were also cooperative and provided information regarding their facilities and operations. Derrick Whitehead was particularly helpful as the City of Roseville representative responsible for project direction.

Mark Fortner and Joseph Countryman of Murray, Burns and Kienlen provided liaison with the parties mentioned above, obtained most of the data required to perform this study, and organized performance of the system pumping test. They also developed background information on the institutional relationships involved in operation and improvement of the Folsom Project water supply facilities and provide helpful comments on the draft report.

ESA's subconsultant was SAI Engineers, Inc. Ishwar Thakur provided leadership on the electrical issues and was assisted by Harminder Singh. SAI also provided the expertise of Gordon Needham who contributed helpful comments from a mechanical viewpoint.

For ESA, Will Betchart served as Project Manager and Project Engineer, directing the needed hydraulic analyses and preparation of his report. He was assisted by D. "Mike" Namikas who provided the benefit of his many years of hydraulic engineering experience and his familiarity with pumping plant design and operation. Peter Jacke performed most of the hydraulic calculations and, with David O' Shea, produced the technical illustrations that are a central component of this report. Shannon Valera provided word processing services.

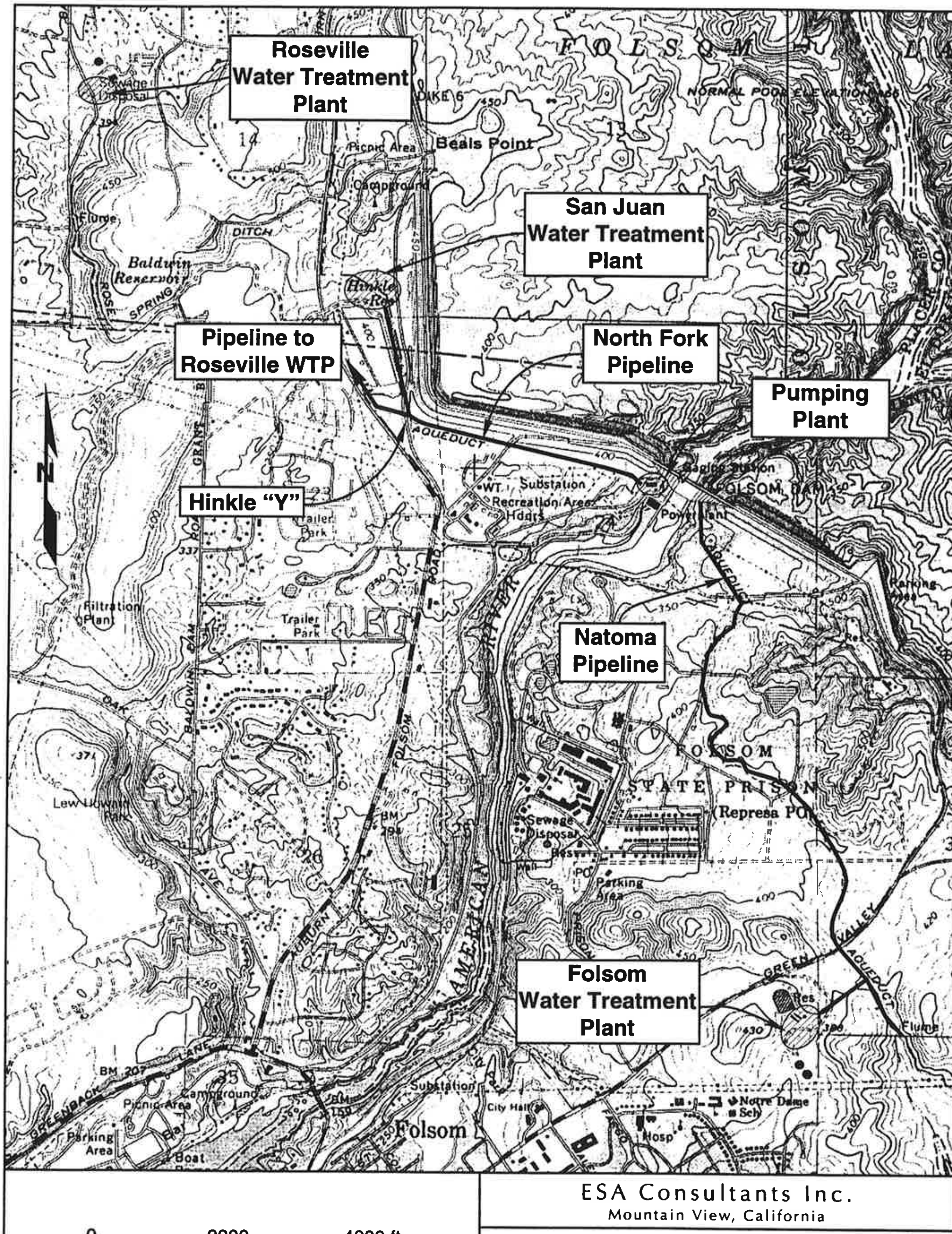


0 5 mi.

ESA Consultants Inc.
Mountain View, California

FOLSOM DAM - WATER SUPPLY PUMPING CAPACITY
VICINITY MAP

Checked By	Date 1/26/96	Project No.	Figure No.
Approved By	Date 1/31/96	043.9501	2-1



0 2000 4000 ft.

Scale 1:24,000

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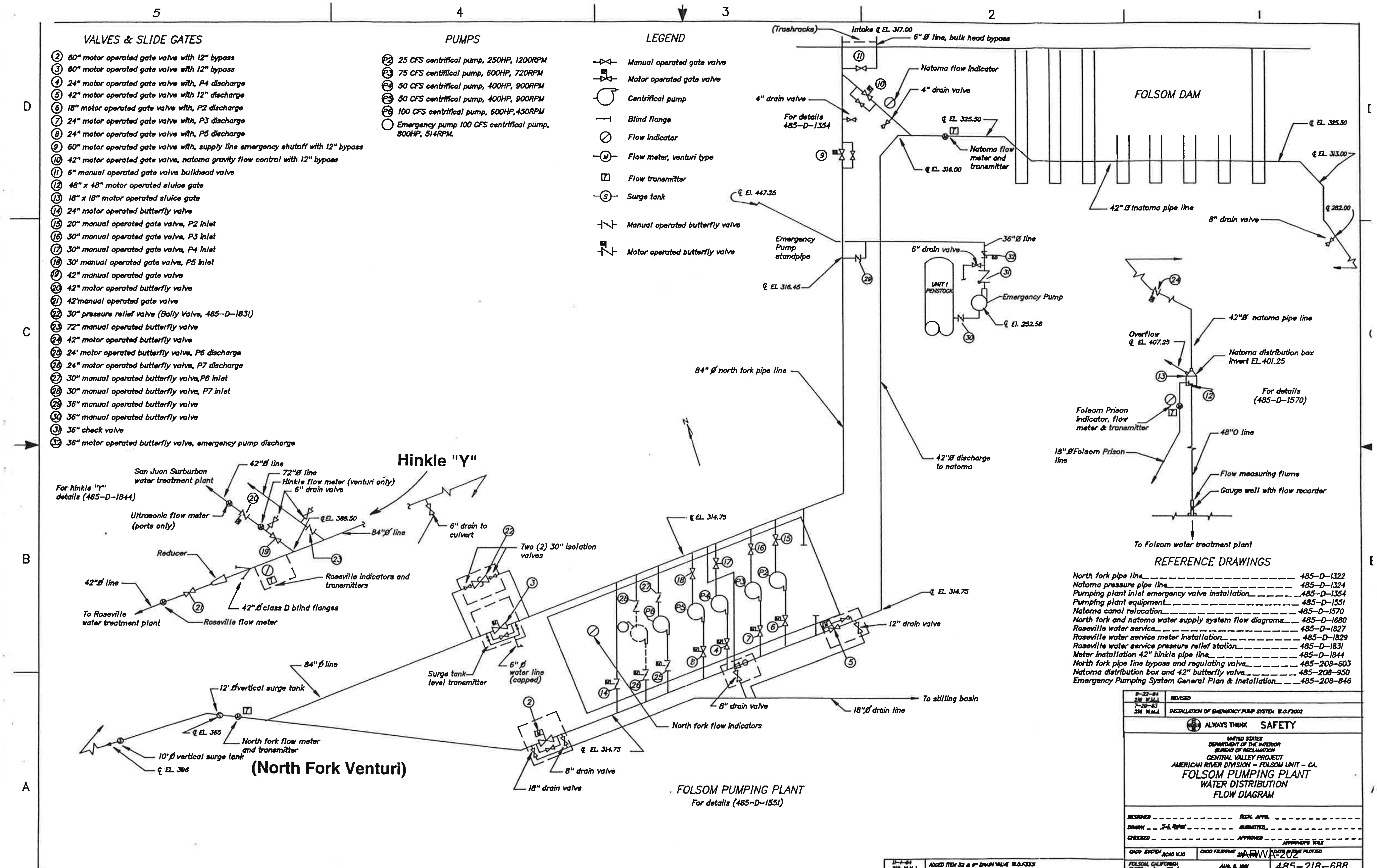
Mountain View, California

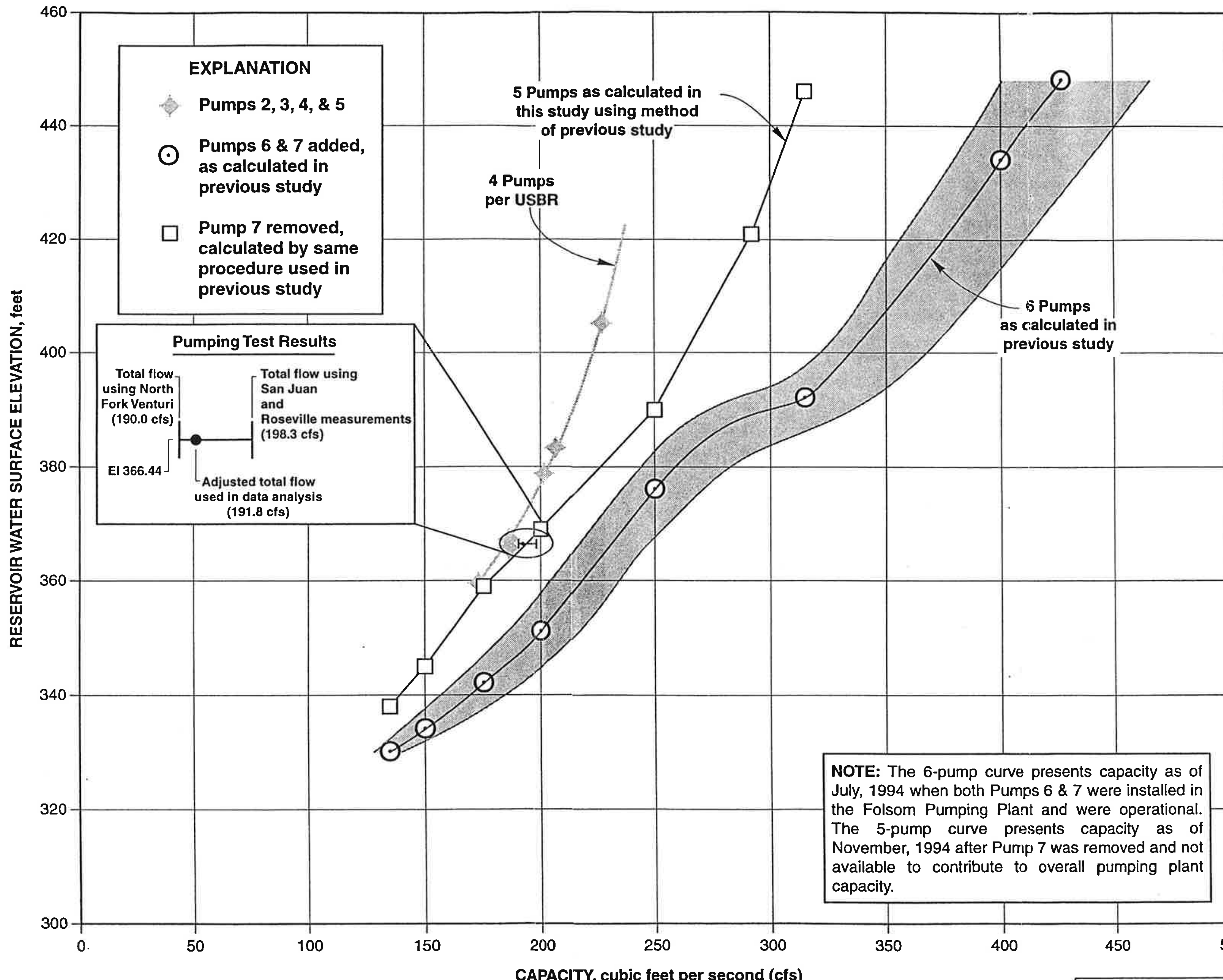
FOLSOM DAM - WATER SUPPLY PUMPING CAPACITY
LOCATION MAP

Checked By Peter V. Jacke	Date 1/26/96	Project No.	Figure No.
Approved By B	Date 1/31/96	043.9501	2-2

ARWA-202



Folsom Dam Water Supply System: Updated Flow Diagram (from USBR) Figure 2-3



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FOLSOM DAM - WATER SUPPLY PUMPING CAPACITY

EXISTING PUMPING CAPACITY AND PUMPING TEST RESULTS

Checked By Peter V. Jacke Date 1/26/96	Project No.	Figure No.
Approved By B Date 1/31/96	043.9501	3-1

3. PUMPING TEST

A pumping test of the Folsom Dam Project water supply system was conducted on November 18, 1994. The details of the test, including preparations, data, and data analyses, are documented in Appendix A. The following summarizes the results.

The primary purpose of the test was to confirm or refine the calculated delivery capacity and head loss findings reported in an earlier study (ESA Consultants, November 1994). Because one of the pumps (Pump No. 7) included in that study had been removed from the pumping plant prior to the test, an initial step was to recalculate the system capacity curve using only the pumps actually available-- i.e. Pumps 2, 3, 4, 5, and 6. The result of the recalculation is presented in Figure 3-1 and the overall result of the pumping test is plotted for comparison.

The total pumping capacity with the reservoir at El. 366.44 was found to be between 190 and 198.3 cfs, depending on whether the measurement from the USBR North Fork venturi was used (190 cfs total) or the summation of Roseville and San Juan flow measurements (198.3 cfs total). The 10 cfs flow to Natoma is included in both totals. For analyzing the pumping test data, a compromise value of 191.8 cfs was used. In any case, the measured capacity was very close to that calculated using the Corps' (1951) predictive calculations of system head losses and the pump manufacturers' discharge versus head curves. The primary sources of variation from the calculated results are thought to be the limited precision in measurements of flow and pressures.

The main conclusions from the pump test are the following:

- The basic calculation approach used in the previous study (ESA Consultants, November 1994) is valid and provides useful results.

- The Corps of Engineers' (1951) head loss predictions for the original portion of the system are remarkably close to actual system performance. The pumping test head loss measurements provided valuable confirmation of the Corps predictions, but the measurements were too variable to refine the Corps predictions. Thus, the Corps calculations continue to be used as the basic head loss characterization for the system. Detailed comparisons of pumping test results with the Corps calculations are provided in Appendix A.

- The test provided an additional basis for calculating the head loss between Hinkle "Y" and San Juan, a portion of the system that was substantially revised and extended after the original system installation. Thus no 1950's Corps head loss calculations are available for this segment of the system. The test data confirmed the approximate magnitude of the head losses previously calculated and were used for minor refinement of the Hinkle "Y" to San Juan head loss parameter.

- The test provided a substantially improved basis for calculating the head loss between Hinkle "Y" and

Roseville. In the previous study, pipe length and rule-of-thumb head loss factors were used to obtain a preliminary estimate. The pump test result shows that head loss in the Roseville line is significantly higher than initially estimated. The most important implication of this measurement is that head loss to Roseville could become the governing factor for system pumping needs under some circumstances. The following examples are indicative:

- If the San Juan system remains unchanged and has peak day flows of 180 to 190 cfs (116 to 123 mgd) as now anticipated, a second 48" diameter pipe to the Roseville Water Treatment Plant (with the same head loss characteristics as the existing Roseville line) and a total flow of 150 cfs for Roseville would result in total head losses that required approximately four feet more pumping head for Roseville than would be required for San Juan.

- If the San Juan system remains unchanged with peak-day flows as above and Roseville installs a 54" diameter second pipe, the resulting head losses with a total flow to Roseville of 150 cfs would likely leave San Juan in the governing position relative to pumping head requirements-- i.e., the pumping head to serve San Juan would continue to exceed that required to serve Roseville..

- If the San Juan system were modified to reduce head losses (e.g., by paralleling the existing segment of single 54" raw water line), then Roseville might have to install an even larger second line in order to avoid the governing position.

Thus, there is an economic issue involving pumping capacity and energy costs to be considered by Roseville when sizing its new raw water line.

4. RECALCULATIONS OF PRESENT SYSTEM HEAD LOSSES

Several developments since the previous study (ESA, November 1994) resulted in the need to refine the calculation approach used to estimate system head losses. These include:

- The removal of Pump 7 from the pumping plant.
- Performance of the pumping test to obtain measurements of head loss.
- Refinements to the data used as a basis for previous calculations.
- The need for a more precise characterization of pumping plant head losses so that alternative improvements to the pumping plant can be evaluated appropriately.

The refined calculation approach is used to specify the system pumping head needed for various flows at any specific reservoir level and, thereby, select additional pumps or modify existing pumps as needed to develop the required increase in pumping capacity.

4.1 Calculation Overview

System head loss occurs in several distinct components in the Folsom Project water supply system. Only those that govern pumping head requirements are addressed here. Those components are:

- The intake/piping system from Folsom Reservoir to the suction header at the pumping plant. The Corps (1951) characterization of head loss was used for this portion of the system.
- The piping and pump system that takes water from the suction header, applies pumping energy and conveys the water into the discharge header. The three subcomponents of this system for each pump are:

    - Suction piping(including turning, entrance, contraction, valve and piping to the pump)
    - Pump (i.e. flange to flange, as characterized by the manufacturer's pump curve)
    - Discharge piping (including piping, valve, expansion and exit)

The head losses through these subcomponents are not dependent on total system water flow, but on the flow through each pump. This, in turn, is dependent on which pumps are running and the total head against which they are pumping. To address this complexity, it is convenient to consider this plant head loss component within the context of each pump's head capacity curve. The manufacturer's pump curve already incorporates head losses within the pump itself. The remainder of the pumping plant losses can be included by adjusting each pump curve for the head loss in that pump's piping system. The calculation of the relevant head losses and development of adjusted pump curves is addressed in more detail in the next section.

- The discharge header and piping system to the junction with the gravity feed bypass. The Corps (1951) characterization was used for this system component.

- The 84" North Fork pipe line from the discharge/gravity junction to Hinkle "Y". The Corps (1951) characterization was used for this system component.

- The feeder line to San Juan from Hinkle "Y". The head loss factor derived from the pumping test results was used for this segment of the system.

Head loss calculations for this study assumed no system hydraulic modifications except in the pumping plant and the installation of a parallel line to Roseville. It was assumed that Roseville's parallel line would be large enough in diameter so that San Juan would continue to govern system pumping requirements, even when Roseville was drawing a full 150 cfs.

In calculating system head losses, it is necessary to assume a specific distribution of flows to the various end users. The present study focuses on enhancing system capacity to 400 cfs responsive to Roseville's increase from 65 cfs to 150 cfs. Thus, the assumed flow distribution was the same as developed for the previous report and was oriented toward this change by using the distribution set forth in Table 4-1. Note that Roseville takes the full increase (85 cfs) as system flow increases from 315 cfs to 400 cfs.

Except for the head loss across the pumping plant, the system head losses under the above flow distribution can be calculated as a function of total system discharge (see Figure 4-1). Then, since the static head change from the reservoir to San Juan can also be calculated for each flow (if the reservoir surface elevation is known), these two components can be combined into a series of curves showing required pumping head versus flow for several reservoir levels (see Figure 4-2).

Note that the required pumping head is referred to as the "adjusted pumping head required". This means that we are referring to the net increase in head supplied by the pumping plant, after allowing (adjusting) for the head losses in the pump's piping system. The actual total pumping head required of each pump will be this adjusted head plus the head losses in that pump's suction and discharge piping. The adjusted pumping head required will be approximately equal for all pumps operating at a given time. The actual total pumping head will vary slightly from pump to pump depending on differences in their piping head losses.

4.2 Head Loss Across Pumping Plant

The Corps (1951) developed calculations for head losses due to the initially installed piping and valves for each pump at one given pump discharge. Based on these calculations, head losses can be estimated for each pump as a function of discharge. The calculated head losses are shown in Figure 4-3. A similar curve for the head loss associated with Pump 6 piping has been developed and included.

The pump manufacturer's head capacity curve for each pump was then taken and adjusted (at each discharge) to show the net head the pump would develop after deducting the head losses across the plant. Thus, the adjusted curve shows the head increase between the suction header and the discharge header. Both the original and the adjusted curves are shown in Figure 4-4. These adjusted curves and the parallel arrangement of the pumps means that the net (adjusted) head across the plant will be essentially equal for all pumps running for a given operating condition. Figure 4-4 allows us to estimate the flow from each pump and sum those flows to obtain the total system capacity for that adjusted head. For example, at 50 feet of "adjusted" pumping head (see Figure 4-4):

Pump 2 = 27.5 cfs
Pump 3 = 65.9 cfs
Pump 4 = 46.8 cfs
Pump 5 = 46.8 cfs
Pump 6 = 62.4 cfs

Total Flow = 249.5 cfs

4.3 Cumulative Head Capacity Curves for the Existing System

Using the adjusted head capacity curves presented in Figure 4-4, a head capacity curve for the existing system as a whole can be developed. Figure 4-5 presents the resulting diagram showing the contribution of each pump (the shaded areas) and the cumulative discharge for the indicated combination of pumps at various adjusted pumping heads.

This curve can then be combined with the system head loss curve to show the system pumping capacity available and required at various reservoir levels. The resultant combined graph is shown in Figure 4-6. Based on a given reservoir level, one can find the flow rate that will be delivered for all pumps operating or a combination of several pumps as indicated in the figure. The system will operate at the intersection of the system curve (for the given reservoir elevation) and the relevant cumulative head-capacity curve (for the combination of pumps operating).

The upper boundary of this curve can be translated into a system capacity curve as developed in the previous ESA study. Figure 4-7 presents the newly calculated system capacity curve in that format showing the curve presented in Figure 3-1 and the pumping test result for comparison. Note that the modified approach for assessing head losses across the pumping plant results in estimates of slightly lower capacities at high and low reservoir levels.

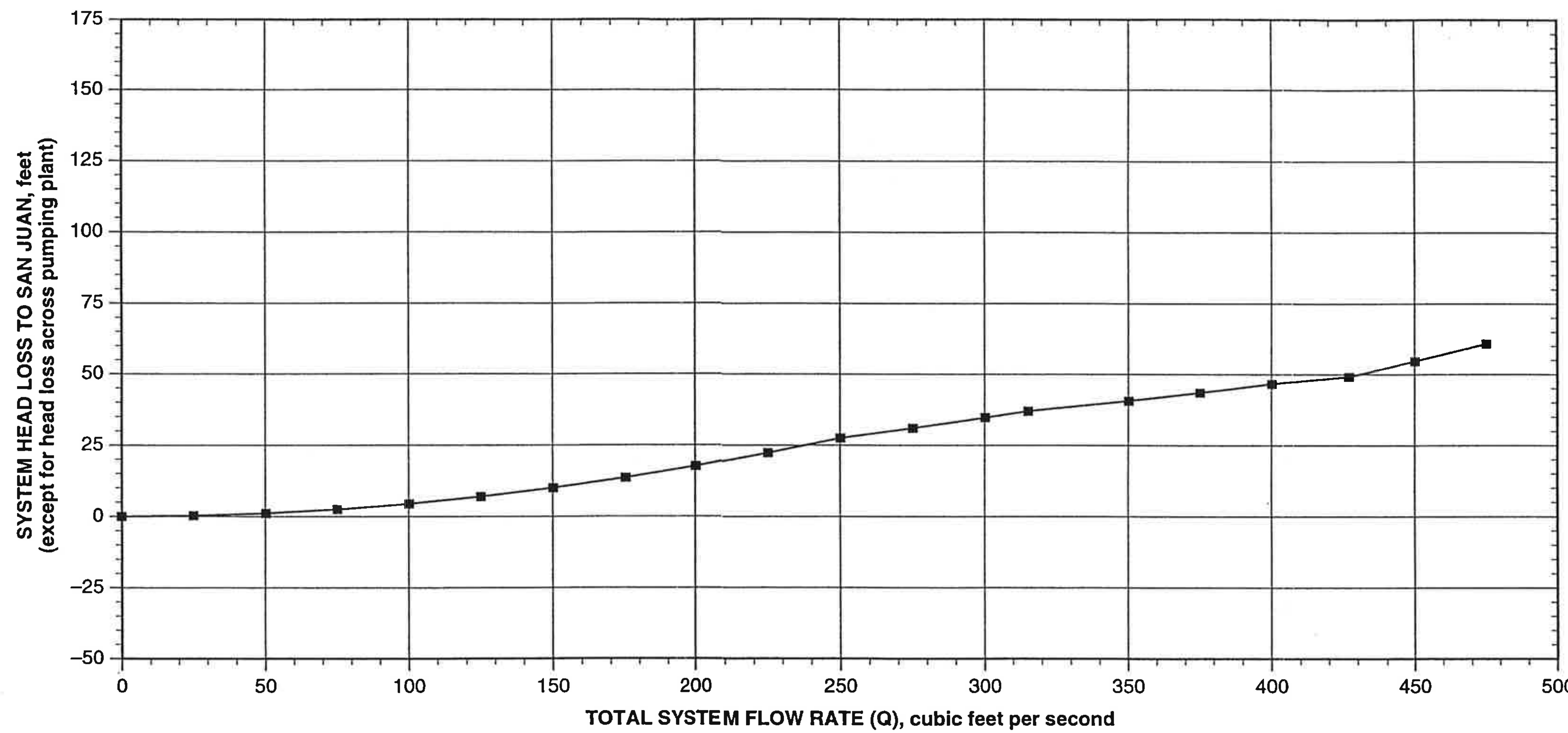
TABLE 4-1

ASSUMED DISTRIBUTION OF SYSTEM FLOW*

Total System Flow Rate	Flow to San Juan		Flow to Roseville		Flow to Natoma	
(cfs)	(cfs)	(%)	(cfs)	(%)	(cfs)	(%)
135	87	64.4%	29	21.5%	19	14.1%
150	97	64.7%	32	21.3%	21	14.0%
175	113	64.6%	37	21.1%	25	14.3%
200	129	64.5%	43	21.5%	28	14.0%
250	161	64.4%	54	21.6%	35	14.0%
315	185	58.7%	65	20.6%	65	20.7%
400	185	46.3%	150	37.5%	65	16.2%
427	190	44.5%	150	35.1%	87	20.4%
> 427		44.5%		35.1%		20.4%

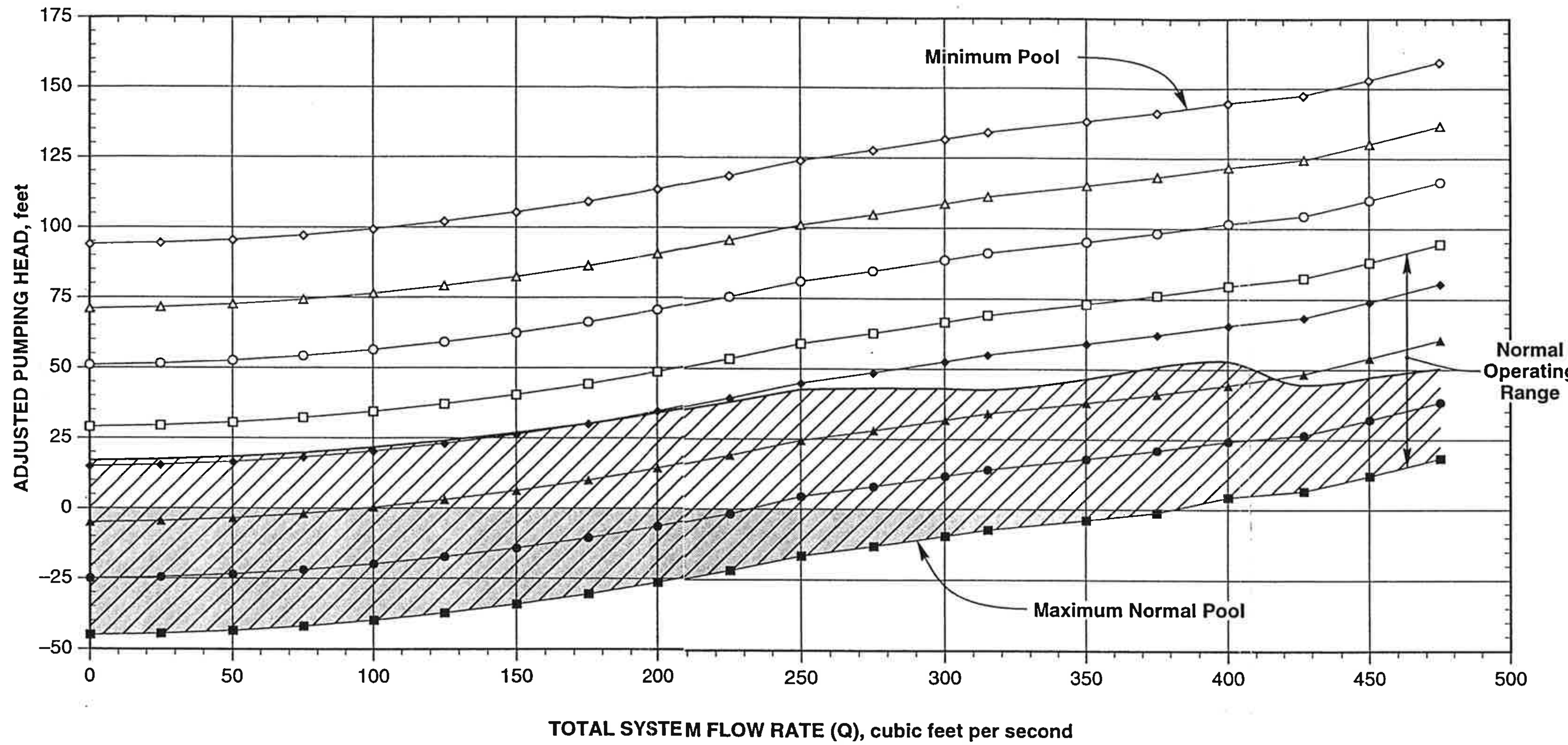
* Flows are based on existing and expected contract amounts.

Note: In review the draft of this report, the City of Folsom's consultant (Robert M. Miles) provided information on water rights, Central Valley Project obligations and non-project water deliveries that could result in a somewhat different distribution table for system flows. Although this would slightly change the head loss calculations presented here, it would not change the study recommendations for pumping plant capacity improvements to address Roseville's objective of increasing their maximum delivery capacity to 150 cfs.



Notes:
1) The head losses included in this curve are those from the reservoir to the centerline pumping plant suction header and from the centerline of the pumping plant discharge header to San Juan.
2) The head losses across the pumping plant (which are not included in this curve) are those that occur in the small diameter pipes to and from the pumps, including entrance, expansion, contraction, valve, and exit losses.
3) Flow proportioning for the system head loss curve is based on existing and proposed contract amounts (see Table 4-1).

ESA Consultants Inc. Mountain View, California		
FOLSOM DAM - WATER SUPPLY PUMPING CAPACITY **SYSTEM HEAD LOSS TO SAN JUAN FOR PUMPED FLOW**		
Checked By Peter V. Jacke Date 1/26/96 Approved By B Date 1/31/96	Project No. 043.9501	Figure No. 4-1



EXPLANATION

System Curves

- Maximum pool - Res. El. 466
- Res. El. 446
- Res. El. 426
- Res. El. 406
- Res. El. 392
- Res. El. 370
- Res. El. 350
- Minimum pool - Res. El. 327

Conditions where gravity flow to Natoma (El 404) are possible.

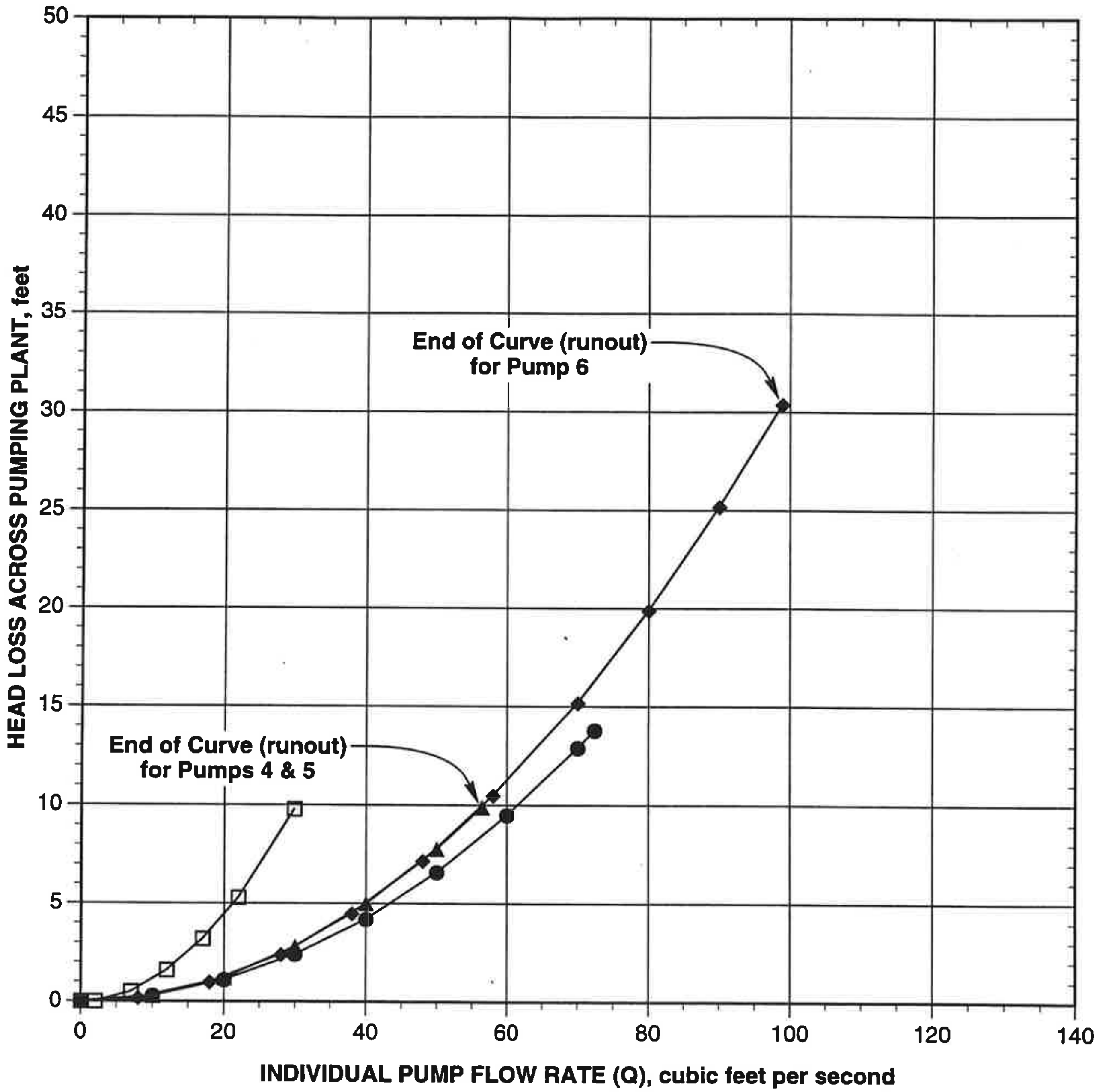
Conditions where gravity flow to San Juan (El 421-425) are possible.

Note: 1) System curves do not include head losses across the pumping plant.

2) System curves are adjusted to reflect head losses occurring during gravity flow conditions to San Juan and/or to Natoma when appropriate.

3) Flow proportioning for the system curves is based on existing and proposed contract amounts (see Table 4-1).

ESA Consultants Inc. Mountain View, California			
FOLSOM DAM - WATER SUPPLY PUMPING CAPACITY **SYSTEM CURVES -- ADJUSTED PUMPING HEAD REQUIREMENTS**			
Checked By Peter V. Jacke	Date 1/26/96	Project No.	Figure No.
Approved By B	Date 1/31/96	043.9501	4-2

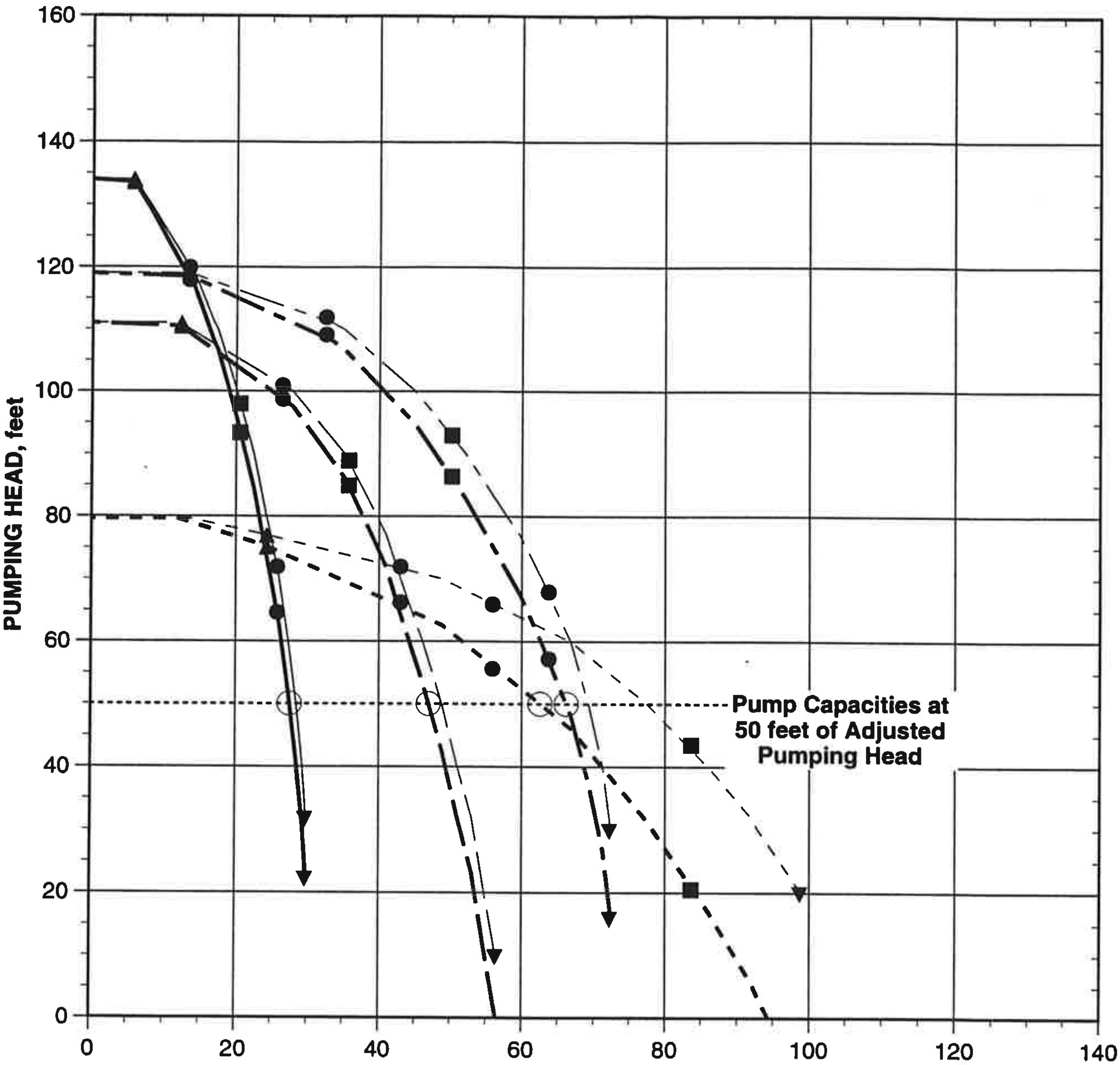


EXPLANATION

- Pump 2
- Pump 3
- Pumps 4 and 5
- Pump 6

Note: 1)The head losses across the pumping plant are those that occur in the small diameter pipes to and from the pumps, including entrance, expansion, contraction, valve, and exit losses.

ESA Consultants Inc. Mountain View, California		
FOLSOM DAM - WATER SUPPLY PUMPING CAPACITY **HEAD LOSS ACROSS PUMPING PLANT vs. FLOW FOR EXISTING CONDITIONS**		
Checked By *Peter J. [signature]* Date *1/26/96* Approved By *[signature]* Date *1/31/96*	Project No. ARWA-202	Figure No. 4-3



EXPLANATION

- Pump 2 (total)
- Pump 2 (adjusted*)
- Pump 3 (total)
- Pump 3 (adjusted*)
- Pumps 4 and 5
- Pumps 4 and 5 (adjusted*)
- Pump 6 (total)
- Pump 6 (adjusted*)

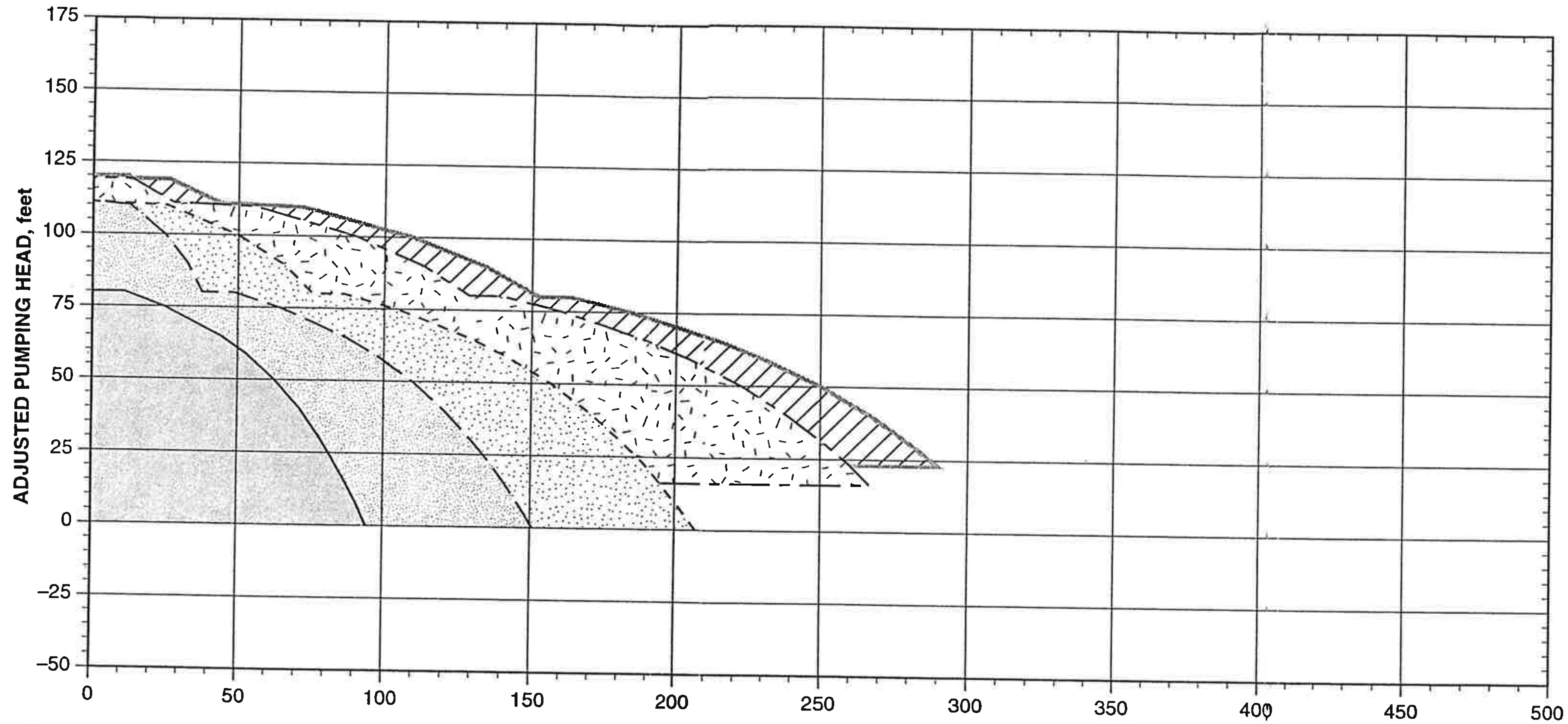
*See Note 1

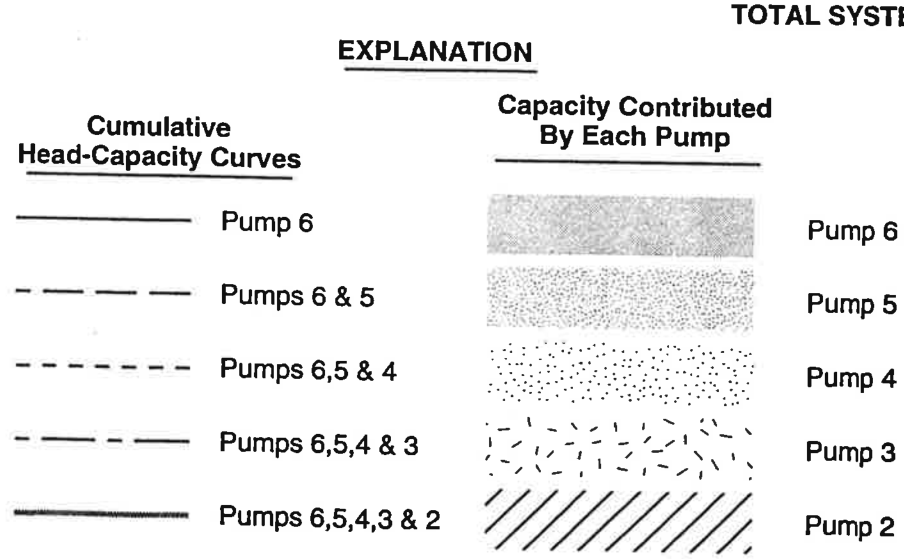
- ■ Best Efficiency Point
- ● 80% Efficiency
- ▲ Shut Off
- ▼ Runout

Notes: 1) Adjusted pumping head for each pump curve is calculated by subtracting the head losses across the pumping plant from the manufacturer's pump curve (see Figure 4-3).

2) The end points of all pump curves have been defined to reflect the expected normal pump operating range.

ESA Consultants Inc. Mountain View, California		
FOLSOM DAM - WATER SUPPLY PUMPING CAPACITY **INDIVIDUAL PUMP HEAD-CAPACITY CURVES ADJUSTED FOR PUMPING PLANT HEAD LOSSES**		
Checked By Peter V. Jacke Date 1/26/96 Approved By [signature] Date 1/31/96	Project No. 043.9501	Figure No. 4-4

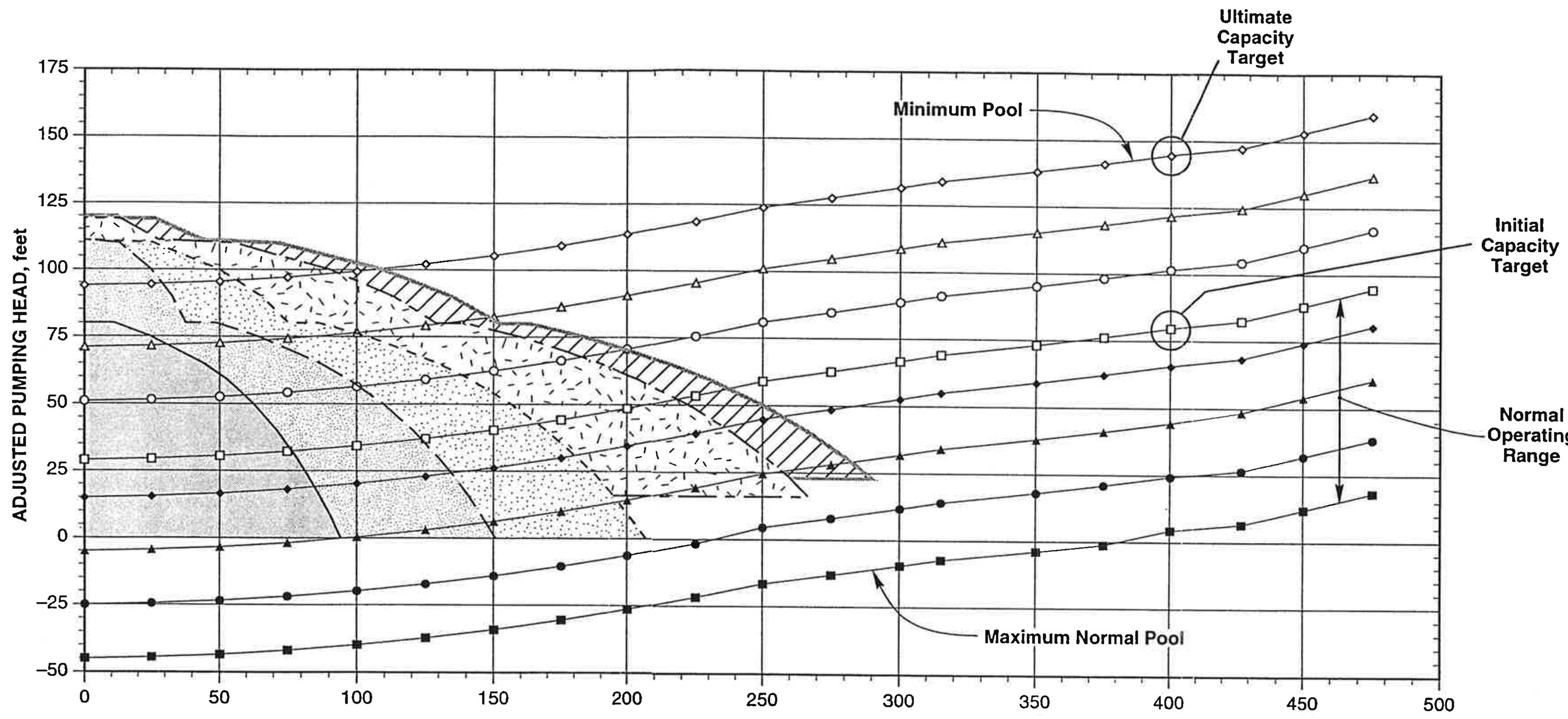




Notes: 1) Adjusted pumping head for each pump curve is calculated by subtracting the head losses across the pumping plant from the manufacturer's pump curve.

2) The end points of all pump curves have been defined to reflect the expected normal pump operating range.

ESA Consultants Inc. Mountain View, California		
FOLSOM DAM - WATER SUPPLY PUMPING CAPACITY **EXISTING HEAD-CAPACITY CURVES**		
Checked By Peter V. Jacke Date 1/26/96 Approved By B Date 1/31/96	Project No. 043.9501	Figure No. 4-5



EXPLANATION

Cumulative Head-Capacity Curves

- Pump 6
- Pumps 6 & 5
- Pumps 6,5 & 4
- Pumps 6,5,4 & 3
- Pumps 6,5,4,3 & 2

Capacity Contributed By Each Pump

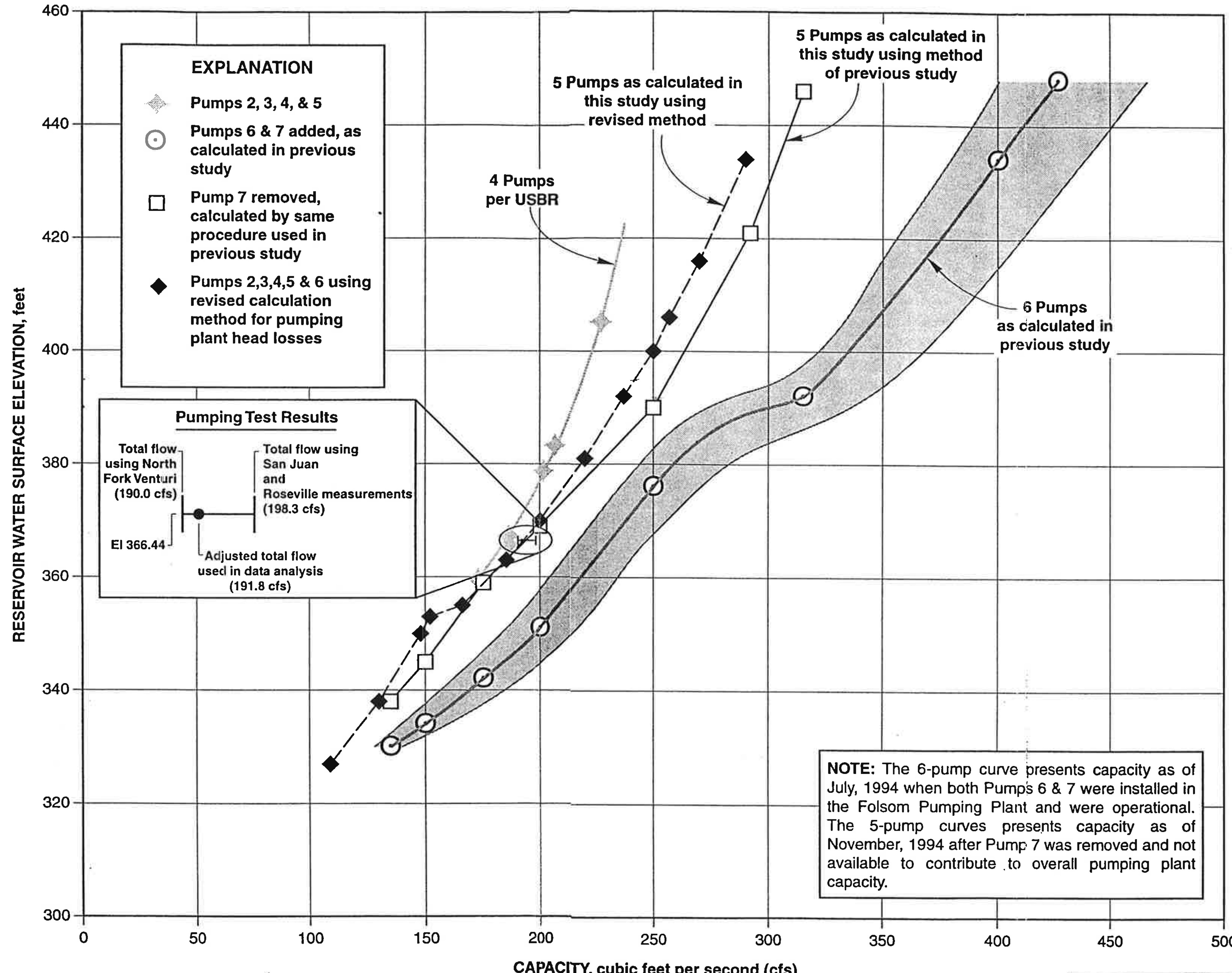
- Pump 6
- Pump 5
- Pump 4
- Pump 3
- Pump 2

System Curves

- Maximum pool - Res. El. 466
- Res. El. 446
- Res. El. 426
- Res. El. 406
- Res. El. 392
- Res. El. 370
- Res. El. 350
- Minimum pool - Res. El. 327

Notes:
1) Adjusted pumping head for each pump curve is calculated by subtracting the head losses across the pumping plant from the manufacturer's pump curve.
2) The end points of all pump curves have been defined to reflect the expected normal pump operating range.
3) System curves do not include head losses across the pumping plant.
4) System curves are adjusted to reflect head losses occurring during gravity flow conditions to San Juan and/or to Natoma when appropriate.
5) Flow proportioning for the system curves is based on existing and proposed contract amounts (see Table 4-1).

ESA Consultants Inc. Mountain View, California		
FOLSOM DAM - WATER SUPPLY PUMPING CAPACITY **EXISTING HEAD- CAPACITY AND SYSTEM CURVES**		
Checked By Peter V. Jacke Date 1/26/96 Approved By B Date 1/31/96	Project No. 043.9501	Figure No. 4-6



ESA Consultants
Mountain View, California

FOLSOM DAM - WATER SUPPLY PUMPING CAPACITY
EXISTING PUMPING CAPACITY (REVISED) AND PUMPING TEST RESULT

Checked By *Peter V. Jacke* Date *1/26/96*	Project No.	Figure No.
Approved By *B* Date *1/31/96*	043.9501	4-7

5. PUMPING CAPACITY IMPROVEMENTS

Preliminary calculations indicated that the vast majority (if not all) of the needed pumping capacity improvements should occur within the pumping plant. Two specific targets were established based on the previous study (ESA Consultants, November 1994) and the scope of work for the present study:

- 400 cfs of delivery capacity when the reservoir is at El. 392. This was based on the system capacity (including Pump 7) that was estimated in the previous study to be 315 cfs at El 392. The idea was that increasing Roseville's peak flow by 85 cfs should not lessen the system's present capability to deliver peak flows under existing contracts. Both the removal of Pump 7 and the revised calculation procedure for head losses would result in changes to this target (see Table 5-1). However, the target of 400 cfs at El. 392 was maintained as stated.

- 400 cfs of delivery capacity when the reservoir is at minimum pool (or as close to minimum pool as practical). The idea of this target was to provide a full water supply contract delivery capability for the potential circumstances where aggressive operation of Folsom Reservoir could result in low water levels much more frequently than experienced in the past.

5.1 Additional Pumps

Primary attention was focused on providing increased pumping capacity by installing additional pumps in the two large-pump positions that are presently not occupied (Nos. 7 and 8).

Pumps from two different manufacturers (Ingersoll-Dresser and Gould) were identified as examples that would be suitable for installation in these positions. The types of pumps to be used would be similar to the existing pumps; they would be:

- single stage
- horizontal shaft
- horizontally split casing
- double-suction
- dual volute
- centrifugal pumps

Required pump capabilities with respect to the performance targets were set as follows:

- For the combination of two added pumps to boost system pumping capacity to 400 cfs at reservoir El. 392, each would need to deliver 118 cfs at 80 feet of adjusted pumping head (see Figure 4-6).

- For the two added pumps to contribute to an ultimate capability of providing 400 cfs at reservoir El. 327, each pump would need to deliver approximately 60 cfs at 140 feet of adjusted pumping head. This assumes an ultimate installation of six large pumps (at 60 cfs each like the two now being considered) and two smaller pumps which combine to provide an additional 40 cfs.

Since the above requirements are indicated in adjusted pumping head, conversions to total pumping head were necessary for discussions with pump manufacturers. Accordingly, the performance targets ultimately developed in terms of total pumping head are:

- 118 cfs at 98 feet
- 60 cfs at 146 feet

Manufacturers identified their applicable pumps as follows:

- Ingersoll-Dresser 750-LNE-1050
  - maximum speed 600 rpm
  - impeller diameter range 32.6 inches to 41.7 inches
  - 36" suction and 30" discharge

  An initial rating was discussed based on 524 rpm and an impeller diameter of 39.9 inches. This rating requires a maximum power of 1415 hp. After detailed hydraulic calculations, slight increases in performance (i.e., 118 cfs at 98 feet)were needed. Within the 1500 hp rating used for electrical considerations this pump can get very close to the target. This can be accomplished by increasing speed or impeller diameter or a combination of both and will be fine tuned during detailed design.

- Gould Pump Model 3420
  - maximum speed 600 rpm
  - impeller diameter range 34" to 46"
  - 42" suction and 30" discharge

  Again, an initial rating was discussed that was slightly less than that required after detailed hydraulic calculations. Within the 1500 hp rating used for electrical aspects, some further fine tuning on speed and impeller diameter will be required to optimize this pump's ability relative to the performance targets. Such fine tuning will occur during detailed design.

All subsequent calculations and analyses have been based on the Ingersoll- Dresser pump as initially rated, simply because that information was available first.

5.2 Pump Performance Curves

Based on data from the manufacturer, pump performance characteristics can be estimated for various pump speeds and impeller diameters. For example, Figure 5-1 shows the curves for the Ingersoll- Dresser pump (750-LNE-1050) at various speeds with a 39.9 inch impeller. Similar curves are shown for a slightly lower speed and larger impeller diameter in Figure 5-2 and a higher speed and smaller impeller diameter in Figure 5-3.

Based on these curves, the pump can be oriented toward the specific application of concern in order to optimize its response to the pump performance targets. Optimizing the choice of speed and impeller diameter in relation to pump power requirements will be addressed in final design.

Of special importance in this study, is consideration of the operating capability of the pumps in terms of the driver (motor system) used. Three major types of drivers are available:

- single speed motors
- two speed motors
- variable speed motors

Figure 5-4 shows the head capacity curve for the pump with a single speed motor (at 590 rpm). The operating range of the pump in this type of installation is along the curve. If the pump is needed for a total pumping head of less than 80 feet, it produces 80 feet of head (or more) and a throttling valve is used to control the system output to the lesser head needed. Similarly, if a lower flow is needed at a particular head, throttling is used to reduce the discharge. With throttling, the pump can serve the combination of heads and discharges shaded in Figure 5-4. However, the throttling dissipates (or wastes) a portion of the energy applied to the hydraulic system by the pump.

Figure 5-5 shows the head-capacity curve for the pump with a two speed motor (at 590 rpm and 505 rpm). The operating range of the pump at its higher speed is identical to the single speed pump discussed above. What the second speed offers is the ability to run at some lower combinations of head and discharge with less throttling (or energy waste). The bold line in Figure 5-5 indicates the capacity of the two speed pump at any given pumping head with no throttling. The pump can still serve the indicated shaded areas under the curves (a slightly larger total area than for the single speed pump). However, the area under the lower speed curve can be served with significantly less energy waste.

Figure 5-6 shows the head-capacity curve for the pump with a variable speed motor (maximum speed of 590 rpm). The operating range of the pump at maximum speed is the same as for the previous examples. However, with the variable speed pump the combinations of head and discharge that can be achieved without throttling are substantial (as indicated by the shading). Similarly, the area indicating combinations that require throttling is much reduced. In the areas that still require throttling, the quantities of energy wasted are also reduced.

Choice of the drivers to be used for the two additional pumps is one of the most significant decisions for conceptual design of the increased pumping capacity at Folsom.

5.3 Pumping Plant Head Loss

Significant head losses are associated with the suction and discharge piping for the two additional pumps. Figure 5-7 shows the across the plant head losses estimated for additional pumps in positions 7 and 8. Cases for existing and enlarged piping sizes are shown. With the existing piping sizes for suction (30 inch) and discharge (24 inch) lines, head losses would amount to nearly 31 feet at 100 cfs and would increase to 43 feet at the 118 cfs target capacity. These head losses are so large that they simply must be reduced where practical.

The other curve in Figure 5-7 shows the maximum practical piping modification to reduce head losses. This would include use of larger pipes and valves (36 inch for suction and 30 inch for discharge) and cutting back the existing cones that connect to the suction and discharge headers. The greatest potential for reducing head loss is from changing the piping sizes (including the header cones) as indicated in Figure 5-8. Of the piping changes, the more important (from a head loss viewpoint) is on the discharge side of the pump because of the large exit loss when flow enters the discharge header. This piping and cone diameter modification to reduce exit velocity will be essential and, fortunately, is relatively easy to accomplish from a constructibility viewpoint; it can be scheduled to occur during a period of gravity operation. On the suction side, the piping modification to the existing cone is less critical and more difficult from a scheduling/constructibility standpoint. If the suction side cones were not modified, but all other modifications were implemented, approximately 1.5 feet of the indicated head loss improvement (at 118 cfs) would be foregone. This can be fine tuned during the final design, based primarily on other construction needs relative to draining the suction header.

For the present study, the total and adjusted head capacity curves for Pumps 7 and 8 were adopted as indicated in Figure 5-9, based on 36 inch piping and valves for suction and 30 inch piping and valves for discharge and including modification of both cones.

5.4 Cumulative Head Capacity Curves

The pump performance curves presented in Section 5.2, as adjusted by the cross plant head loss estimates developed in Section 5.3 can now be used to develop cumulative adjusted pumping head versus capacity curves for the plant. The three different drivers result in distinct head-capacity curves as follows:

- Single speed: Figure 5-10
- Two speed: Figure 5-11
- Variable speed: Figure 5-12

The system head loss curves are also shown on these figures indicating in each case that the proposed pump additions come very close to meeting the target of 400 cfs when the reservoir is at El. 392. Figure 5-13 presents the same information as Figure 5-12, but puts Pumps 7 and 8 on the left side of the figure. This is to facilitate looking at the curves from the viewpoint of operation, assuming that the proposed additional pumps would be operated preferentially because of the flexibility and energy efficiency provided by the variable speed capability.

5.5 Pumping Demand

Although the foregoing sections have provided extensive information on present and prospective pumping capacity, little has been presented on pumping needs. To provide perspective on this topic, the following analysis was performed.

- The 1995 and expected 2020 annual demands for each agency were obtained or estimated (Table 5-2)

- The distribution of each agency's annual demand by calendar month was obtained or estimated (Table 5-2). This distribution was assumed to apply for both 1995 and 2020.

- Based on these inputs, the monthly average demands for total Folsom Dam water supply deliveries could be estimated for 1995 and 2020 (Table 5-3).

- Based on the monthly average demand, the system head loss to San Juan (except across the pumping plant) could be estimated.

- The simulated Folsom monthly storage levels for 70 years of hydrologic record were obtained. The data base used was obtained from Murray Burns and Kienlen and is the output from a computer run dated June 20, 1994 (Run number 4671c) assuming a reoperation flood control pool of 467,000 acre-feet (but allowing for appropriate credits for space available in upstream reservoirs). The input data are set forth in Table 5-4.

- The end of month storage numbers obtained were converted to average storage for each month and then to average lake level for each month. The lake level data are presented in Table 5-5.

- The average lake level was converted to average static pumping head, using San Juan at El. 423.

- The system head losses to San Juan for each calendar month (1995 or 2020) could then be combined with the 70 years of monthly static pumping head requirements to obtain a frequency distribution of adjusted pumping head required versus monthly average total flow rates. The results are presented as follows:

    - 1995 demand in Table 5-6 and Figure 5-14
    - 2020 demand in Table 5-7 and Figure 5-15

- These results can be converted into a contour-type diagram of pumping conditions as demonstrated for the 1995 results in Figure 5-16. Note that where adjusted total pumping head is zero or less, gravity flow conditions prevail and pumping is not needed. The system head loss curves can then be added to the diagram as demonstrated in 5-17 for the 1995 results.

- Finally, the distribution of pumping requirements can be compared to pumping capabilities as shown in Figure 5-18 for 1995 and 5-19 for 2020. These curves show the pumping capability for Pumps 7 and 8, assuming variable speed pumps. Figure 5-20 and 5-21 show the 2020 demands with the single speed and two-speed pumps respectively.

5.6 Comparison of Drive Alternatives

The important observation from the figures presented and discussed above is that installing the variable speed pumps will provide substantial flexibility to serve the pumping needs in the normal reservoir operating range with relatively little throttling and energy waste. However, with either the single speed or two speed pumps, substantial throttling and energy waste will continue.

As one example of the energy efficiency, consider a demand of 400 cfs at reservoir El. 446, which will likely become a relatively normal (post-2020) summer time pumping condition. For this condition an adjusted pumping head of 25 feet is indicated in Figure 5-19. With variable speed pumps 7 and 8, this particular point can be served without throttling, by running pumps 3 through 8 and slightly decreasing the speeds on pumps 7 and 8 from the maximum speed indicated for that pumping head (see Figure 5-19). In contrast, both the single speed (Figure 5-20) and two speed (Figure 5-21) pumps would require significant throttling-- the system, with single speed pumps would have to run at 80 feet of adjusted pumping head and, with the indicated two speed pumps, the system would have run at 65 feet of adjusted pumping head. Energy calculations reveal the comparison of power and energy requirements shown in Table 5-8. The differences are substantial. They indicate that the throttling energy costs can easily amount to tens of thousands of dollars per month.

There are other problems with operating the system at 400 cfs and 25 feet of pumping head. The above calculation (Table 5-8) assumed that the present practice of throttling at San Juan would continue. However, both the single speed and two speed throttling heads indicated above would create too much system head. They would overflow the standpipes and trip the system. Standpipe extensions may not even solve the problem, since exceeding the existing maximum might eventually over pressurize the system. Thus, a different throttling approach that addresses Pumps 7 and 8 individually would be required. First stage throttling would need to be performed using the discharge line valves for these two pumps. Although this would significantly increase the complexity of system operation, it would have the advantage of less throttling energy waste. The existing pumps could be operated near the 25 feet of adjusted pumping head required and only the two new pumps would be throttled. In fact, only one of the new pumps would be needed. With the new single speed pumps, a (pre-throttling) adjusted pumping head of 80 feet would be needed on one pump to throttle to 110 cfs and with the new two speed system, Pump 8 (with its higher low speed) could provide the needed flow at approximately 45 feet of (pre-throttling) adjusted pumping head. The marked changes in power and energy requirements due to individual pump throttling are shown by Table 5-9 (which can be compared to Table 5-8). Although the wasted energy and its cost are reduced by 86 percent and 73 percent in the two speed and single speed cases (respectively), they still constitute $8,400 and $23,000 per month for this operating point. Furthermore this two speed case is one of the more energy efficient throttling circumstances that can be expected.

A specific operating mode (which pumps are on and which pumps are throttling) can be defined for each relevant combination of flow and adjusted pumping head for the three drive alternatives. The power and energy differences could be calculated and summed over the 70 years of monthly operating points available for the two demand years (1995 and 2020). However, even without this effort some conclusions seem obvious:

- High-head pumps with single speed drives are unreasonable. Large amounts of energy would be consumed while throttling these pumps in order to supplement system capacity throughout most of the normal reservoir operating range. If single speed drives are desired, then lower head pumps should be considered even though pumping capacity would be augmented less for low reservoir levels.

- Even with two speed pumps, a new throttling strategy will be required to conserve energy. System flow needs to be limited by throttling a minimal number of pumps in the pumping plant, while avoiding or minimizing throttling at San Juan.

- The difference in cost estimated herein for two two-speed drives compared with two variable speed drives is $290,000 (See Section 10). Using a 25-year life and 8 percent discount rate, the amortization of the cost difference is only $2,240 per month. For reference, this is equivalent to energy consumed for throttling 100 cfs to waste 5.7 feet of head (assuming $0.05/kwh) or 9.5 feet of head (assuming $0.03/kwh). Even if near-term energy savings are modest, there will be some. Furthermore the energy savings are bound to grow as demand increases over time. When convenience of operation is also considered, there is little question that variable speed drives are the system choice.

Detailed water demand data were only recently obtained in refined form. A rigorous assessment of throttling energy cost savings could now be developed to confirm the tentative direction set forth above, if desired.

ESA believes our client's interests are best served by the variable speed drives. We believe detailed analysis to assess future energy savings is not necessary.

TABLE 5-1
SYSTEM CAPACITY BENCHMARKS
UNDER VARIOUS CIRCUMSTANCES

	System Capacity (cfs)	Reservoir Elevation (feet)
As estimated by ESA, November 1994 for Pumps 2 through 7	315	392
Same calculation method; without Pump 7	315 or 253	446 or 392
Revised calculation method; without Pump 7	315 or 237	N/A* or 392

* The existing pumping plant (per pump manufacturer's operating rules and the revised calculation procedure) is not capable of delivering 315 cfs. The maximum pumping capability is 290 cfs at reservoir El 434 (or higher). Under gravity operation, 315 cfs can be delivered when the reservoir is above approximately El 455.

TABLE 5-2
FOLSOM PROJECT WATER
SUPPLY DEMANDS

A. Annual Demand

	1995		2020	
	acre feet	(cfs)	acre feet	(cfs)
Roseville	17,855	24.67	46,950	64.87
San Juan	53,100	73.36	82,200	113.57
City of Folsom	15,500	21.41	34,400	47.52
Folsom Prison	2,172	3 ±	2,900	4 ±
Total	88,627	122.44	166,450	230.0

B. Monthly Demand
(% of annual; estimated)

	Roseville	San Juan	City of Folsom	Folsom Prison
January	4.1	4.2	5.0	7.7
February	3.9	3.5	4.9	7.5
March	4.7	5.5	6.5	7.8
April	6.8	7.7	7.0	8.1
May	10.0	9.3	9.6	8.5
June	11.8	13.6	11.2	8.8
July	13.9	15.4	12.3	9.1
August	13.9	14.5	12.3	9.2
September	11.6	11.5	10.4	8.9
October	8.9	7.7	8.7	8.5
November	5.8	3.6	6.7	8.1
December	4.7	3.6	5.5	7.8
Total	100	100	100	100

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TABLE 5-3
MONTHLY AVERAGE TOTAL
SYSTEM DEMANDS*

	1995 (cfs)	2020 (cfs)
January	63.6	119.2
February	62.6	119.1
March	80.2	149.3
April	110.2	204.1
May	136.2	257.9
June	189.6	350.7
July	207.3	384.5
August	199.8	373.1
September	167.6	314.6
October	117.1	223.2
November	70.0	138.1
December	61.4	118.4
Annual Average	122.4	230.0

* Monthly average total system demands are presented only to illustrate typical pumping requirements. Maximum pumping requirements are dictated by peak day demand and are often estimated to be between 2.0 and 2.3 times average annual demand. For 2020, this would indicate peak day demand between 460 and 530 cfs. Per water agency estimates, 2020 peak day demand for the Folsom Project is expected to be approximately 172(San Juan) + 143(Roseville) + 96(Folsom) + 9(prison) = 420 cfs.

TABLE 5-4
FOLSOM END-OF-MONTH STORAGE
(Folsom Reoperation Study, Run No. 4671c; 467,000 Flood Control Pool)

(in thousand Acre Feet)

Year	Oct	Nov	Dec	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep
1922	425.0	418.9	502.4	527.0	575.0	631.0	800.0	975.0	975.0	843.6	750.7	608.1
1923	586.8	574.0	575.0	575.0	575.0	613.1	800.0	975.0	946.5	777.4	566.2	512.7
1924	375.0	346.0	318.5	283.6	312.5	280.1	327.6	377.3	375.0	360.0	378.1	400.7
1925	362.3	361.8	362.3	355.2	575.0	668.1	800.0	975.0	810.7	615.1	401.6	291.3
1926	200.0	234.8	270.0	269.5	466.2	520.7	800.0	838.1	680.0	450.0	275.0	256.2
1927	255.7	407.5	472.9	574.9	575.0	680.0	800.0	975.0	975.0	793.8	623.5	500.0
1928	480.9	540.8	573.9	575.0	575.0	631.0	800.0	885.6	633.6	555.4	450.0	449.2
1929	415.2	386.2	368.6	329.7	364.6	419.2	497.5	631.3	647.7	652.5	637.3	620.4
1930	571.0	415.7	502.7	542.9	575.0	668.6	768.7	828.4	733.0	475.3	275.0	257.6
1931	242.2	278.0	270.0	281.6	298.3	334.9	347.0	380.5	386.5	359.9	343.0	334.4
1932	299.7	305.9	389.4	465.4	575.0	650.0	750.2	925.4	975.0	950.0	800.0	650.0
1933	600.0	574.0	555.8	512.8	494.0	536.4	610.1	788.4	861.7	842.7	799.9	650.0
1934	600.0	517.6	559.4	575.0	575.0	650.1	687.1	667.6	635.8	603.0	570.5	541.0
1935	484.0	470.9	444.1	497.5	575.0	622.4	800.0	975.0	975.0	789.8	509.0	400.0
1936	375.0	370.0	364.5	575.0	575.0	654.0	800.0	954.7	975.0	811.8	606.4	408.1
1937	375.0	356.0	343.5	342.5	575.0	636.0	800.0	975.0	949.0	750.9	541.7	482.2
1938	375.0	396.0	574.0	575.0	575.0	631.0	800.0	975.0	975.0	950.0	800.0	650.0
1939	600.0	574.0	573.4	561.0	558.8	654.6	747.7	795.5	806.0	794.9	550.7	511.2
1940	476.1	435.0	270.0	575.0	575.0	631.0	800.0	953.8	798.5	600.0	450.0	395.1
1941	387.7	393.6	574.0	575.0	575.0	680.0	800.0	975.0	975.0	946.6	800.0	650.0
1942	600.0	574.0	575.0	575.0	575.0	671.0	800.0	975.0	975.0	950.0	800.0	650.0
1943	600.0	572.0	564.0	554.0	553.0	642.0	800.0	975.0	957.2	787.0	585.7	500.0
1944	478.9	468.8	459.2	441.8	510.8	630.8	694.2	886.0	863.2	655.4	436.7	415.3
1945	378.2	436.1	492.6	504.2	575.0	650.1	750.2	925.4	880.0	691.2	479.3	400.0
1946	375.0	460.0	575.0	575.0	575.0	680.0	800.0	890.1	702.0	600.0	400.0	347.3
1947	341.1	417.8	451.3	415.9	495.9	660.8	792.7	797.2	675.7	450.0	275.0	200.0
1948	231.1	255.9	257.2	322.7	354.4	390.5	720.9	975.0	975.0	798.8	645.8	475.9
1949	385.8	360.3	374.8	354.8	382.7	599.9	799.8	974.9	887.2	641.4	426.7	366.9
1950	353.6	356.5	270.0	485.3	575.0	680.0	800.0	975.0	975.0	810.5	691.9	555.2
1951	442.2	337.0	305.0	306.0	309.0	587.0	800.0	975.0	807.7	600.0	450.0	399.1
1952	409.7	471.6	574.0	575.0	575.0	632.0	800.0	975.0	975.0	950.0	800.0	650.0
1953	600.0	574.0	574.0	575.0	575.0	616.8	767.5	942.6	975.0	950.0	800.0	650.0
1954	600.0	574.0	574.0	575.0	575.0	654.0	800.0	943.5	797.0	600.0	450.0	450.0
1955	420.0	409.9	479.4	557.0	575.0	608.9	695.0	791.3	736.0	468.7	275.0	205.4
1956	207.2	241.0	541.0	398.0	426.0	571.6	750.5	975.0	975.0	950.0	800.0	650.0
1957	600.0	574.0	574.0	572.6	575.0	680.0	752.0	974.0	974.0	818.3	762.2	650.0
1958	600.0	574.0	574.0	575.0	575.0	631.0	800.0	975.0	975.0	950.0	800.0	650.0
1959	600.0	574.0	558.4	575.0	575.0	642.9	759.4	826.0	735.6	600.0	400.0	385.7
1960	358.4	326.3	275.8	297.3	570.9	678.0	800.0	870.5	723.0	450.0	275.0	200.0
1961	200.0	240.8	288.5	278.0	336.7	385.8	475.3	597.0	618.2	450.0	275.0	244.8
1962	226.4	218.2	229.8	228.7	573.2	680.0	800.0	915.5	828.8	600.0	400.0	393.5
1963	600.0	574.0	574.0	575.0	535.0	617.4	800.0	975.0	975.0	778.8	707.5	595.0
1964	583.6	574.0	574.0	575.0	575.0	586.1	684.7	803.6	775.7	507.3	292.4	206.5
1965	209.3	283.1	335.0	330.0	352.0	452.8	800.0	975.0	975.0	821.8	780.2	650.0
1966	600.0	574.0	574.0	575.0	575.0	664.9	795.2	887.7	740.8	600.0	400.0	378.2
1967	351.9	377.8	551.3	575.0	575.0	652.0	800.0	975.0	975.0	950.0	800.0	650.0
1968	600.0	574.0	574.0	575.0	575.0	652.1	744.8	779.5	672.4	552.7	400.0	374.4
1969	346.7	399.7	483.1	575.0	575.0	680.0	800.0	975.0	975.0	950.0	800.0	650.0
1970	600.0	574.0	565.0	336.0	353.0	543.8	644.2	693.0	600.0	565.8	450.0	421.5
1971	388.6	504.6	575.0	575.0	575.0	680.0	799.6	975.0	975.0	931.1	800.0	650.0
1972	600.0	574.0	574.0	575.0	575.0	680.0	780.1	934.6	769.4	600.0	516.0	400.0
1973	375.0	424.0	550.4	575.0	575.0	680.0	787.6	975.0	774.7	600.0	450.0	407.1
1974	398.2	561.0	556.0	452.0	515.0	631.0	800.0	975.0	975.0	950.0	800.0	650.0
1975	600.0	574.0	574.0	575.0	575.0	631.0	740.1	975.0	975.0	950.0	800.0	650.0
1976	600.0	567.0	560.9	535.5	524.3	531.0	535.1	550.6	463.3	360.0	337.7	309.2
1977	272.1	250.8	196.1	156.0	129.6	132.8	146.6	164.3	155.2	131.3	109.9	96.4
1978	89.4	100.8	216.7	575.0	575.0	662.0	800.0	975.0	975.0	806.8	638.1	587.8
1979	548.5	535.3	527.7	575.0	575.0	680.0	793.1	975.0	903.7	682.7	598.4	535.8
1980	523.6	545.4	574.0	421.0	373.0	618.0	783.7	958.8	958.8	950.0	800.0	650.0
1981	600.0	574.0	574.0	575.0	575.0	646.1	744.8	740.8	636.3	450.0	275.0	240.6
1982	213.2	528.0	332.0	356.0	309.0	570.0	800.0	975.0	975.0	950.0	800.0	650.0
1983	600.0	498.0	513.0	523.0	523.0	631.0	800.0	975.0	975.0	975.0	833.9	673.9
1984	600.0	333.0	305.0	308.0	330.0	529.8	660.8	836.3	743.8	600.0	450.0	408.1
1985	369.2	446.2	499.7	493.3	552.3	627.4	727.6	740.6	573.9	450.0	275.0	284.2
1986	247.6	264.3	354.7	546.7	358.0	570.0	766.8	936.2	940.6	728.5	538.8	495.4
1987	435.3	386.2	329.7	297.8	374.7	468.7	517.9	523.5	496.5	423.9	275.0	233.3
1988	174.9	128.8	185.4	281.3	342.7	359.9	374.5	413.6	374.8	359.9	349.9	359.4
1989	284.3	274.8	279.8	298.4	361.1	631.0	800.0	836.9	678.3	463.3	307.6	272.3
1990	246.7	264.5	273.8	303.3	355.9	463.0	519.2	450.0	392.9	359.9	349.9	337.8
1991	309.7	274.9	237.2	206.1	196.6	387.2	499.7	616.5	655.4	643.7	627.2	609.1

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TABLE 5-5
MID MONTH FOLSOM LAKE LEVELS
(Folsom Reoperation Study, 4671c; 467,000 Flood Control Pool)
(ft Above MSL)

Year	Oct	Nov	Dec	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep
1922	406.5	406.0	411.0	418.0	422.5	428.5	440.5	457.5	465.5	459.5	449.0	437.0
1923	428.0	426.0	425.5	425.5	425.5	427.5	440.0	457.5	464.5	455.0	436.0	421.0
1924	409.0	397.0	392.5	387.0	386.5	386.0	387.5	395.5	399.5	398.0	398.0	401.0
1925	400.0	397.0	397.0	396.5	412.0	430.5	442.5	457.5	458.0	440.5	417.5	394.5
1926	376.5	371.0	378.0	381.5	398.0	415.5	435.0	451.0	445.0	424.0	397.0	380.5
1927	379.0	392.0	408.5	419.5	425.5	431.5	443.0	457.5	465.5	457.5	440.0	424.0
1928	415.0	417.5	423.5	425.5	425.5	428.5	440.5	453.5	445.0	427.5	416.5	409.5
1929	407.5	403.0	399.5	395.0	395.0	401.5	411.0	424.0	432.5	434.0	433.0	431.5
1930	427.5	415.5	411.0	419.0	423.5	430.5	441.0	449.0	447.5	428.5	399.0	381.0
1931	377.5	379.5	382.0	382.5	385.0	390.0	394.0	397.5	400.5	399.0	395.5	393.5
1932	390.0	387.5	395.0	406.5	419.0	429.5	439.0	453.0	463.5	464.5	456.5	441.5
1933	431.0	426.5	424.0	420.5	416.5	418.0	425.0	439.0	451.5	454.0	451.5	441.5
1934	431.0	423.5	421.0	424.5	425.5	429.5	436.0	436.5	434.0	430.5	426.5	423.0
1935	418.0	413.5	411.0	412.5	421.0	428.0	440.5	457.5	465.5	457.0	433.5	410.5
1936	401.0	398.5	398.0	412.5	425.5	430.0	442.0	456.5	464.5	458.0	440.0	417.0
1937	401.5	397.5	395.0	394.0	411.0	429.0	441.0	457.5	464.5	454.0	433.5	418.0
1938	407.0	400.5	414.5	425.5	425.5	428.5	440.5	457.5	465.5	464.5	456.5	441.5
1939	431.0	426.5	425.0	424.5	423.5	429.0	439.0	446.5	449.5	449.5	436.0	420.0
1940	415.5	410.5	395.5	406.0	425.5	428.5	440.5	456.5	456.5	439.0	419.5	406.0
1941	401.5	401.5	414.0	425.5	425.5	431.5	443.0	457.5	465.5	464.5	456.0	441.5
1942	431.0	426.5	425.5	425.5	425.5	431.0	443.0	457.5	465.5	464.5	456.5	441.5
1943	431.0	426.5	424.5	423.5	423.0	428.0	441.5	457.5	465.0	456.0	437.5	421.5
1944	415.0	413.0	411.5	410.0	413.0	425.0	435.0	448.0	456.5	445.0	422.0	406.5
1945	402.5	404.0	411.5	416.0	421.0	429.5	439.0	453.0	459.0	448.0	426.5	408.5
1946	401.0	405.5	418.5	425.5	425.5	431.5	443.0	453.5	449.0	434.0	416.5	399.0
1947	394.5	400.0	407.5	407.5	410.5	425.5	442.0	448.5	443.0	424.0	397.0	375.0
1948	370.5	376.5	379.0	385.0	393.5	398.5	423.0	454.0	465.5	457.5	441.5	423.5
1949	407.0	399.0	398.0	397.5	398.0	415.0	439.0	457.5	461.5	445.5	420.5	402.5
1950	397.0	396.0	389.0	399.5	420.0	431.5	443.0	457.5	465.5	458.0	444.5	431.0
1951	416.0	401.5	390.5	388.0	388.0	409.5	438.5	457.5	458.0	439.5	419.5	406.5
1952	403.5	408.5	419.0	425.5	425.5	428.5	441.0	457.5	465.5	464.5	456.5	441.5
1953	431.0	426.5	425.0	425.5	425.5	428.0	438.5	454.5	464.0	464.5	456.5	441.5
1954	431.0	426.5	425.0	425.5	425.5	430.0	442.0	456.0	456.0	439.0	419.5	410.0
1955	407.5	405.0	409.0	418.5	424.5	427.5	434.0	443.5	445.5	428.5	398.5	375.5
1956	368.0	372.0	401.5	412.5	404.5	416.0	435.0	455.0	465.5	464.5	456.5	441.5
1957	431.0	426.5	425.0	425.0	425.0	431.5	441.0	455.5	465.5	458.5	448.5	439.5
1958	431.0	426.5	425.0	425.5	425.5	428.5	440.5	457.5	465.5	464.5	456.5	441.5
1959	431.0	426.5	424.5	424.5	425.5	429.0	439.0	448.5	447.5	435.5	416.5	402.0
1960	398.5	394.0	387.0	384.5	407.5	431.0	443.0	452.5	449.0	426.5	397.0	375.0
1961	366.5	371.5	380.5	384.0	388.0	397.0	407.0	420.5	429.0	420.5	397.0	379.5
1962	374.5	372.0	372.0	373.5	403.0	431.0	443.0	455.0	456.0	440.5	416.5	402.5
1963	416.0	426.5	425.0	425.5	423.0	425.5	440.0	457.5	465.5	456.5	443.5	434.0
1964	427.0	426.0	425.0	425.5	425.5	426.0	432.0	443.5	448.0	433.0	403.0	377.5
1965	368.5	377.0	388.5	392.5	394.0	403.0	431.0	457.5	465.5	458.5	449.5	440.5
1966	431.0	426.5	425.0	425.5	425.5	430.5	442.0	453.0	450.5	436.0	416.5	401.0
1967	397.5	397.5	411.5	424.0	425.5	429.5	442.0	457.5	465.5	464.5	456.5	441.5
1968	431.0	426.5	425.0	425.5	425.5	429.5	439.0	445.5	442.0	429.5	413.5	401.0
1969	397.0	399.0	408.5	420.0	425.5	431.5	443.0	457.5	465.5	464.5	456.5	441.5
1970	431.0	426.5	424.5	410.0	394.5	409.5	427.5	436.0	433.5	426.5	417.5	408.0
1971	403.5	409.5	421.0	425.5	425.5	431.5	443.0	457.5	465.5	463.5	455.5	441.5
1972	431.0	426.5	425.0	425.5	425.5	431.5	442.0	454.5	454.0	437.5	423.5	411.0
1973	401.0	402.5	414.5	424.0	425.5	431.5	442.5	457.0	456.5	438.0	419.5	407.0
1974	403.0	413.5	423.5	417.0	414.0	425.0	440.5	457.5	465.5	464.5	456.5	441.5
1975	431.0	426.5	425.0	425.5	425.5	428.5	437.5	455.0	465.5	464.5	456.5	441.5
1976	431.0	426.5	424.0	422.0	420.0	419.5	420.5	421.5	417.0	404.5	395.0	391.0
1977	385.0	380.0	372.0	360.5	351.0	347.0	350.0	354.5	356.0	351.0	343.0	336.0
1978	331.5	332.5	355.5	402.0	425.5	430.5	442.5	457.5	465.5	458.0	441.5	429.5
1979	424.5	421.5	420.0	422.5	425.5	431.5	443.0	457.5	462.5	448.5	432.5	424.5
1980	420.0	420.5	423.5	416.0	402.5	415.5	439.0	456.0	464.0	463.5	456.5	441.5
1981	431.0	426.5	425.0	425.5	425.5	429.5	438.5	443.5	438.0	421.5	397.0	379.0
1982	373.0	398.5	407.0	394.5	392.5	408.5	437.5	457.5	465.5	464.5	456.5	441.5
1983	431.0	422.5	417.0	418.5	419.0	425.5	440.5	457.5	465.5	465.5	459.0	444.5
1984	432.5	412.0	390.0	388.0	390.0	407.0	427.5	444.0	448.0	436.0	419.5	407.0
1985	401.0	404.0	413.0	416.0	419.0	427.0	436.5	442.5	434.5	418.0	397.0	383.0
1986	380.5	379.0	388.5	410.0	410.0	411.5	435.5	454.0	462.5	452.5	432.0	418.5
1987	412.0	404.5	396.5	389.5	393.0	406.0	415.5	419.0	417.5	411.0	395.0	378.5
1988	367.5	353.5	355.0	374.0	389.0	395.5	398.0	402.0	402.0	398.0	396.0	396.0
1989	390.5	383.0	383.0	385.0	392.0	416.0	440.5	451.0	445.0	425.0	400.5	385.0
1990	379.5	378.5	381.5	385.0	392.0	404.0	415.0	414.5	406.0	399.5	396.0	394.0
1991	391.0	385.5	379.0	371.5	367.0	385.5	409.0	423.5	432.0	433.5	432.0	430.5

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TABLE 5-6
1995 PUMPING DEMAND FREQUENCY DISTRIBUTION*
(based on Folsom reoperation and 70 years of record)

Adjusted Pumping Head Ranges (ft)	Average Monthly Q Ranges (cfs) 25-50	50-75	75-100	100-125	125-150	150-175	175-200	200-225	225-250
100-110	0	0	0	0	0	0	0	0	0
90-100	0	1	0	1	0	1	1	1	0
80-90	0	0	0	0	0	0	1	0	0
70-80	0	2	1	1	1	0	0	0	0
60-70	0	3	0	2	0	0	0	0	0
50-60	0	10	0	7	0	10	0	0	0
40-50	0	17	0	5	0	4	14	4	0
30-40	0	33	3	8	2	9	6	2	0
20-30	0	38	6	19	1	11	15	2	0
10-20	0	41	7	11	2	9	5	11	0
0-10	0	48	8	10	4	6	16	14	0
-10-0	0	87	45	31	3	19	19	5	0
-20--10	0	0	0	45	11	1	30	15	0
-30--20	0	0	0	0	46	0	33	16	0
-40--30	0	0	0	0	0	0	0	0	0

*Note: Values indicate the number of months a given pumping condition is expected to be encountered out of 70 years (840 months) assuming Folsom reoperation and 1995 average monthly demands.

TABLE 5-7
2020 PUMPING DEMAND FREQUENCY DISTRIBUTION*
(based on Folsom reoperation and 70 years of record)

Adjusted Pumping Head Ranges (ft)	Average Monthly Q Ranges (cfs)							
	50-100	100-150	150-200	200-250	250-300	300-350	350-400	400-450
120-130	0	0	0	0	0	0	2	0
110-120	0	0	0	0	0	1	0	0
100-110	0	0	0	1	0	0	1	0
90-100	0	1	0	0	0	0	0	0
80-90	0	1	0	1	1	0	0	0
70-80	0	4	0	1	0	10	15	0
60-70	0	1	0	7	0	3	10	0
50-60	0	11	0	5	0	10	12	0
40-50	0	24	0	6	3	11	22	0
30-40	0	40	0	17	0	8	28	0
20-30	0	42	0	16	4	6	15	0
10-20	0	49	0	6	2	20	48	0
0-10	0	152	0	32	3	1	31	0
-10-0	0	25	0	48	13	0	26	0
-20--10	0	0	0	0	44	0	0	0
-30--20	0	0	0	0	0	0	0	0

*Note: Values indicate the number of months a given pumping condition is expected to be encountered out of 70 years (840 months) assuming Folsom reoperation and estimated 2020 average monthly demands.

TABLE 5-8
PUMP DRIVE COMPARISON
ASSUMING THROTTLING AT DELIVERY POINT
(at 400 cfs and 25 ft of adjusted head)

	Adjusted Head Before Throttling (ft)	Power Required (MW)	Monthly Energy Required (million kwh)	Extra Monthly Energy Required for Throttling (million kwh)	Extra Monthly Energy Cost for Throttling (@ 5¢/kwh)
Variable Speed	25	1.06	0.76	base	base
Two Speed*	65	2.75	1.98	1.22	$61,000
Single Speed**	80	3.38	2.44	1.68	$84,000

* With two new two-speed pumps and throttling at San Juan (and other delivery points), all the pumped water would be lifted 65 feet, then at least 40 feet of head would be dissipated by throttling.

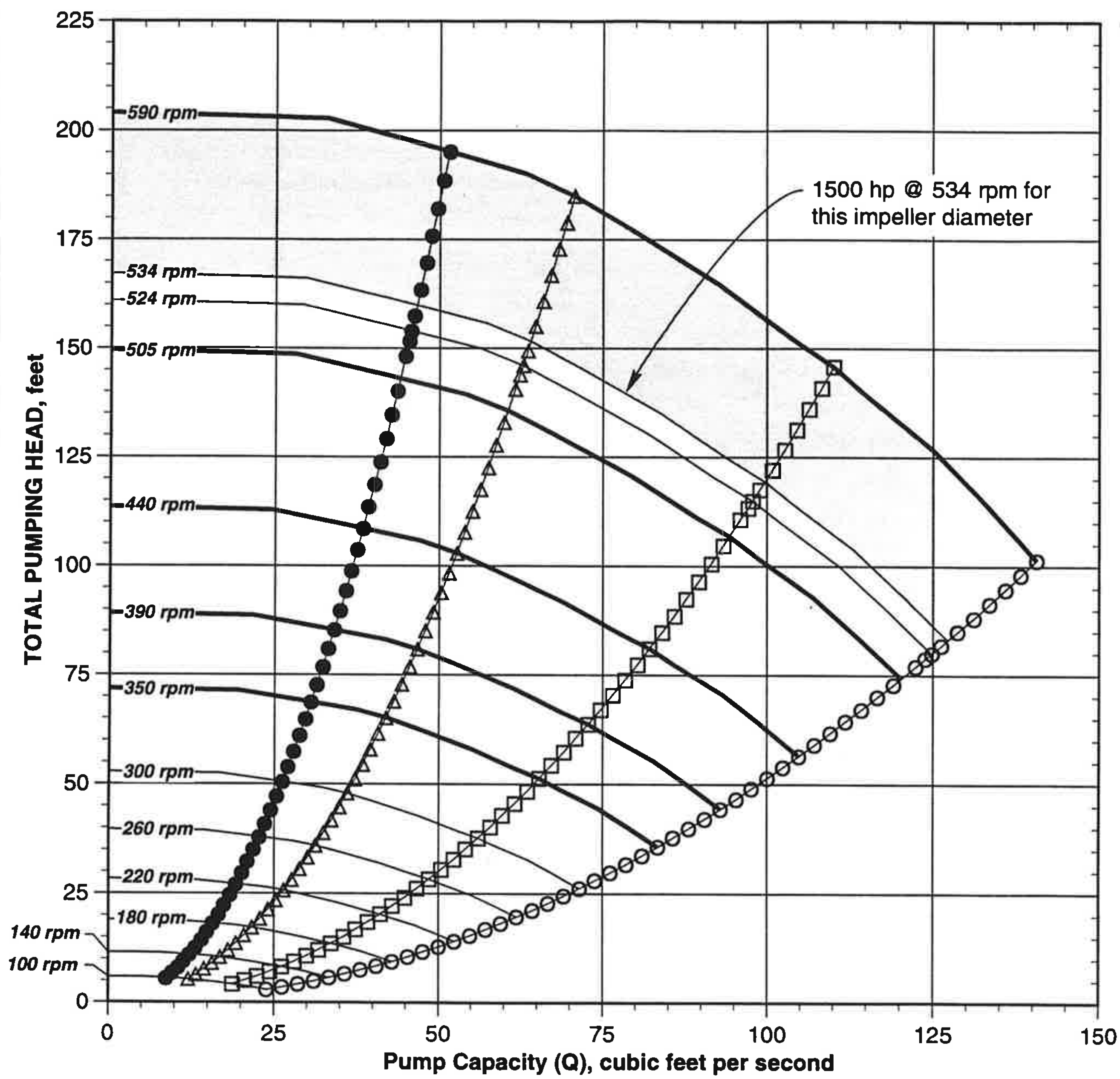
** With two new single-speed pumps and throttling at San Juan, all the pumped water would be lifted 80 feet, then at least 55 feet of head would be dissipated by throttling.

TABLE 5-9
PUMP DRIVE COMPARISON
ASSUMING THROTTLING OF INDIVIDUAL PUMPS
(at 400 cfs and 25 ft of adjusted head)

	Adjusted Heads Old/New Pumps (ft)	Power Required (MW)	Monthly Energy Required (million kwh)	Extra Monthly Energy Required for Throttling (million kwh)	Extra Monthly Energy Cost for Throttling (@ 5¢/kwh)
Variable Speed	25/25	1.06	0.76	base	base
Two Speed*	25/45	1.29	0.93	0.17	$8,400
Single Speed**	25/80	1.70	1.22	0.46	$23,000

* With throttling of individual pumps at the pumping plant and no throttling at San Juan, only one of the new two speed pumps would need to operate; it could operate at its lower speed and provide 45 feet of adjusted pumping head (of which 20 feet would be dissipated by throttling). However, all of the existing pumps could be operated at 25 feet of adjusted pumping head and high flow, with no throttling.

** One single speed pump would operate at 80 feet of adjusted pumping head and its flow would be throttled in the pumping plant to dissipate 55 feet of head. However, all of the other pumps could contribute high flows without throttling.

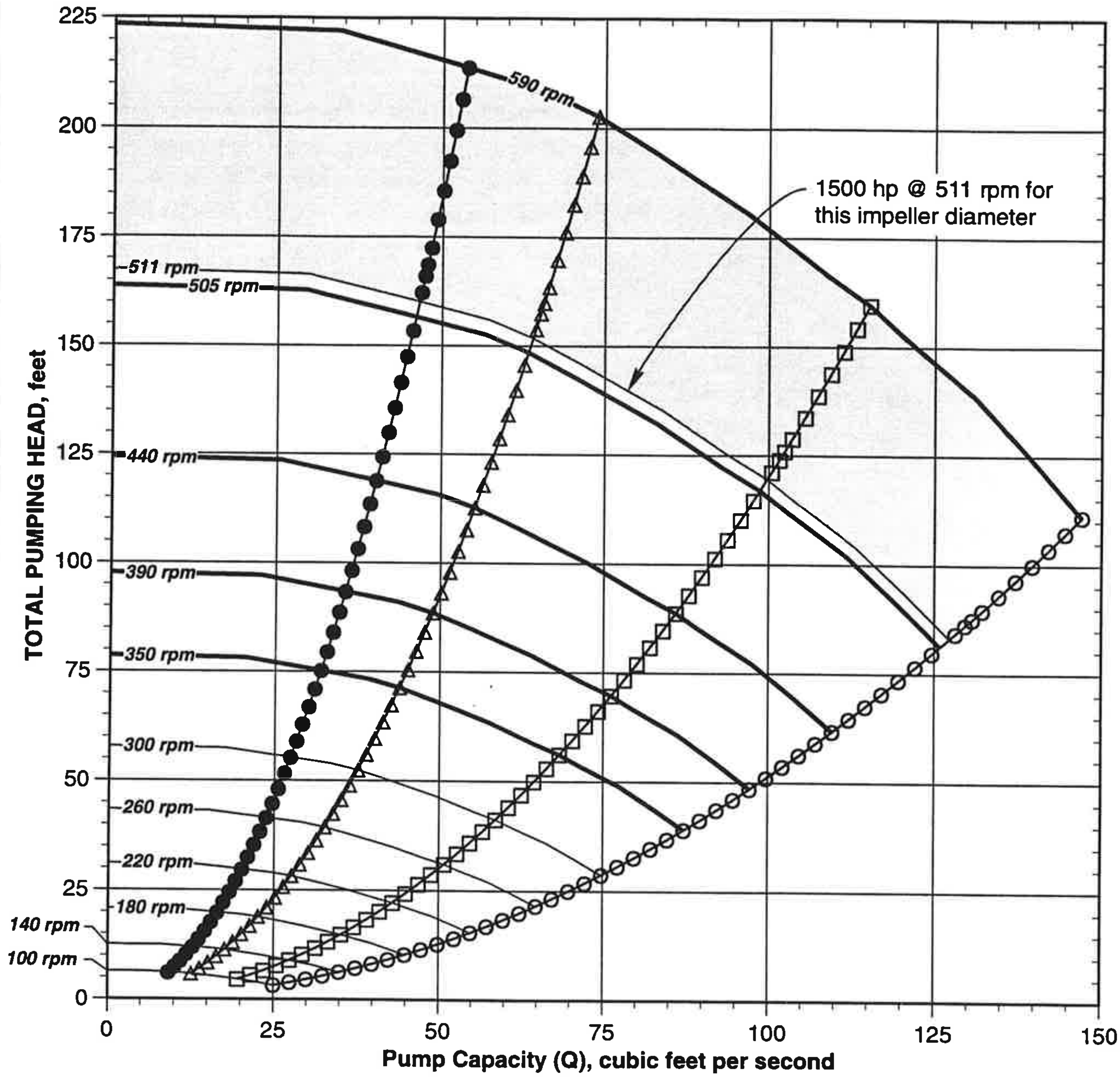


EXPLANATION

- Best Efficiency Point
- 80% Efficiency
- Shut Off
- Runout
- Operation in this range will be limited by motor horsepower.

Notes: 1) Based on manufacturer's curve for 750-LNE-1050 Pump; 524 rpm; 39.9" impeller; maximum power 1415 hp.
2) Values of total pumping head are not adjusted for head losses across the pumping plant.

ESA Consultants Inc. Mountain View, California		
FOLSOM DAM - WATER SUPPLY PUMPING CAPACITY **VARIABLE SPEED PUMP PERFORMANCE CURVES** **39.9 INCH IMPELLER**		
Checked By *Peter V. Joch* Date *1/26/96* Approved By *B* Date *1/31/96*	Project No. **043.9501**	Figure No. **5-1**



EXPLANATION

- Best Efficiency Point
- 80% Efficiency
- Shut Off
- Runout
- Operation in this range will be limited by motor horsepower.

Notes: 1) Based on manufacturer's curve for 750-LNE-1050 Pump (see Figure 5-1); converted to show 505 rpm and 41.7" impeller; maximum power 1454 hp.
2) Values of total pumping head are not adjusted for head losses across the pumping plant.

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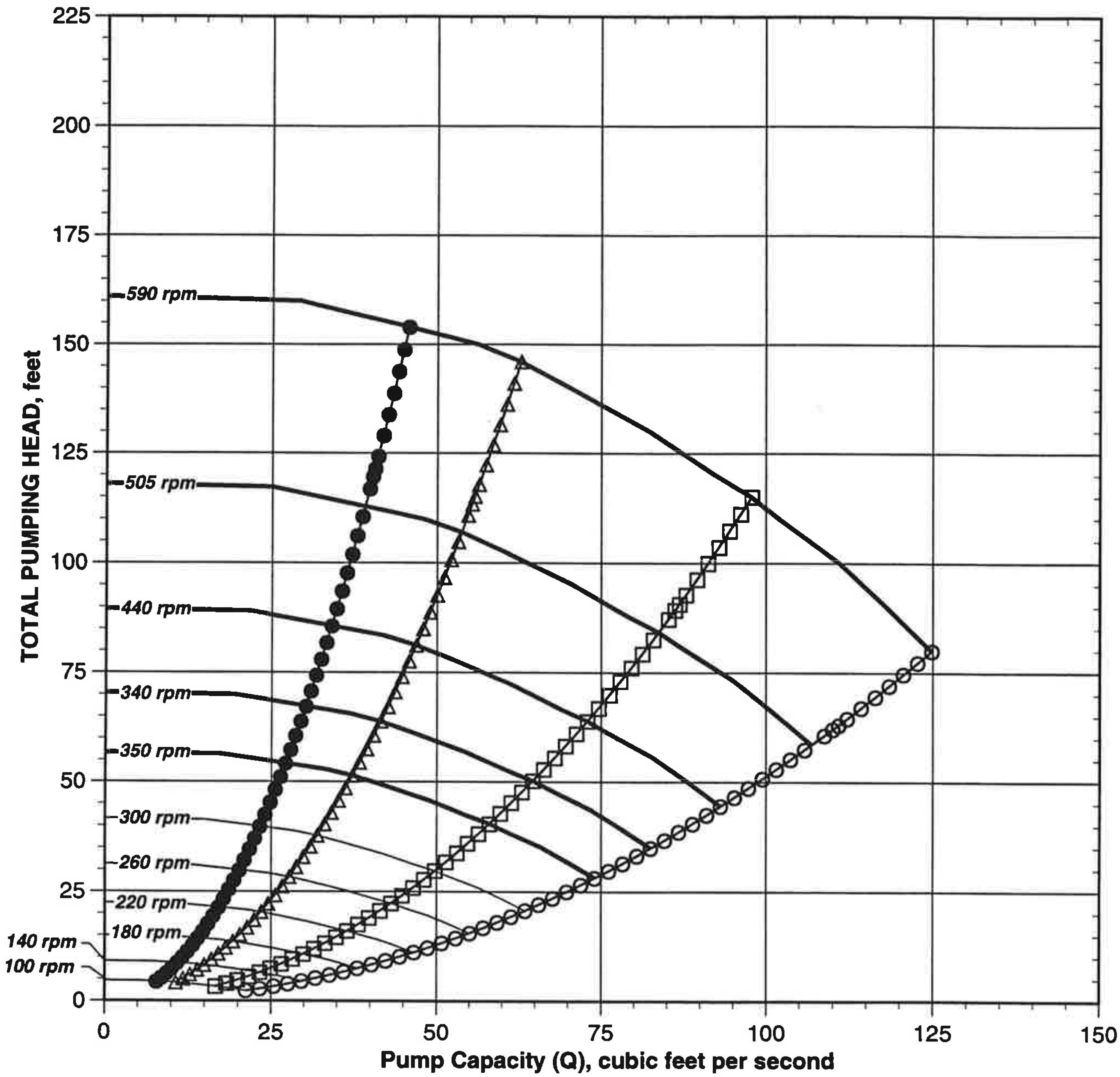
Mountain View, California

FOLSOM DAM - WATER SUPPLY PUMPING CAPACITY
VARIABLE SPEED PUMP PERFORMANCE CURVES
41.7 INCH IMPELLER

Checked By Peter V. Jade Date 1/26/96	Project No.	Figure No.
Approved By B Date 1/31/96	043.9501	5-2

ARWA-202



EXPLANATION

- Best Efficiency Point
- 80% Efficiency
- Shut Off
- Runout

Notes: 1) Based on manufacturer's curve for 750-LNE-1050 Pump (see Figure 5-1); converted to show 590 rpm and 35.4" impeller; maximum power 1500 hp.
2) Values of total pumping head are not adjusted for head losses across the pumping plant.

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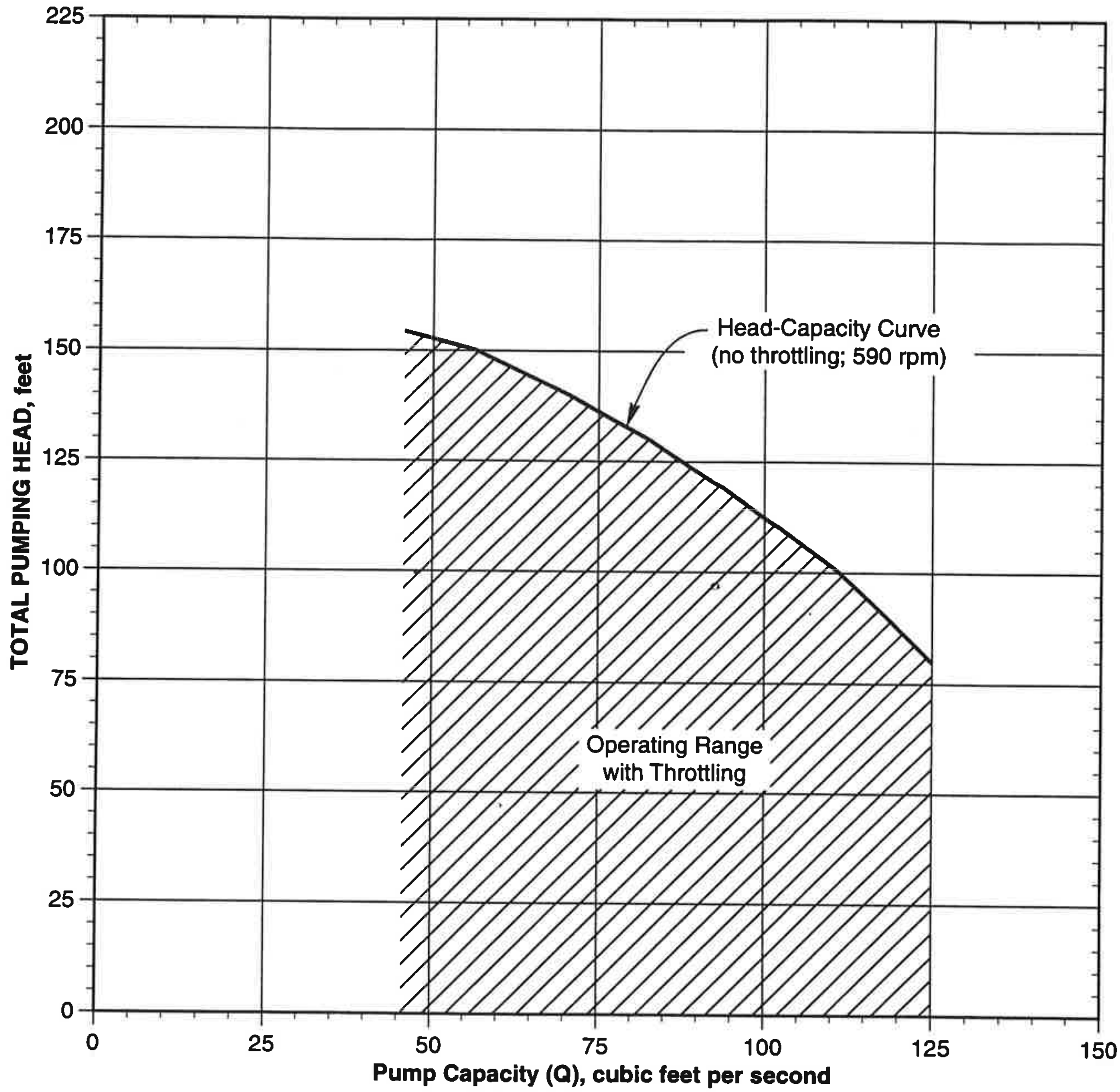
FOLSOM DAM - WATER SUPPLY PUMPING CAPACITY
VARIABLE SPEED PUMP PERFORMANCE CURVES
35.4 INCH IMPELLER

Checked By Peter V. Jocke	Date 1/26/96	Project No.	Figure No.
Approved By B	Date 1/31/96	043.9501	5-3



Notes: 1) Based on manufacturer's curve for 750-LNE-1050 Pump (see Figure 5-1); converted to show 590 rpm and 35.4" impeller; maximum power 1500 hp.
2) Values of total pumping head are not adjusted for head losses across the pumping plant.

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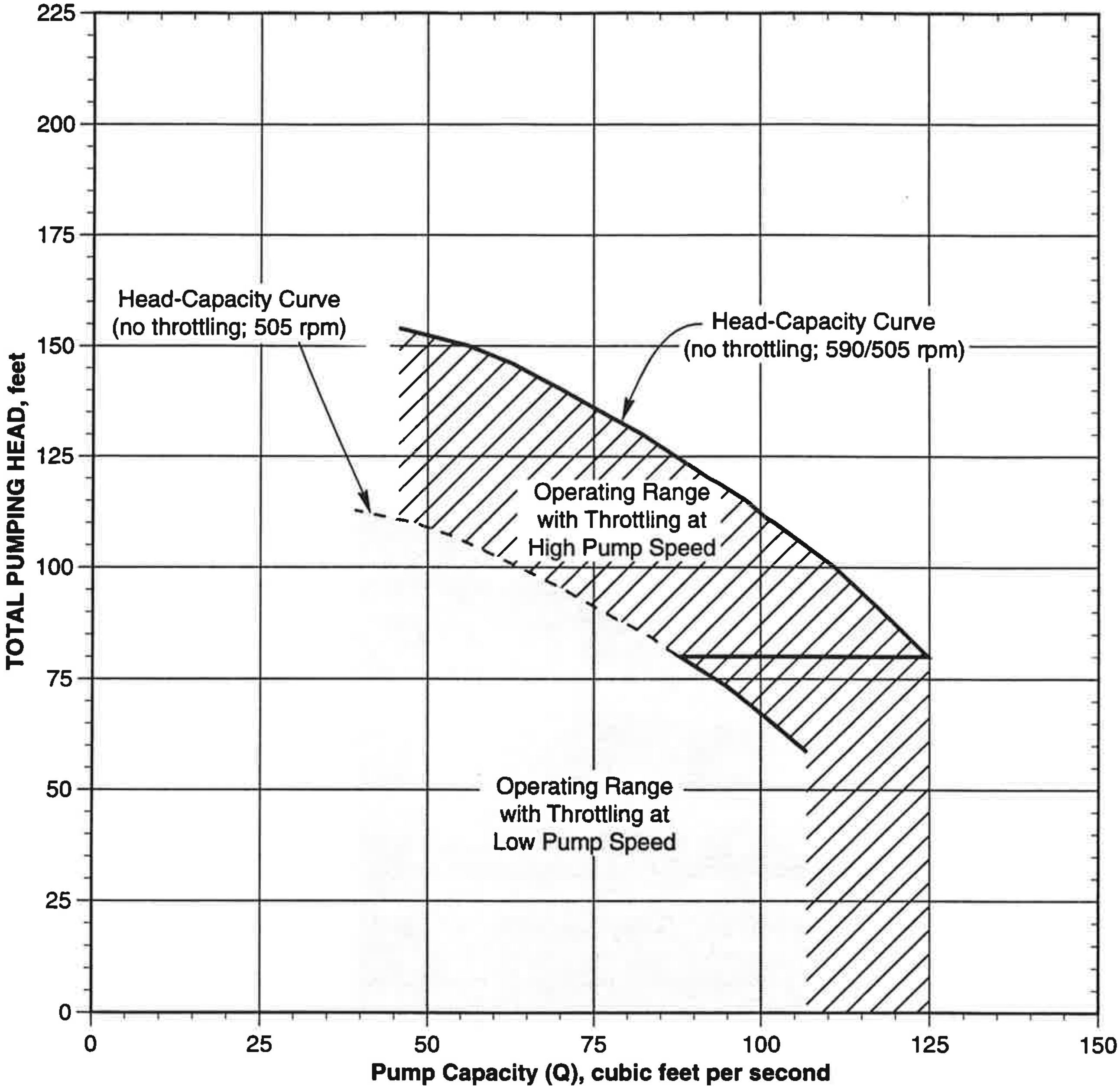
FOLSOM DAM - WATER SUPPLY PUMPING CAPACITY

SINGLE-SPEED PUMP OPERATING RANGE

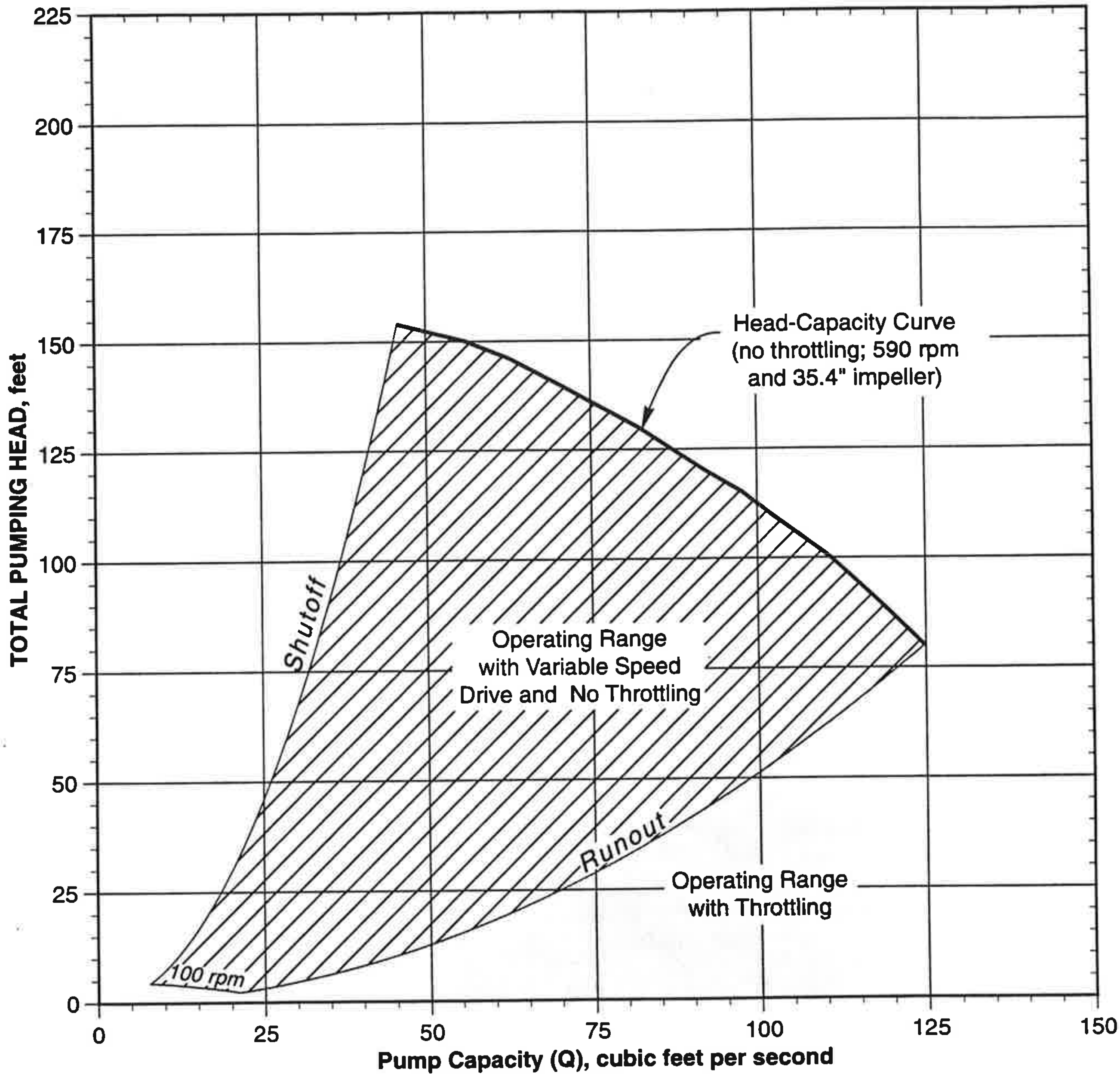
		Project No.	Figure No.
Checked By Peter V. Yoder	Date 1/26/96	043.9501	5-4
Approved By B	Date 1/31/96		



Notes: 1) **Based on manufacturer's curve for 750-LNE-1050 Pump (see Figure 5-1); converted to show 590 rpm and 505 rpm and 35.4" impeller; maximum power 1500 hp.**

2) **Values of total pumping head are not adjusted for head losses across the pumping plant.**

ESA Consultants Inc. Mountain View, California		
FOLSOM DAM - WATER SUPPLY PUMPING CAPACITY **TWO-SPEED PUMP OPERATING RANGE**		
Checked By Peter V. Jacke Date 1/26/96 Approved By [signature] Date 1/31/96	Project No. 043.9501	Figure No. 5-5



Notes: 1) Based on manufacturer's curve for 750-LNE-1050 Pump (see Figure 5-1); converted to various values between 100 and 590 rpm and 35.4" impeller; maximum power 1500 hp.
2) Values of total pumping head are not adjusted for head losses across the pumping plant.

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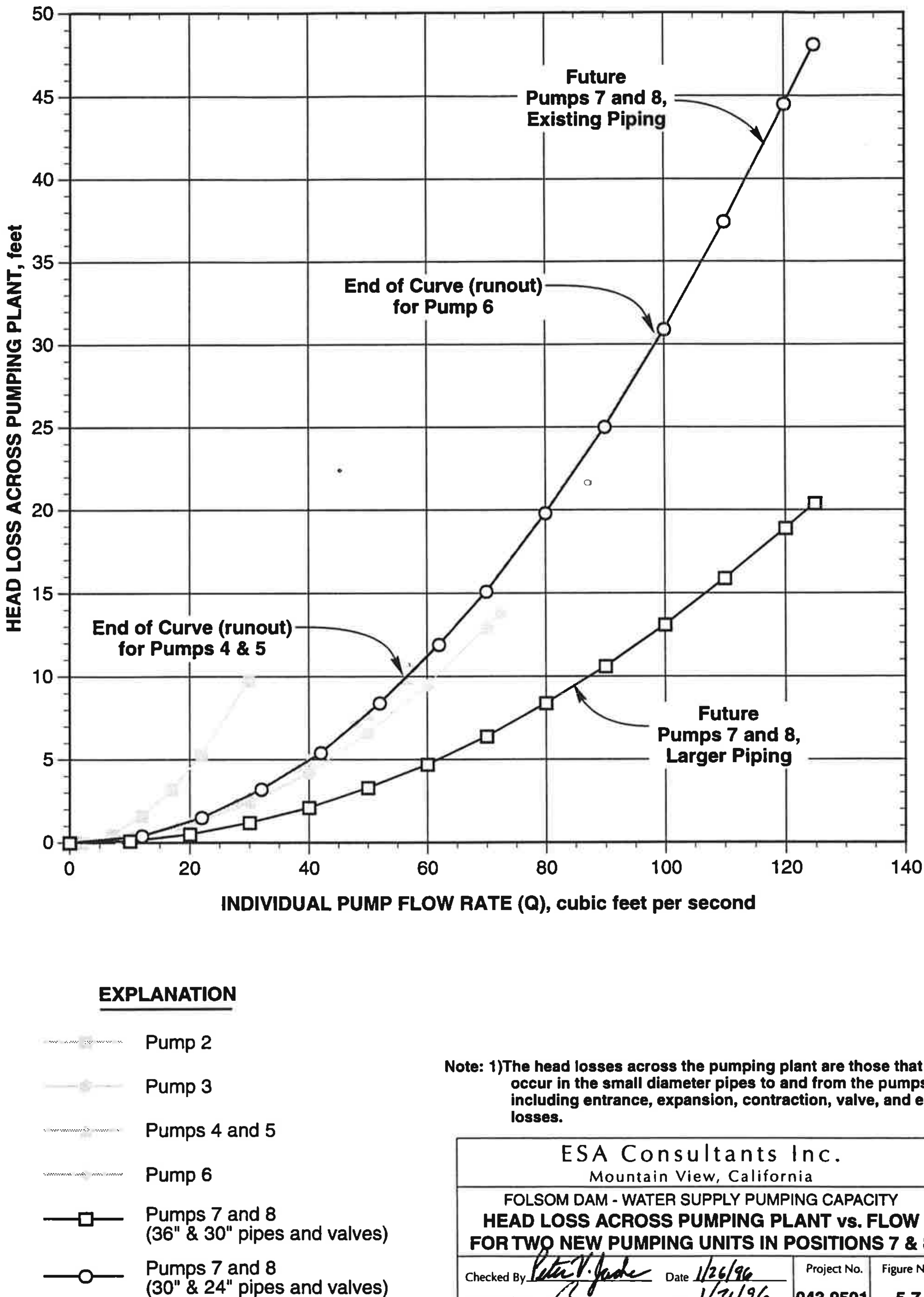
FOLSOM DAM - WATER SUPPLY PUMPING CAPACITY

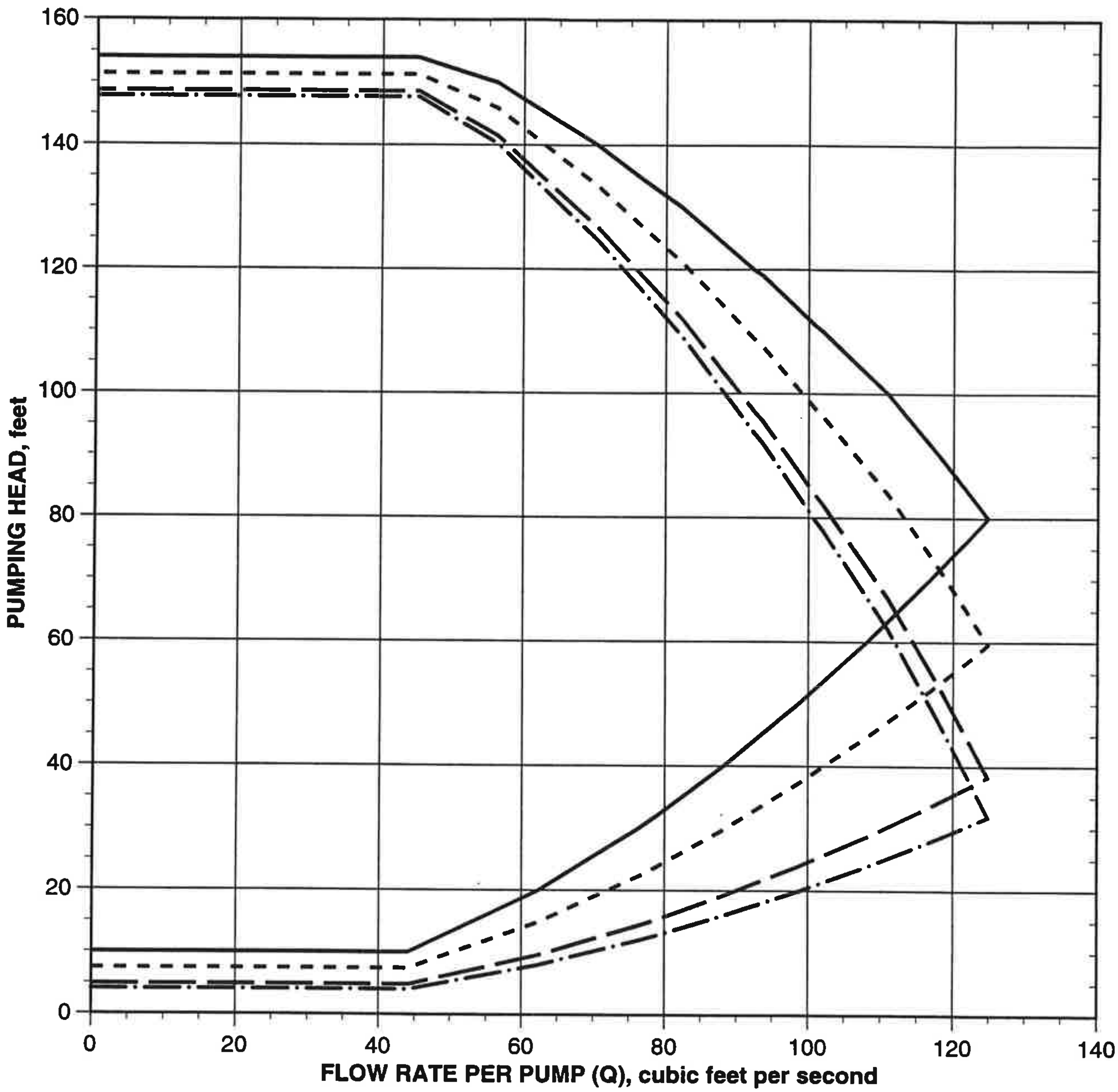
VARIABLE-SPEED PUMP OPERATING RANGE

		Project No.	Figure No.
Checked By Peter V. Jade	Date 1/26/96		
Approved By B	Date 1/31/96	043.9501	5-6



Note: 1) The head losses across the pumping plant are those that occur in the small diameter pipes to and from the pumps, including entrance, expansion, contraction, valve, and exit losses.

ESA Consultants Inc. Mountain View, California		
FOLSOM DAM - WATER SUPPLY PUMPING CAPACITY **HEAD LOSS ACROSS PUMPING PLANT vs. FLOW FOR TWO NEW PUMPING UNITS IN POSITIONS 7 & 8**		
Checked By *Peter V. Jacke* Date *1/26/96* Approved By *[signature]* Date *1/31/96*	Project No. 043.9501	Figure No. 5-7



EXPLANATION

- ——— Pump 7 (pump only with no adjustment for head losses, see note 1)
- —·—·– Pump 7 (adjusted for losses with 30" & 24" cones, pipes and valves, see note 2)
- — — Pump 7 (adjusted for losses with 30" & 24" cones and 36" & 30" valves, see note 2)
- - - - - Pump 7 (adjusted for losses with 36" & 30" cones, pipes and valves, see note 2)

Notes: 1) Based on manufacturer's curve for 750-LNE-1050 Pump (see Figure 5-1); 524 rpm; 39.9" impeller; maximum power 1415 hp.

2) Adjusted head-capacity curves are developed by subtracting head losses in the pumping plant piping from the manufacturer's pump curves.

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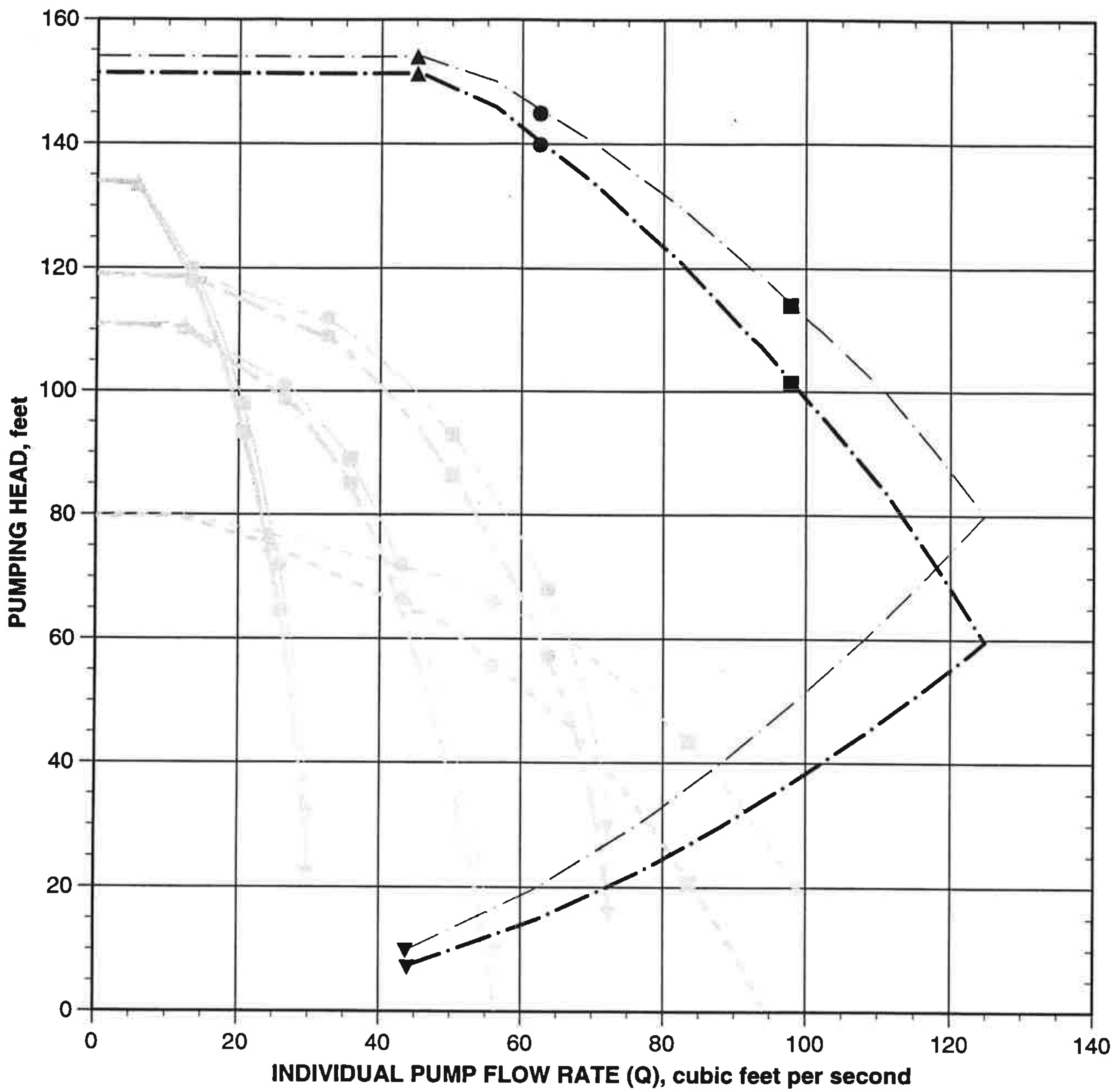
FOLSOM DAM - WATER SUPPLY PUMPING CAPACITY

PUMPS 7,8 HEAD-CAPACITY CURVES ADJUSTED FOR VARIOUS PIPING SCENARIOS

Checked By Peter V. Jocke	Date 1/26/96	Project No.	Figure No.
Approved By B	Date 1/31/96	043.9501	5-8



EXPLANATION

Pump 2 (total)
Pump 2 (adjusted*)

Pump 3 (total)
Pump 3 (adjusted*)

Pumps 4 and 5
Pumps 4 and 5 (adjusted*)

Pump 6 (total)
Pump 6 (adjusted*)

Pumps 7 and 8 (total)
Pumps 7 and 8 (adjusted*; 36" & 30" pipes and valves)

*See Note 1

■ or ■ Best Efficiency Point
● or ● 80% Efficiency
▲ or ▲ Shut Off
▼ or ▼ Runout

Notes: 1) **Adjusted pumping head for each pump curve is calculated by subtracting the head losses across the pumping plant from the manufacturer's pump curve.**
2) **The end points of all pump curves have been defined to reflect the expected normal pump operating range.**

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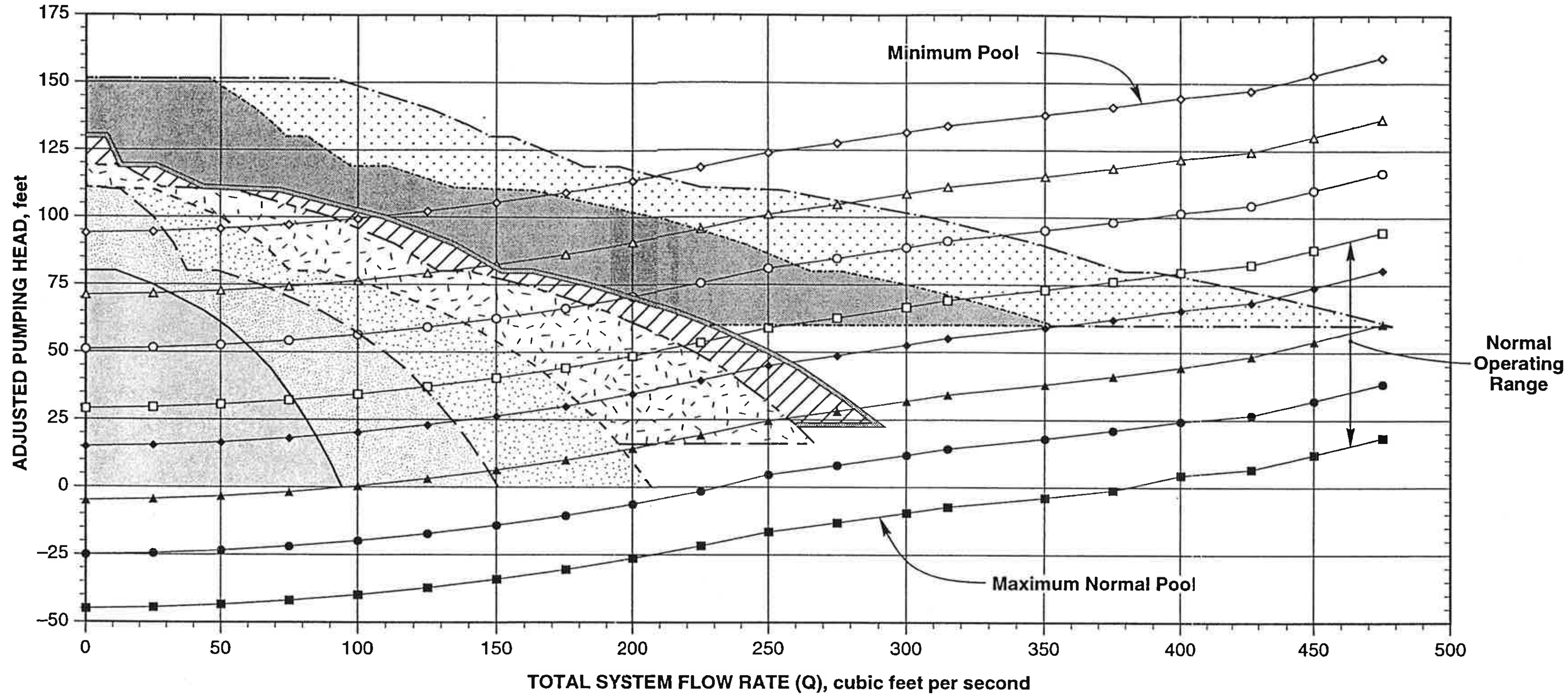
FOLSOM DAM - WATER SUPPLY PUMPING CAPACITY
INDIVIDUAL PUMP HEAD-CAPACITY CURVES ADJUSTED FOR PUMPING PLANT HEAD LOSSES

Checked By Peter V. Jacobs Date 1/26/96	Project No.	Figure No.
Approved By JB Date 1/31/96	043.9501	5-9



EXPLANATION

Cumulative Head-Capacity Curves
- Pumps 6,5,4,3,2,7 & 8
- Pumps 6,5,4,3,2 & 7
- Pump 6
- Pumps 6 & 5
- Pumps 6,5 & 4
- Pumps 6,5,4 & 3
- Pumps 6,5,4,3 & 2

Capacity Contributed By Each Pump
- Pump 8
- Pump 7
- Pump 6
- Pump 5
- Pump 4
- Pump 3
- Pump 2

System Curves
- Maximum pool - Res. El. 466
- Res. El. 446
- Res. El. 426
- Res. El. 406
- Res. El. 392
- Res. El. 370
- Res. El. 350
- Minimum pool - Res. El. 327

Notes:
1) Adjusted pumping head for each pump curve is calculated by subtracting the head losses across the pumping plant from the manufacturer's pump curve.
2) The end points of all pump curves have been defined to reflect the expected normal pump operating range.
3) The head-capacity curves for pumps 7 and 8 are based on a 750-LNE-1050 pump; 590 rpm; 35.4" impeller; maximum power 1500 hp and reflect the assumption that the suction and discharge lines for these pumps have been adjusted to 36" and 30" respectively.
4) System curves do not include head losses across the pumping plant.
5) System Curves are adjusted to reflect head losses occuring during gravity flow conditions to San Juan and/or to Natoma when appropriate.
6) Flow proportioning for the system curves is based on existing and proposed contract amounts (see Table 4-1).

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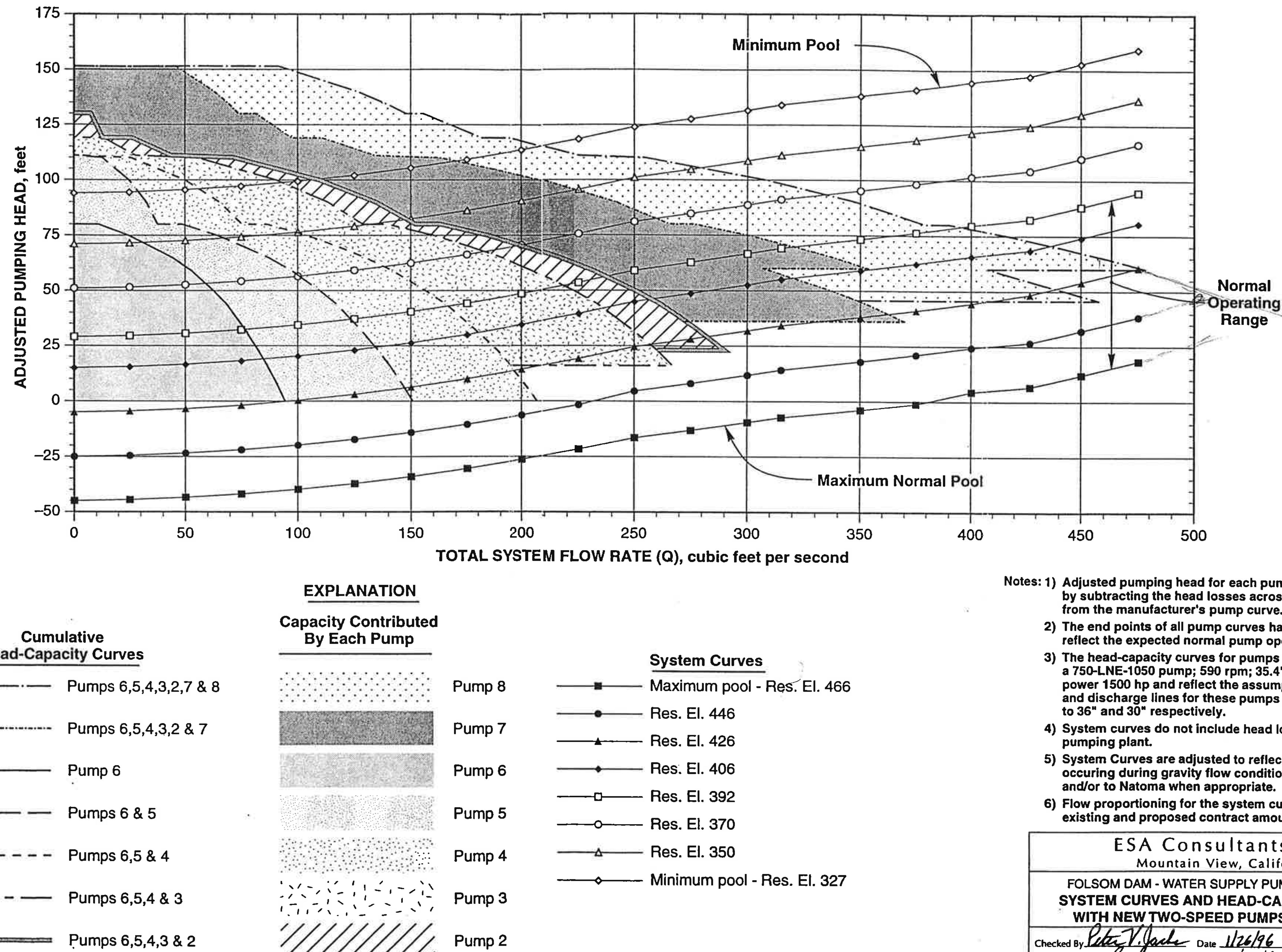
FOLSOM DAM - WATER SUPPLY PUMPING CAPACITY
SYSTEM CURVES AND HEAD-CAPACITY CURVES WITH SINGLE-SPEED PUMPS FOR 7 AND 8

Checked By Peter V. Jacke Date 1/26/96	Project No.	Figure No.
Approved By B Date 1/31/96	043.9501	5-10



Notes:
1) Adjusted pumping head for each pump curve is calculated by subtracting the head losses across the pumping plant from the manufacturer's pump curve.
2) The end points of all pump curves have been defined to reflect the expected normal pump operating range.
3) The head-capacity curves for pumps 7 and 8 are based on a 750-LNE-1050 pump; 590 rpm; 35.4" impeller; maximum power 1500 hp and reflect the assumption that the suction and discharge lines for these pumps have been adjusted to 36" and 30" respectively.
4) System curves do not include head losses across the pumping plant.
5) System Curves are adjusted to reflect head losses occuring during gravity flow conditions to San Juan and/or to Natoma when appropriate.
6) Flow proportioning for the system curves is based on existing and proposed contract amounts (see Table 4-1).

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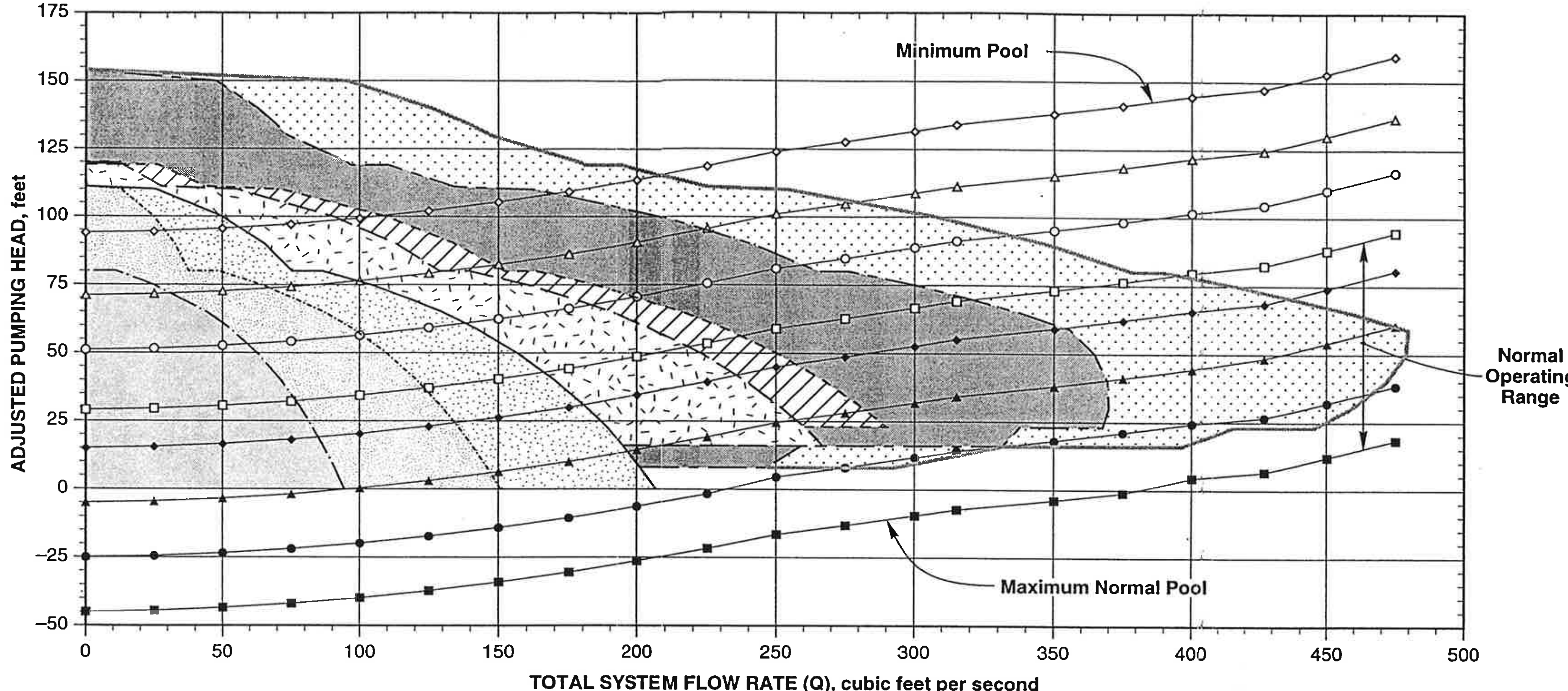
FOLSOM DAM - WATER SUPPLY PUMPING CAPACITY
SYSTEM CURVES AND HEAD-CAPACITY CURVES WITH NEW TWO-SPEED PUMPS FOR 7 AND 8

		Project No.	Figure No.
Checked By Peter V. Jacke	Date 1/26/96	043.9501	5-11
Approved By B	Date 1/31/96		



EXPLANATION

Cumulative Head-Capacity Curves

- Pumps 6,5,4,3,2,7 & 8
- Pumps 6,5,4,3,2 & 7
- Pump 6
- Pumps 6 & 5
- Pumps 6,5 & 4
- Pumps 6,5,4 & 3
- Pumps 6,5,4,3 & 2

Capacity Contributed By Each Pump

- Pump 8
- Pump 7
- Pump 6
- Pump 5
- Pump 4
- Pump 3
- Pump 2

System Curves

- Maximum pool - Res. El. 466
- Res. El. 446
- Res. El. 426
- Res. El. 406
- Res. El. 392
- Res. El. 370
- Res. El. 350
- Minimum pool - Res. El. 327

Notes:
1) Adjusted pumping head for each pump curve is calculated by subtracting the head losses across the pumping plant from the manufacturer's pump curve.
2) The end points of all pump curves have been defined to reflect the expected normal pump operating range.
3) System curves do not include head losses across the pumping plant.
4) The head-capacity curves for pumps 7 and 8 are based on a 750-LNE-1050 pump and a speed and impeller diameter combination to develop a maximum power 1500 hp and reflect the assumption that the suction and discharge lines for these pumps have been adjusted to 36" and 30" respectively.
5) System Curves are adjusted to reflect head losses occuring during gravity flow conditions to San Juan and/or to Natoma when appropriate.
6) Flow proportioning for the system curves is based on existing and proposed contract amounts (see Table 4-1).

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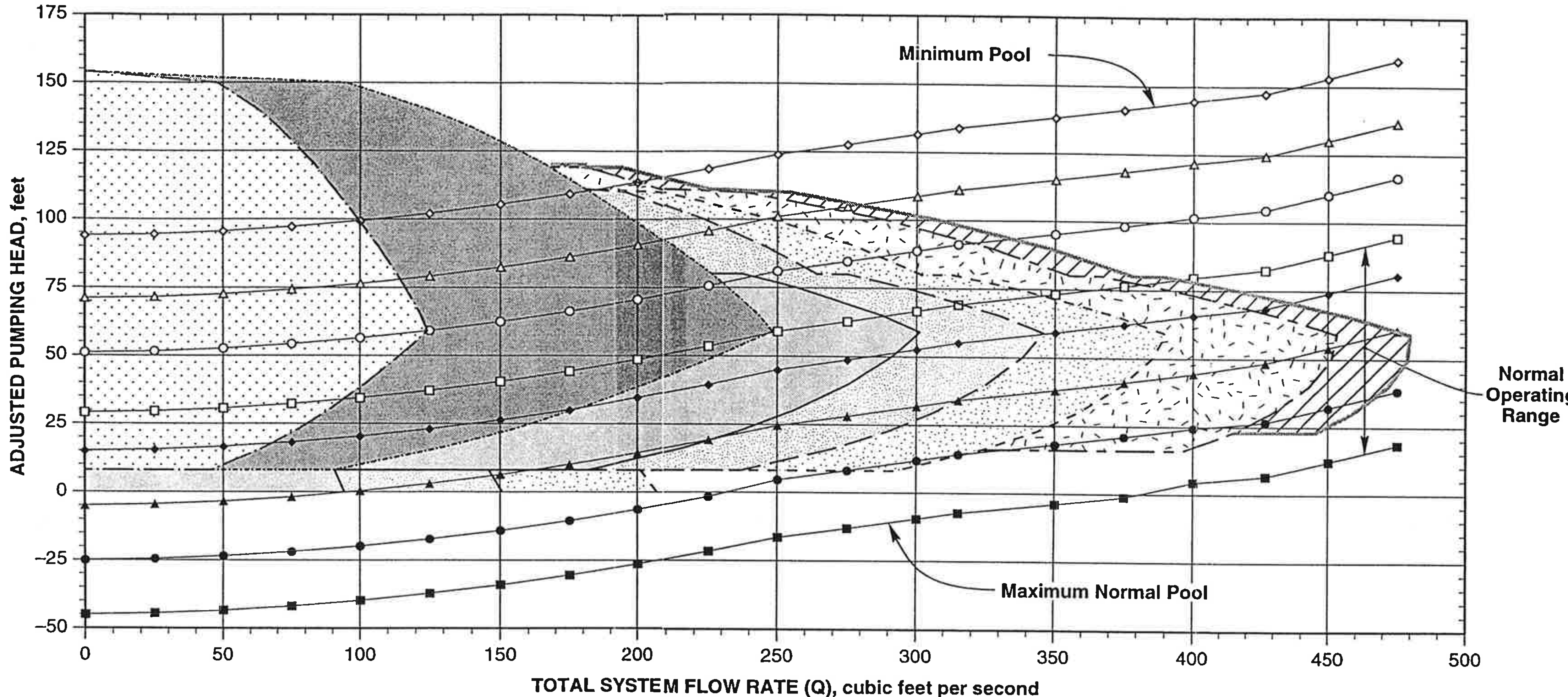
FOLSOM DAM - WATER SUPPLY PUMPING CAPACITY

HEAD-CAPACITY AND SYSTEM CURVES WITH TWO NEW VARIABLE SPEED PUMPS

Checked By Peter V. Jacke	Date 1/26/96	Project No.	Figure No.
Approved By B	Date 1/31/96	043.9501	5-12



EXPLANATION

Cumulative Head-Capacity Curves

- Pump 8
- Pumps 8 & 7
- Pumps 8,7, & 6
- Pumps 8,7,6 & 5
- Pumps 8,7,6,5 & 4
- Pumps 8,7,6,5,4 & 3
- Pumps 8,7,6,5,4,3 & 2

Capacity Contributed By Each Pump

- Pump 8
- Pump 7
- Pump 6
- Pump 5
- Pump 4
- Pump 3
- Pump 2

System Curves

- Maximum pool - Res. El. 466
- Res. El. 446
- Res. El. 426
- Res. El. 406
- Res. El. 392
- Res. El. 370
- Res. El. 350
- Minimum pool - Res. El. 327

Notes:
1) Adjusted pumping head for each pump curve is calculated by subtracting the head losses across the pumping plant from the manufacturer's pump curve.
2) The end points of all pump curves have been defined to reflect the expected normal pump operating range.
3) System curves do not include head losses across the pumping plant.
4) The head-capacity curves for pumps 7 and 8 are based on a 750-LNE-1050 pump and a speed and impeller diameter combination to develop a maximum power 1500 hp and reflect the assumption that the suction and discharge lines for these pumps have been adjusted to 36" and 30" respectively.
5) System Curves are adjusted to reflect head losses occuring during gravity flow conditions to San Juan and/or to Natoma when appropriate.
6) Flow proportioning for the system curves is based on existing and proposed contract amounts (see Table 4-1).

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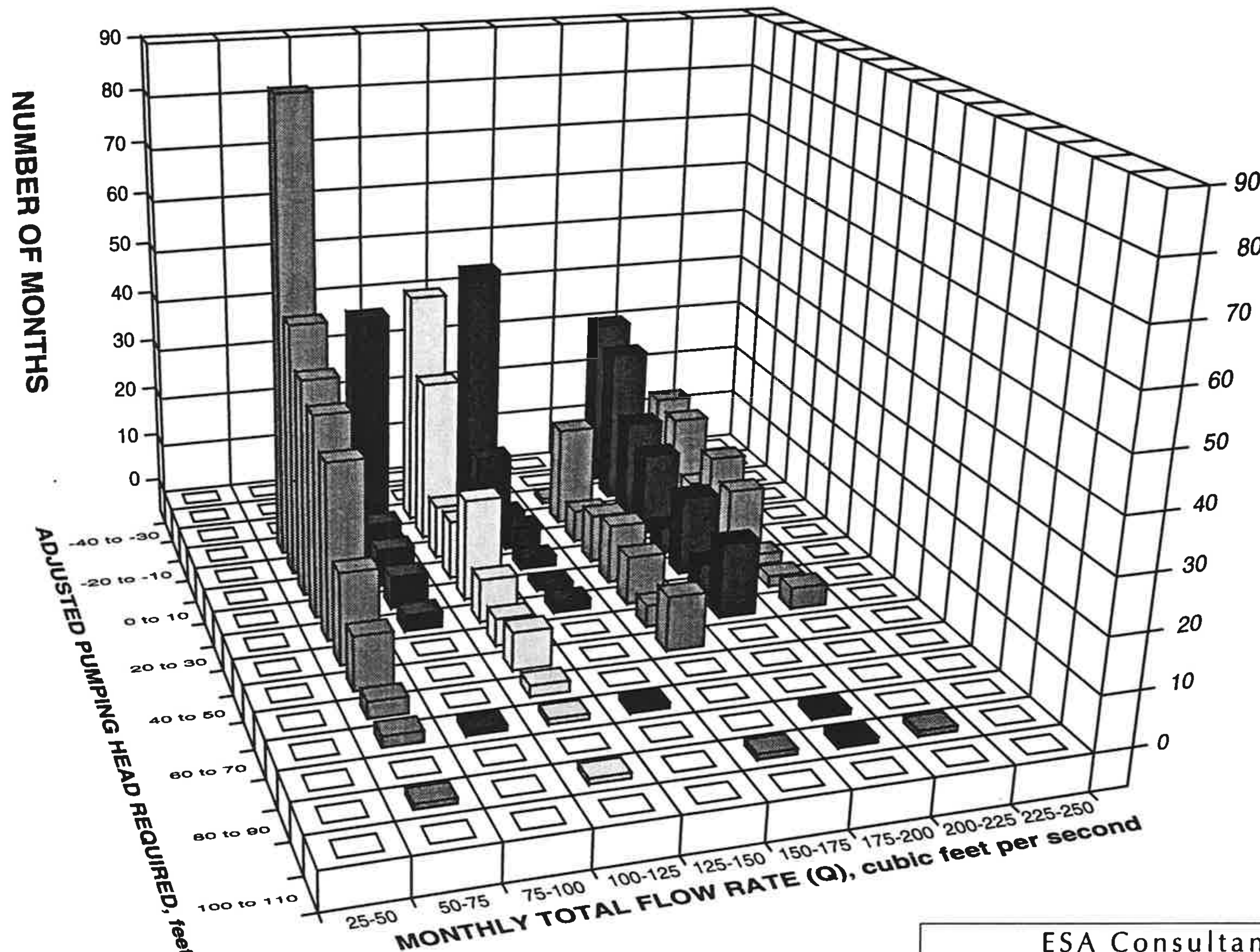
FOLSOM DAM - WATER SUPPLY PUMPING CAPACITY
HEAD-CAPACITY AND SYSTEM CURVES WITH TWO NEW VARIABLE SPEED PUMPS (ALT.)

Checked By Peter V. Jacke	Date 1/26/96	Project No.	Figure No.
Approved By B	Date 1/31/96	043.9501	5-13

ARWA-202



Notes: 1) Monthly total flow rate is calculated as a monthly average based on an estimate of each agency's total annual demand and a percentage allocation to each calendar month.

2) Adjusted pumping head required for a month is calculated by adding the system head losses (excluding head losses across the pumping plant) to that month's static head difference between the reservoir and San Juan (at El. 423).

3) Frequency distribution of monthly demand versus adjusted pumping head is based on estimated 1995 average monthly demands and 70 years of hydrologic record assuming Folsom reoperation.

4) An unfilled cell indicates a value of zero months.

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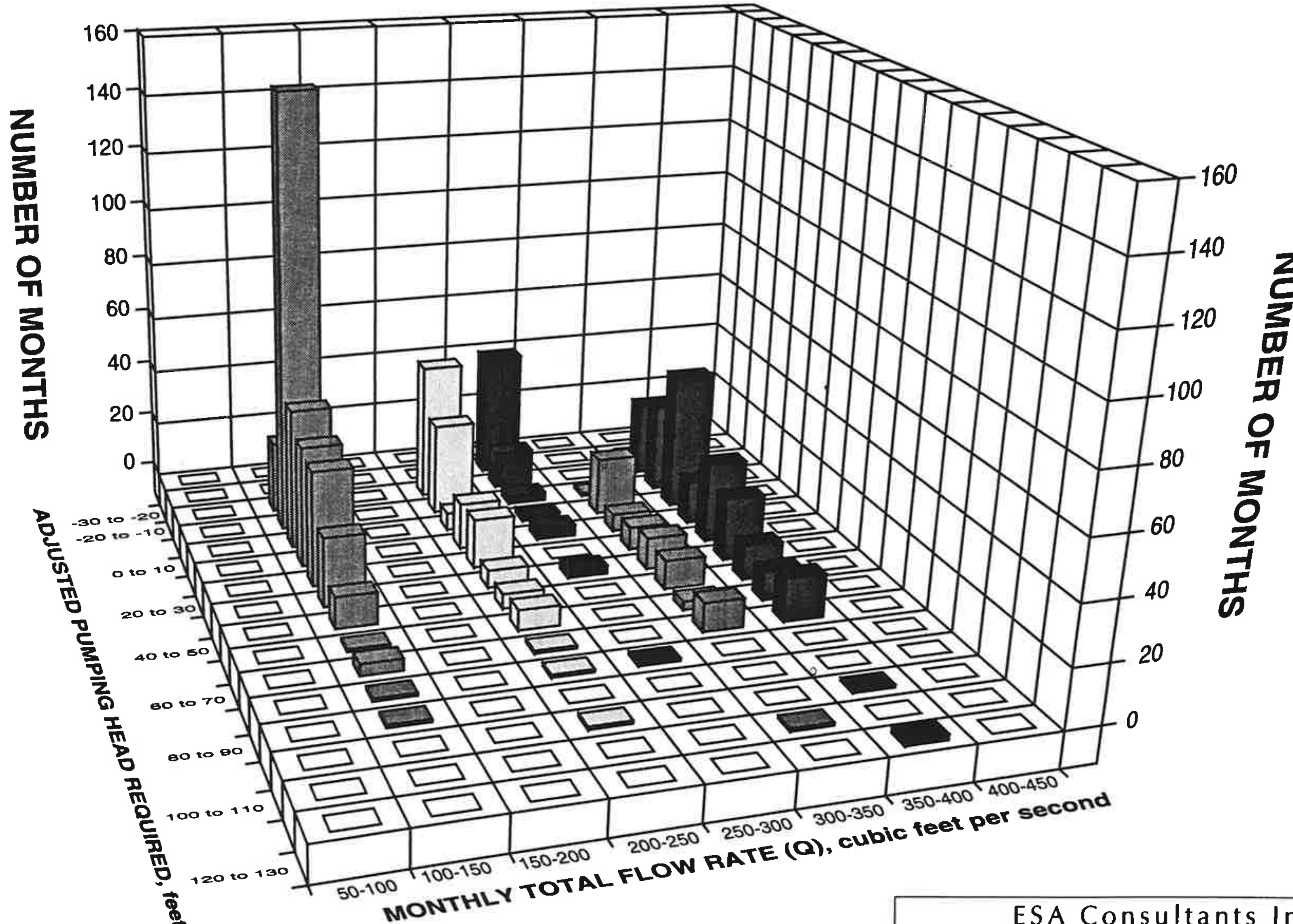
FOLSOM DAM - WATER SUPPLY PUMPING CAPACITY

FREQUENCY DISTRIBUTION OF 1995 MONTHLY DEMAND VERSUS ADJUSTED PUMPING HEAD

		Project No.	Figure No.
Checked By Peter V. Jacke	Date 1/26/96	043.9501	5-14
Approved By B	Date 1/31/96		



Notes: 1) Monthly total flow rate is calculated as a monthly average based on an estimate of each agency's total annual demand and a percentage allocation to each calendar month.

2) Adjusted pumping head required for a month is calculated by adding the system head losses (excluding head losses across the pumping plant) to that month's static head difference between the reservoir and San Juan (at El. 423).

3) Frequency distribution of monthly demand versus adjusted pumping head is based on estimated 2020 average monthly demands and 70 years of hydrologic record assuming Folsom reoperation.

4) An unfilled cell indicates a value of zero months.

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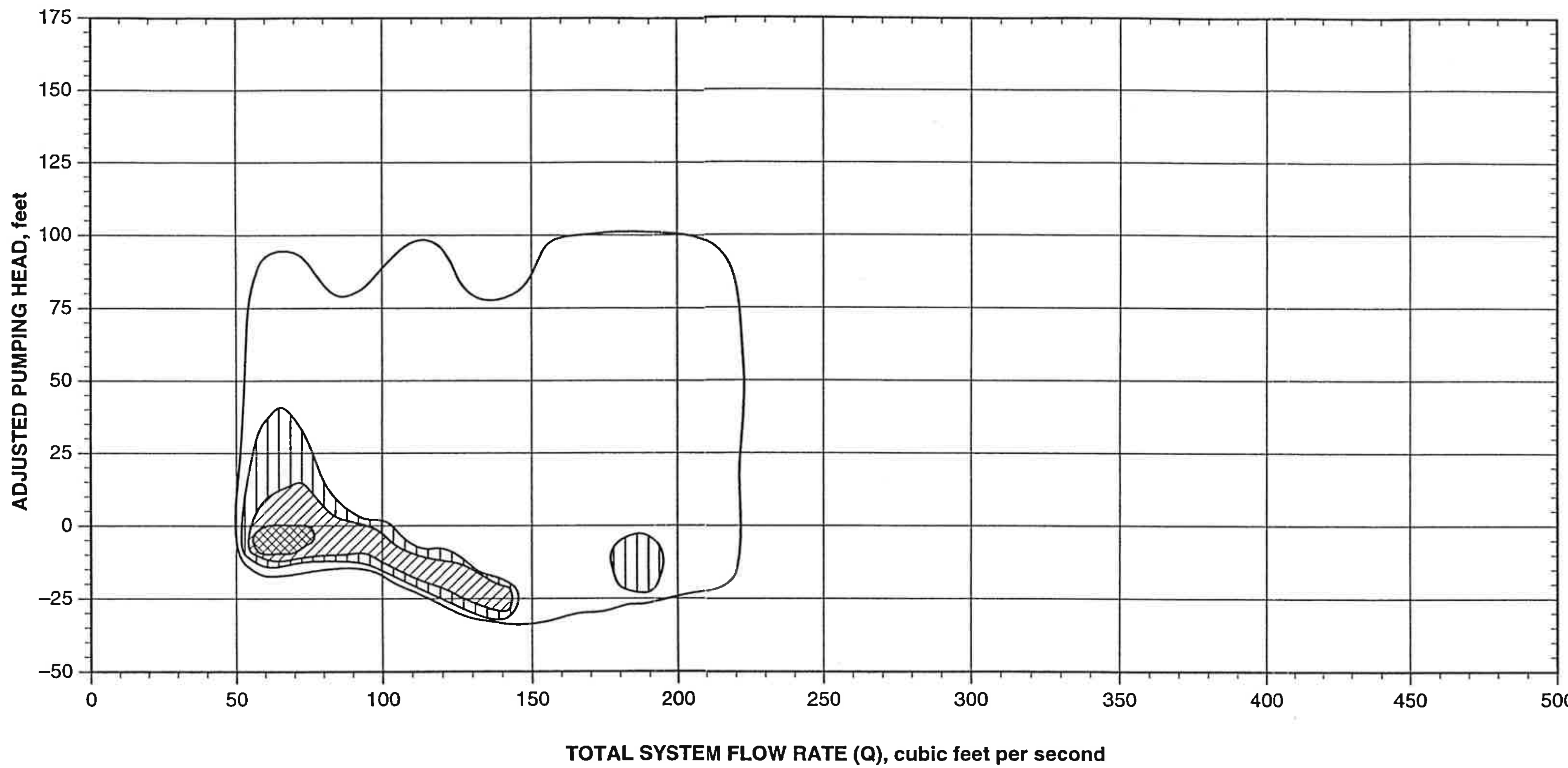
FOLSOM DAM - WATER SUPPLY PUMPING CAPACITY
FREQUENCY DISTRIBUTION OF 2020 MONTHLY DEMAND VERSUS ADJUSTED PUMPING HEAD

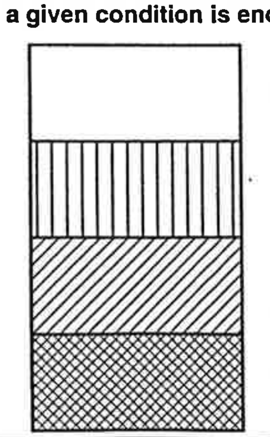
		Project No.	Figure No.
Checked By Peter V. Jacke	Date 1/26/96		
Approved By B	Date 1/31/96	043.9501	5-15



EXPLANATION

Distribution of Pumping Conditions

Percent of months during which a given condition is encountered



0 to 2.5%

2.5 to 5.0%

5.0 to 10%

10 to 11.3%

Notes: 1) Distribution of pumping conditions is based on estimated 1995 average monthly demands and 70 years of hydrologic record assuming Folsom reoperation.

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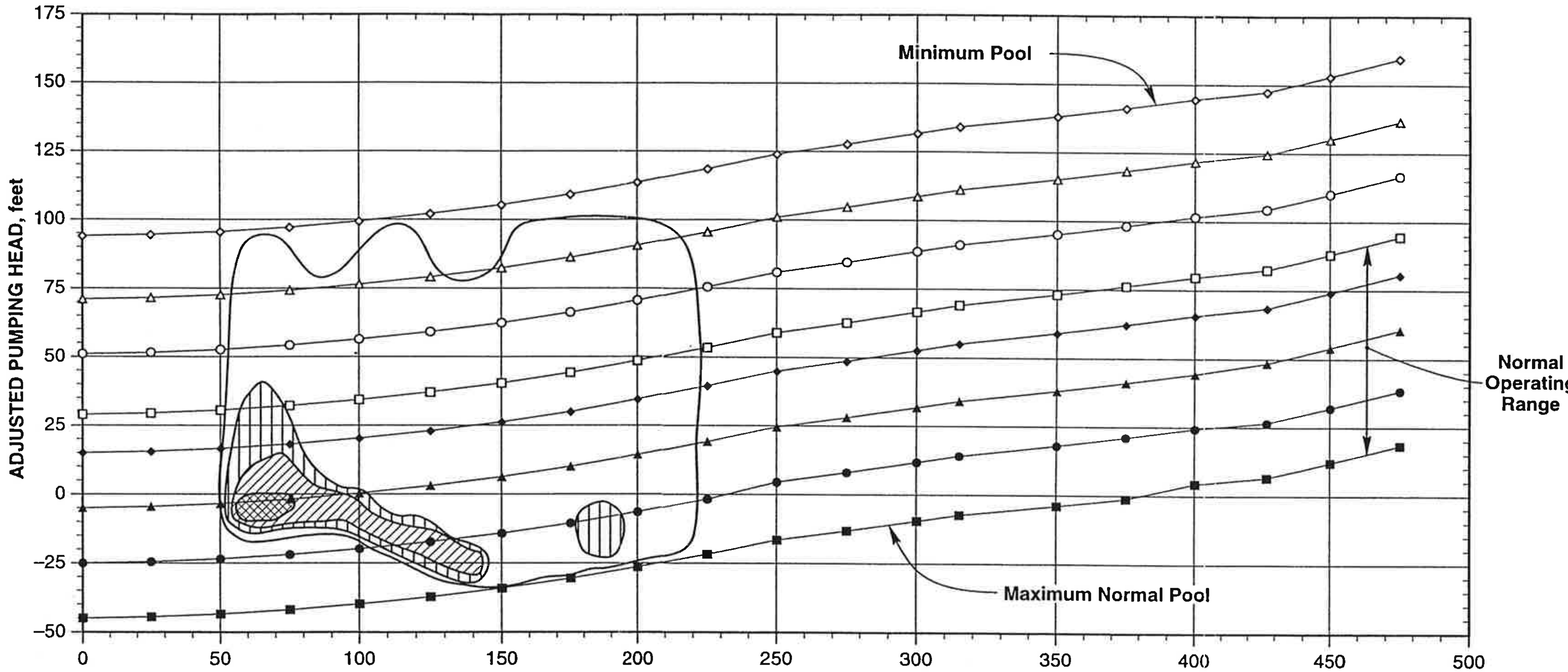
FOLSOM DAM - WATER SUPPLY PUMPING CAPACITY

DISTRIBUTION OF 1995 PUMPING CONDITIONS

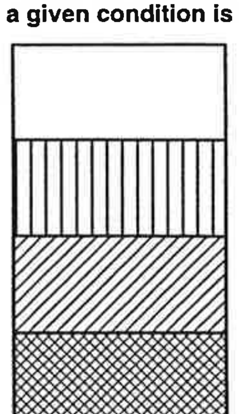
Checked By / Approved By	Date	Project No.	Figure No.
Checked By	Date 1/26/96	**043.9501**	**5-16**
Approved By	Date 1/31/96		



EXPLANATION

Distribution of Pumping Conditions

Percent of months during which a given condition is encountered



System Curves

- Maximum pool - Res. El. 466
- Res. El. 446
- Res. El. 426
- Res. El. 406
- Res. El. 392
- Res. El. 370
- Res. El. 350
- Minimum pool - Res. El. 327

Notes:
1) System curves do not include head losses across the pumping plant.
2) Distribution of pumping conditions is based on estimated 1995 average monthly demands and 70 years of hydrologic record assuming Folsom reoperation.
3) System curves are adjusted to reflect head losses occurring during gravity flow conditions to San Juan and/or to Natoma when appropriate.
4) Flow proportioning for the system curves is based on existing and proposed contract amounts (see Table 4-1).

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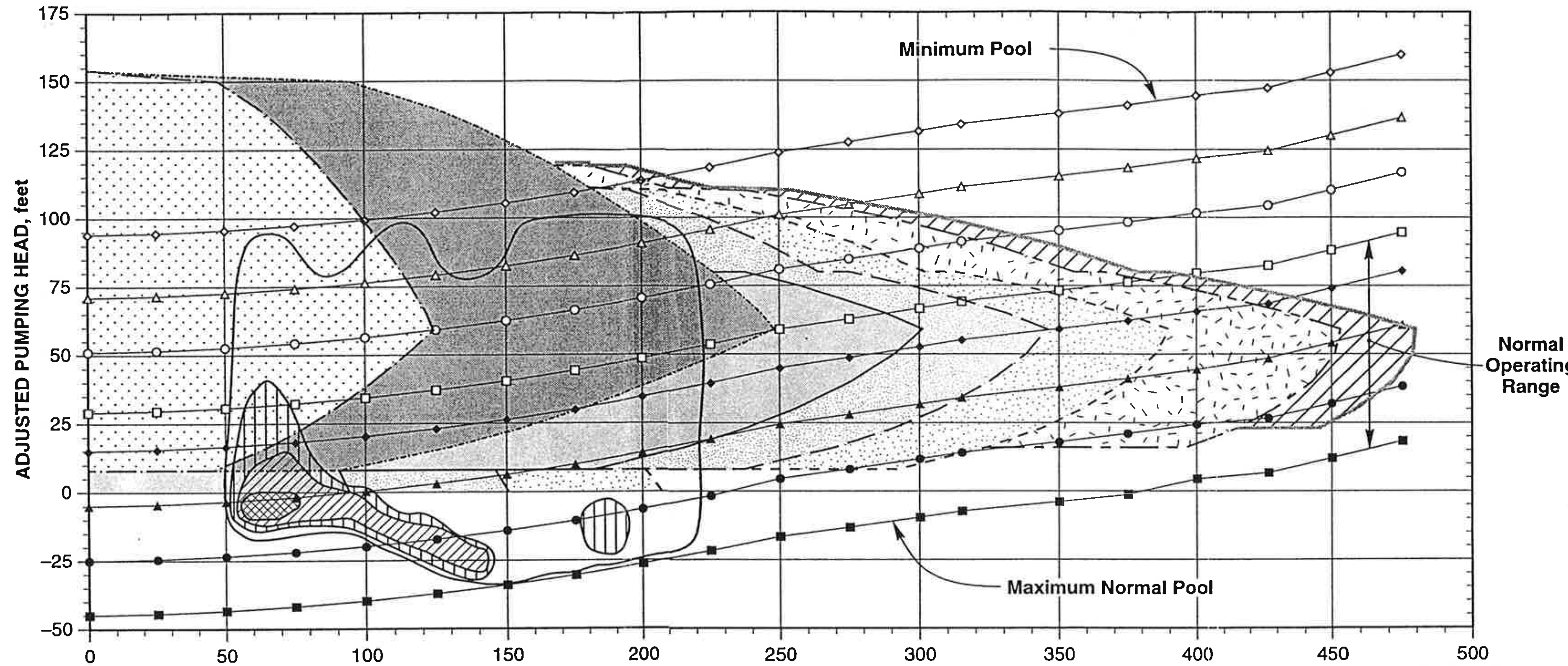
Mountain View, California

FOLSOM DAM - WATER SUPPLY PUMPING CAPACITY

DISTRIBUTION OF 1995 PUMPING CONDITIONS AND SYSTEM CURVES

Checked By Peter V. Jacke Date 1/26/96	Project No.	Figure No.
Approved By [signature] Date 1/31/96	043.9501	5-17

ARWA-202



Notes:
1) Adjusted pumping head for each pump curve is calculated by subtracting the head losses across the pumping plant from the manufacturer's pump curve.
2) The end points of all pump curves have been defined to reflect the expected normal pump operating range.
3) The head-capacity curves for pumps 7 and 8 are based on a 750-LNE-1050 pump and a speed and impeller diameter combination to develop a maximum power 1500 hp and reflect the assumption that the suction and discharge lines for these pumps have been modified to 36" and 30" respectively.
4) System curves do not include head losses across the pumping plant.
5) Distribution of pumping conditions is based on estimated 1995 average monthly demands and 70 years of hydrologic record assuming Folsom reoperation.
6) System curves are adjusted to reflect head losses occurring during gravity flow conditions to San Juan and/or to Natoma when appropriate.
7) Flow proportioning for the system curves is based on existing and proposed contract amounts (see Table 4-1).

EXPLANATION

Cumulative Head-Capacity Curves

- Pump 8
- Pumps 8 & 7
- Pumps 8,7, & 6
- Pumps 8,7,6 & 5
- Pumps 8,7,6,5 & 4
- Pumps 8,7,6,5,4 & 3
- Pumps 8,7,6,5,4,3 & 2

Capacity Contributed By Each Pump

- Pump 8
- Pump 7
- Pump 6
- Pump 5
- Pump 4
- Pump 3
- Pump 2

System Curves

- Maximum pool - Res. El. 466
- Res. El. 446
- Res. El. 426
- Res. El. 406
- Res. El. 392
- Res. El. 370
- Res. El. 350
- Minimum pool - Res. El. 327

Distribution of Pumping Conditions

Percent of months during which a given condition is encountered

- 0 to 2.5%
- 2.5 to 5.0%
- 5.0 to 10%
- 10 to 11.3%

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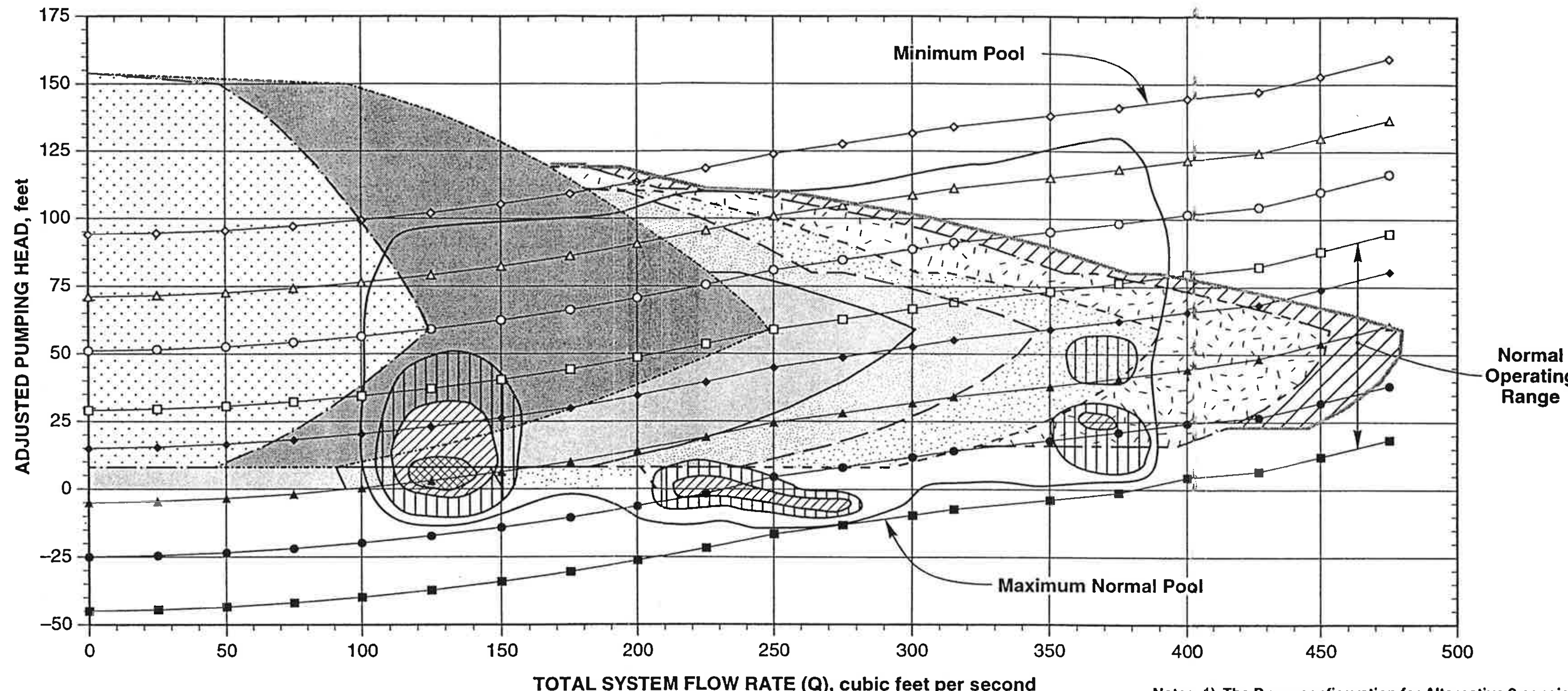
FOLSOM DAM - WATER SUPPLY PUMPING CAPACITY
SYSTEM CURVES, DISTRIBUTION OF 1995
PUMPING CONDITIONS, AND HEAD-CAPACITY CURVES
WITH TWO NEW VARIABLE SPEED PUMPS

Checked By Peter V. Jache Date 1/26/96	Project No.	Figure No.
Approved By B Date 1/31/96	043.9501	5-18



EXPLANATION

Cumulative Head-Capacity Curves

- Pump 8
- Pumps 8 & 7
- Pumps 8,7, & 6
- Pumps 8,7,6 & 5
- Pumps 8,7,6,5 & 4
- Pumps 8,7,6,5,4 & 3
- Pumps 8,7,6,5,4,3 & 2

Capacity Contributed By Each Pump

- Pump 8
- Pump 7
- Pump 6
- Pump 5
- Pump 4
- Pump 3
- Pump 2

System Curves

- Maximum pool - Res. El. 466
- Res. El. 446
- Res. El. 426
- Res. El. 406
- Res. El. 392
- Res. El. 370
- Res. El. 350
- Minimum pool - Res. El. 327

Distribution of Pumping Conditions

Percent of months during which a given condition is encountered

- 0 to 2.5%
- 2.5 to 5.0%
- 5.0 to 10%
- 10 to 18.1%

Notes: 1) The Pump configuration for Alternative 2 consists of a new pump 7 with a two-speed (590 and 505 rpm) motor
1) Adjusted pumping head for each pump curve is calculated by subtracting the head losses across the pumping plant from the manufacturer's pump curve.
2) The end points of all pump curves have been defined to reflect the expected normal pump operating range.
3) The head-capacity curves for pumps 7 and 8 are based on a 750-LNE-1050 pump and a speed and impeller diameter combination to develop a maximum power 1500 hp and reflect the assumption that the suction and discharge lines for these pumps have been adjusted to 36" and 30" respectively.
4) System curves do not include head losses across the pumping plant.
5) Distribution of pumping conditions is based on estimated 2020 average monthly demands and 70 years of hydrologic record assuming Folsom reoperation.
6) System curves are adjusted to reflect head losses occurring during gravity flow conditions to San Juan and/or to Natoma when appropriate.
7) Flow proportioning for the system curves is based on existing and proposed contract amounts (see Table 4-1).

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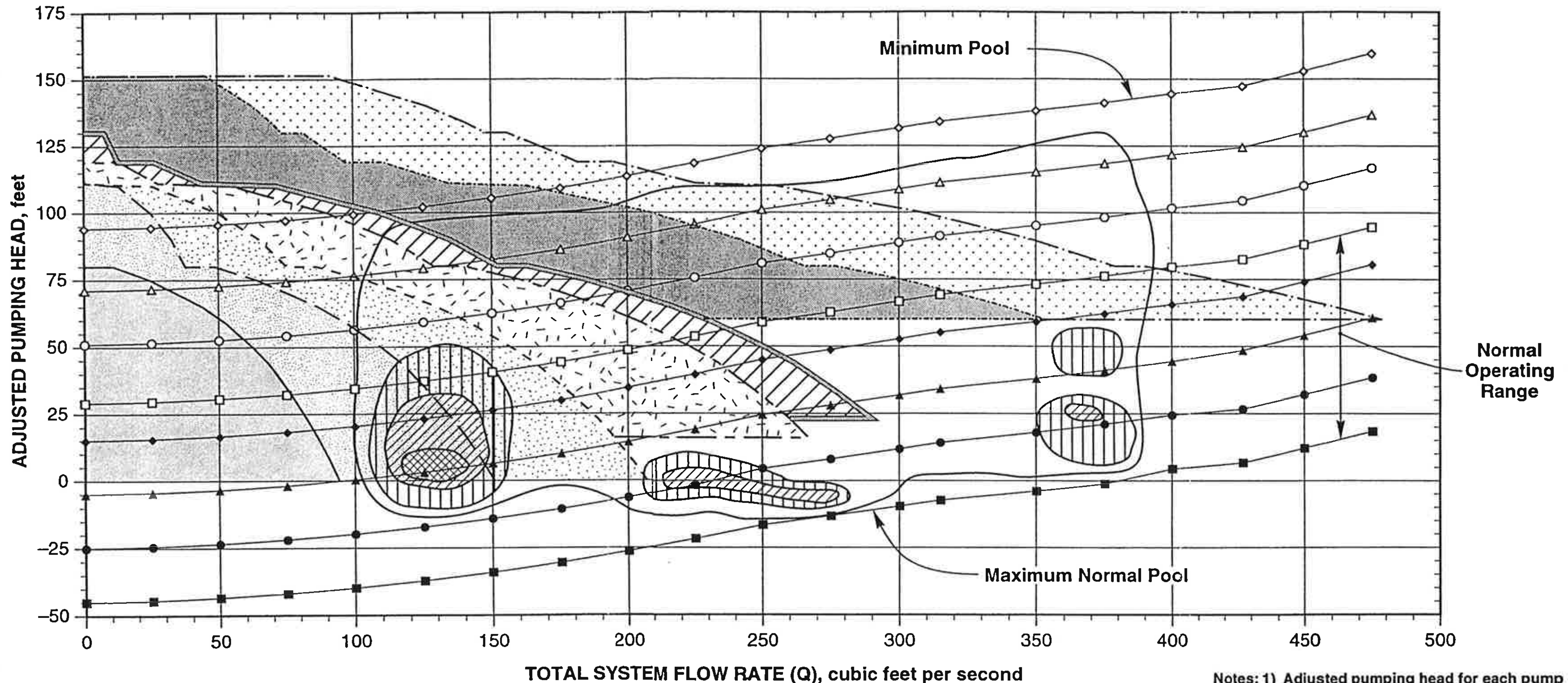
FOLSOM DAM - WATER SUPPLY PUMPING CAPACITY
SYSTEM CURVES, DISTRIBUTION OF 2020 PUMPING CONDITIONS, AND HEAD-CAPACITY CURVES WITH TWO NEW VARIABLE SPEED PUMPS

		Project No.	Figure No.
Checked By Peter V. Yache	Date 1/26/96	043.9501	5-19
Approved By B	Date 1/31/96		

ARWA-202



EXPLANATION

Cumulative Head-Capacity Curves

- Pump 8
- Pumps 8 & 7
- Pumps 8,7, & 6
- Pumps 8,7,6 & 5
- Pumps 8,7,6,5 & 4
- Pumps 8,7,6,5,4 & 3
- Pumps 8,7,6,5,4,3 & 2

Capacity Contributed By Each Pump

- Pump 8
- Pump 7
- Pump 6
- Pump 5
- Pump 4
- Pump 3
- Pump 2

System Curves

- Maximum pool - Res. El. 466
- Res. El. 446
- Res. El. 426
- Res. El. 406
- Res. El. 392
- Res. El. 370
- Res. El. 350
- Minimum pool - Res. El. 327

Distribution of Pumping Conditions

Percent of months during which a given condition is encountered

- 0 to 2.5%
- 2.5 to 5.0%
- 5.0 to 10%
- 10 to 18.1%

Notes:
1) Adjusted pumping head for each pump curve is calculated by subtracting the head losses across the pumping plant from the manufacturer's pump curve.
2) The end points of all pump curves have been defined to reflect the expected normal pump operating range.
3) The head-capacity curves for pumps 7 and 8 are based on a 750-LNE-1050 pump; and a speed and impeller diameter combination to develop a maximum power 1500 hp and reflect the assumption that the suction and discharge lines for these pumps have been modified to 36" and 30" respectively.
4) System curves do not include head losses across the pumping plant.
5) Distribution of pumping conditions is based on estimated 2020 average monthly demands and 70 years of hydrologic record assuming Folsom reoperation.
6) System curves are adjusted to reflect head losses occurring during gravity flow conditions to San Juan and/or to Natoma when appropriate.
7) Flow proportioning for the system curves is based on existing and proposed contract amounts (see Table 4-1).

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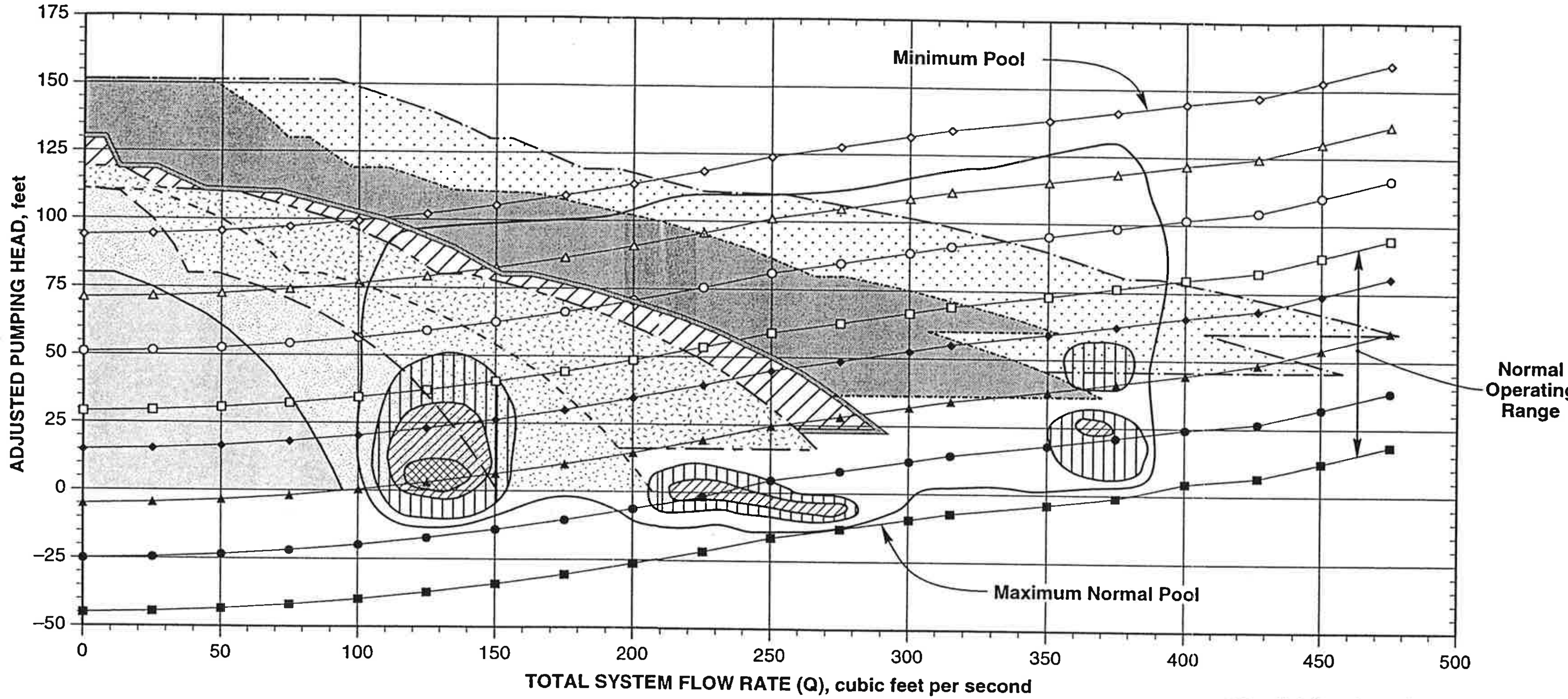
FOLSOM DAM - WATER SUPPLY PUMPING CAPACITY
SYSTEM CURVES, DISTRIBUTION OF 2020
PUMPING CONDITIONS AND HEAD-CAPACITY CURVES
WITH SINGLE-SPEED PUMPS FOR 7 AND 8

Checked By Peter V. Jacks	Date 1/26/96	Project No.	Figure No.
Approved By B	Date 1/31/96	043.9501	5-20



EXPLANATION

Cumulative Head-Capacity Curves

- Pump 8
- Pumps 8 & 7
- Pumps 8,7, & 6
- Pumps 8,7,6 & 5
- Pumps 8,7,6,5 & 4
- Pumps 8,7,6,5,4 & 3
- Pumps 8,7,6,5,4,3 & 2

Capacity Contributed By Each Pump

- Pump 8
- Pump 7
- Pump 6
- Pump 5
- Pump 4
- Pump 3
- Pump 2

System Curves

- Maximum pool - Res. El. 466
- Res. El. 446
- Res. El. 426
- Res. El. 406
- Res. El. 392
- Res. El. 370
- Res. El. 350
- Minimum pool - Res. El. 327

Distribution of Pumping Conditions

Percent of months during which a given condition is encountered

- 0 to 2.5%
- 2.5 to 5.0%
- 5.0 to 10%
- 10 to 18.1%

Notes:
1) Adjusted pumping head for each pump curve is calculated by subtracting the head losses across the pumping plant from the manufacturer's pump curve.
2) The end points of all pump curves have been defined to reflect the expected normal pump operating range.
3) The head-capacity curves for pumps 7 and 8 are based on a 750-LNE-1050 pump; and a speed and impeller diameter combination to develop a maximum power 1500 hp and reflect the assumption that the suction and discharge lines for these pumps have been modified to 36" and 30" respectively.
4) System curves do not include head losses across the pumping plant.
5) Distribution of pumping conditions is based on estimated 2020 average monthly demands and 70 years of hydrologic record assuming Folsom reoperation.
6) System curves are adjusted to reflect head losses occurring during gravity flow conditions to San Juan and/or to Natoma when appropriate.
7) Flow proportioning for the system curves is based on existing and proposed contract amounts (see Table 4-1).

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FOLSOM DAM - WATER SUPPLY PUMPING CAPACITY
SYSTEM CURVES, DISTRIBUTION OF 2020 PUMPING CONDITIONS AND HEAD-CAPACITY CURVES WITH NEW TWO-SPEED PUMPS FOR 7 AND 8

Checked By Peter V. Jacke Date 1/26/96	Project No.	Figure No.
Approved By B Date 1/31/96	043.9501	5-21

6. SIXTY INCH INTAKE GATE VALVE LOCATED IN DAM

The 84" diameter intake line from the reservoir to the pumping plant includes a 60" diameter gate valve (see Figures 6-1, 6-2 and 6-3 from Corps Drawing AM-1-9-414 / 2). With an increase of maximum flow to the pumping plant from 315 cfs to 400 cfs the question arises whether this valve becomes a significant restriction for flow. The Corps (1951) calculations show the following flow conditions and head loss calculations for the originally visualized 315 cfs:

- 84" Velocity: 8.19 ft/sec
- 84" Velocity Head: 1.04 ft
- 60" Velocity: 16.04 ft/sec
- 60" Velocity Head: 4.00 ft
- Head Loss for Contraction/Valve/Expansion:
  0.1(4.00-1.04)+0.19 x 4.00 + 0.2 (4.00-1.04) = 1.65 ft

Figure 6-4 presents the hydraulic and energy grade lines for the intake portion of the piping under this flow, assuming that the reservoir water surface is at minimum pool (El. 327).

Increasing the intake flow to 400 cfs would create the following flow conditions, using the same calculation procedures:

- 84" Velocity: 10.39 ft/sec
- 84" Velocity Head: 1.68 ft
- 60" Velocity: 20.40 ft/sec
- 60" Velocity Head: 6.45 ft
- Head Loss for Contraction/Valve/Expansion:
  0.1(6.45-1.68)+0.19 x 6.45 + 0.2 (6.45-1.68) = 2.66 ft

Figure 6-5 presents the hydraulic and energy grade lines for the intake portion of the piping with 400 cfs of flow, again assuming that the reservoir water surface is at minimum pool (El. 327).

An additional 1.01 feet of head loss due to the flow increase to 400 cfs does not represent a significant restriction of flow from a head loss viewpoint.

A second question is whether the valve is suitable for such a flow. The valve was originally specified as follows (quoted from Corps specifications 1532r1, pp. 16-9 to 16-10):

> 16-12 VALVES IN MAIN PIPE LINES: The following valves, suitable for the service required and complete with required appurtenances, shall be furnished and installed where shown on the drawings and/or specified herein.
>
> a. 60-Inch Valve: One valve shall be installed in the pumping plant inlet emergency valve chamber, and 1 valve shall be installed in each of valve pits Nos. 2 and 3 at the pumping plant, all as shown. These valves shall be standard, iron body, bronze mounted, flanged, electrically operated gate valves, faced and drilled with double discs and parallel seats, O.S. and Y, and square bottom construction - suitable for 120 lb. non-shock cold water pressure - for installation in a vertical position in a

horizontal pipe line - equipped with a 12 inch O.S. and Y hand-operated bypass valve. The opening or closing speed shall be approximately 1 foot per minute and the limits of travel shall be governed by a mechanical torque responsive switch such as Limitorque Type SM or approved equal, having ample capacity for the service, complete with all appurtenances, including valve position indicator and three "open, stop, close" pushbutton controls. The available electric service in the valve chamber is 440 volts, 3-phase, 60 cycle. The available electric service near the pumping plant is 208 volts, 3-phase, 60 cycle. The maximum static head with the valve closed is approximately 160 feet and 100 feet on the supply and delivery sides respectively, assuming the opposite side of the valve drained in each instance. These valves shall be designed for throttling operation throughout their full travel, and under a maximum hydrostatic pressure differential of 30 p.s.i.

The valve was purchased from the A.P. Smith Mfg. Co. (order No. 90524; April 4, 1952) as Item No. 11. Three 60" valves were ordered as follows (direct quote from the order):

> 60" D.F. 150# test Vertical Electrically operated
> Rising Stem square bottom case and disc Valve
> with 12" O.S. & Y. by-pass 35" face to face
> Standard Drilling. Electrical Equipment to
> consist of SM-4 60 ft. # Limitorque Motor Unit
> mounted on Valve. size 1 NEMA I Controller, AS
> 3B/2L NEMA I push button station.__Valves for
> 440 volt 3 phase 60 cycle__Valves for 208 Volt
> 3 phase 60 cycle. 26# unbalanced pressure. 75#
> static pressure. B.M.#G2S3-A4 Open Left.

Thus, the valve specification language was apparently relaxed in two ways prior to purchase-- the 120 psi static pressure requirement was reduced to 75 psi (combined with a 150 psi test) and the 30 psi pressure differential for throttling was reduced 26 psi.

There is no apparent velocity or flow rating for the valve from the above language. Based on the 26 psi of unbalanced pressure (60 feet of head) for throttling throughout the full range of valve travel, one can infer high flow rates for a wide open valve-- i.e., velocities exceeding 60 feet/second. Based on this inference, a 400 cfs flow rate (V=20.40 ft/sec) would not be considered excessive.

It is noted that for this valve and the flow range being considered, transients created during valve closure are not significant. This is because of the slow closure speed (one foot per minute) and primarily because of the valve location, in close proximity to the reservoir.

The next question is whether the higher velocity in the 60" segment will lead to flow separation within the downstream flow expansion. The expansion length is only 5 feet. A useful rule of thumb is that the expansion section should have a length equivalent to 3 times the Froude Number (for the smaller diameter section) for each unit increase in radius. The following calculations apply:

• Froude Number (60" diameter @ 400 cfs)

$$= \frac{V}{\sqrt{gd}}$$

$$= \frac{20.40}{\sqrt{32.2 \times 5}}$$

$$= 1.61$$

• Safe expansion length:

$$= 3 \text{ Fr} \times 0.5 \ (7-5)$$

$$= 4.82$$

Thus, with a 5 foot expansion length, no flow separation is expected at 400 cfs.

A final question regarding the existing 60" gate valve is its capability for emergency closure if there were a rupture downstream. Such a rupture is conceivable in the context of a strong earthquake. If such a rupture occurred between the dam and the pumping plant, it could not be repaired until the flow was shut off and the 60" valve would be the only means of shut-off. This is because the intake stoplog was designed to be placed only under balanced head (zero flow) conditions. Water supply deliveries would be interrupted until the flow was shut-off and downstream repairs were made.

The primary emergency closure evidence provided by the specifications and purchase language set forth above is the indications that the valve is capable of throttling (and presumably closure) under up to 26 psi of unbalanced pressure (or 60 feet of head) through its full range of disc travel. If the reservoir were full at the time of rupture and emergency closure, the actual unbalanced hydrostatic pressure would be approximately 160 feet of head or 70 psi. This appeared to raise uncertainty as to whether emergency closure could be achieved. The manufacturer was contacted and indicated that, when new, the valve should have been capable of emergency closure under a 75 psi differential, based on its rated working pressure. The manufacturer did express concern about the valve's age and lack of knowledge about operation and maintenance activities.

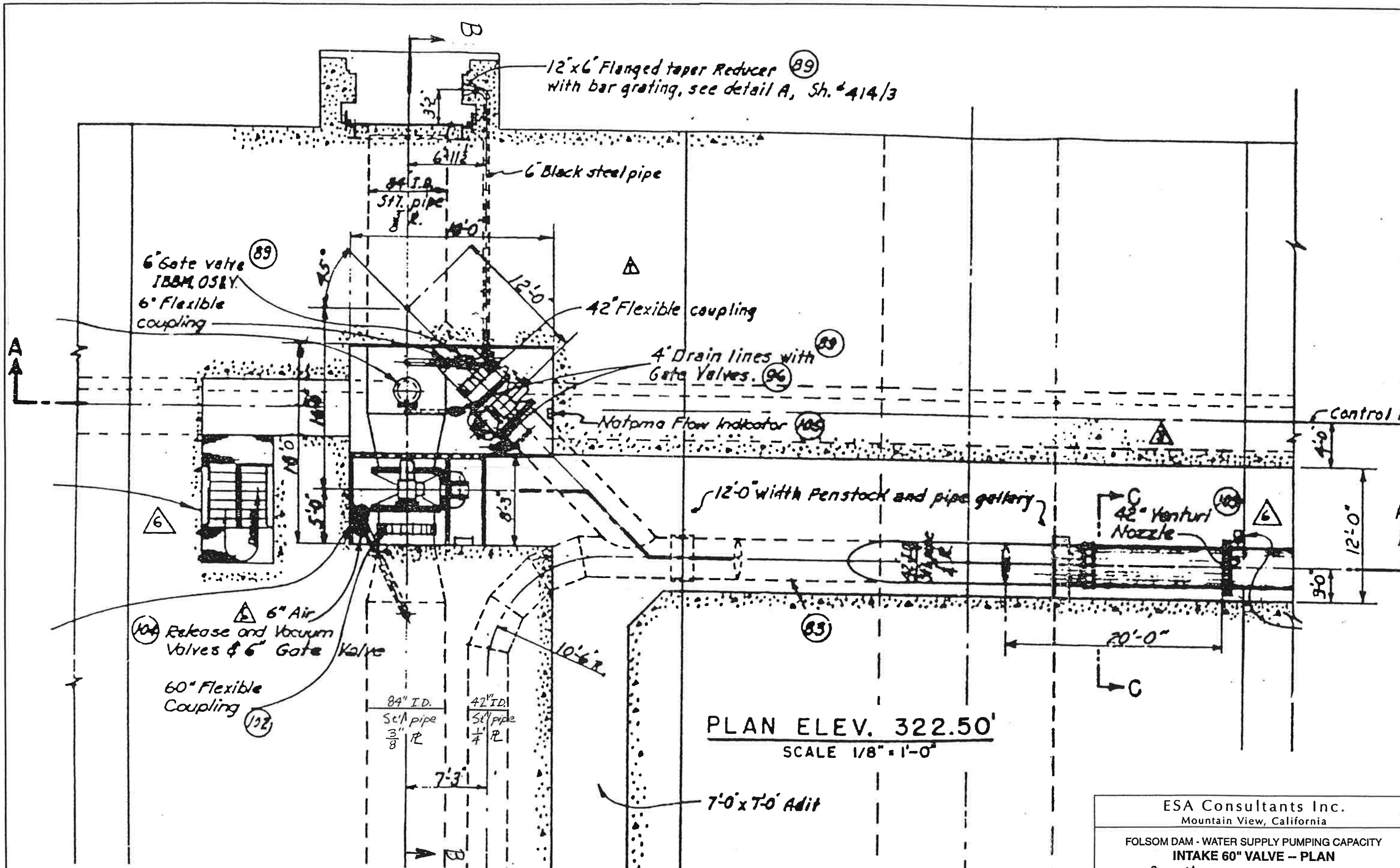
Even if the internal parts of the valve proved capable of closure under the above described conditions, there is a further uncertainty as to whether the actuator has been designed to deliver the required torque. Torque calculations likely were based on the 30 or 26 psi of unbalanced pressure. Furthermore the electrical actuator has been specifically designed to limit torque delivery to 60 ft-lbs. This is to limit damage in case of disc blockage. The relevant calculations have not yet been located for review during this study.

The emergency closure issue is not really impacted by the primary focus of the present study-- the increase of Folsom Dam water supply system capacity from 315 cfs to 400 cfs. The valve either is or is not capable of emergency closure at 160 ft (70 psi) of differential pressure and the answer will be the same whether the system's nominal capacity is 315 cfs or 400 cfs. Given the need for reservoir drawdown and the interruption of water supply deliveries that could occur with a downstream rupture and failure of efforts to close the valve, it seems that a full review of emergency closure capability is advisable. Such a detailed review is beyond the scope of the present study.

Should a review of emergency closure capability indicate that valve replacement is necessary, consideration should be given to a larger diameter replacement valve to provide lower velocities, less head loss and the possibility of future capacity increases.

It should be noted that the same emergency closure question applies to the 42" valve that is located in the dam to provide gravity feed to the Natoma Line. In this case, however, the valve is rated for throttling at least 40 psi (92.3 feet) of differential pressure according to the A.P. Smith Manufacturing Co. order document. It is also a 75 psi working pressure valve so, presumably, it was capable of emergency closure when new. This valve was used extensively for throttling of Natoma gravity flows until the mid 1970's. According to Joye (1995) of the USBR, a pin failed within the valve opening/closing mechanism, making the valve inoperable until repaired. Mr. Joye attributes the failure primarily to the throttling service.

More detailed information has been requested on the operation and maintenance history of both these valves.



Note: Figure abstracted from Corps Drawing AM-1-9-414/2.

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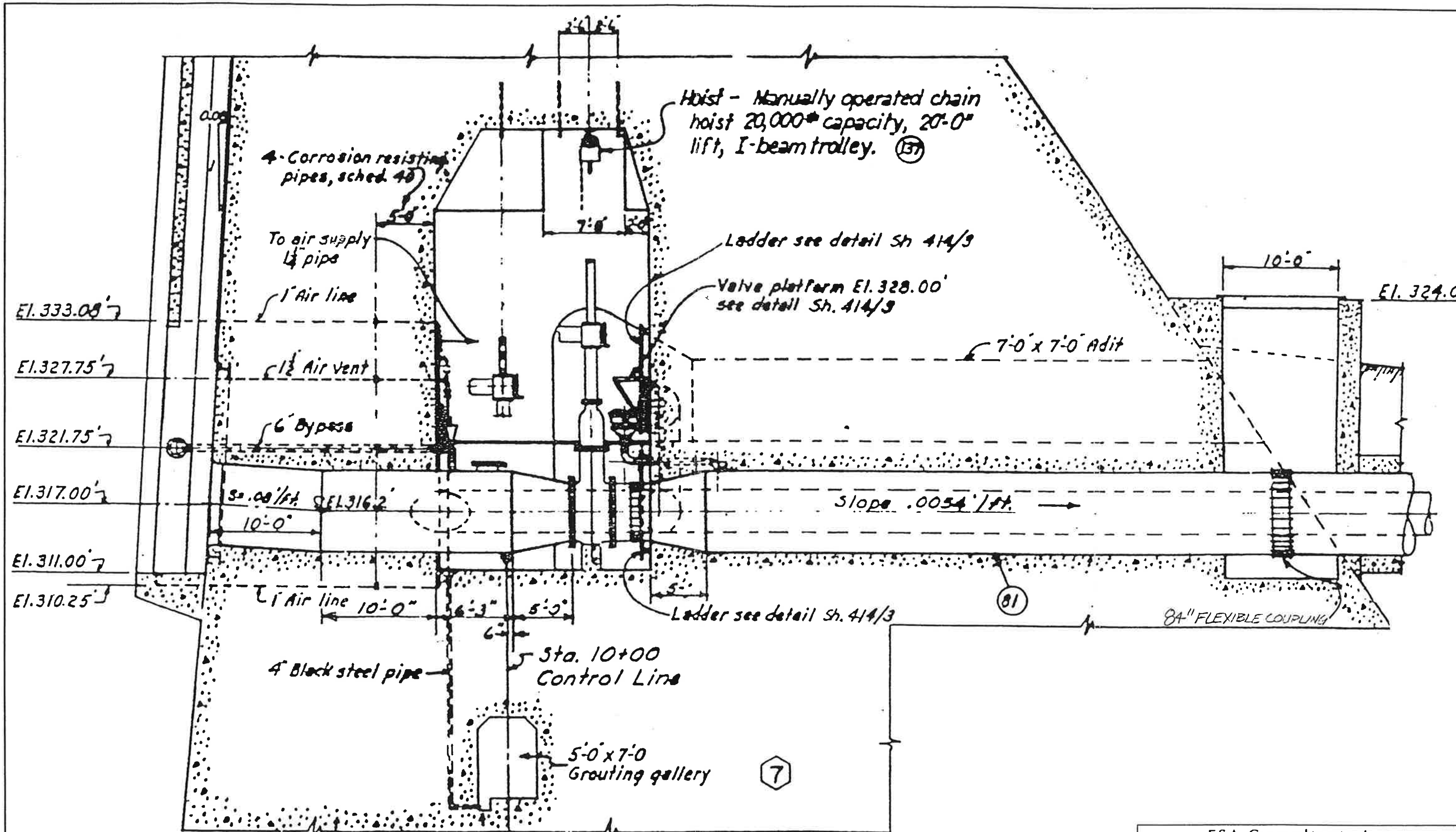
FOLSOM DAM - WATER SUPPLY PUMPING CAPACITY
INTAKE 60" VALVE – PLAN

Checked By	Date 1/26/96	Project No. 043.9501	Figure No. 6-1
Approved By	Date 1/31/96		

ARWA-202



SECTION B-B

SCALE 1/8" = 1'-0"

Note: Figure abstracted from Corps Drawing AM-1-9-414/2.

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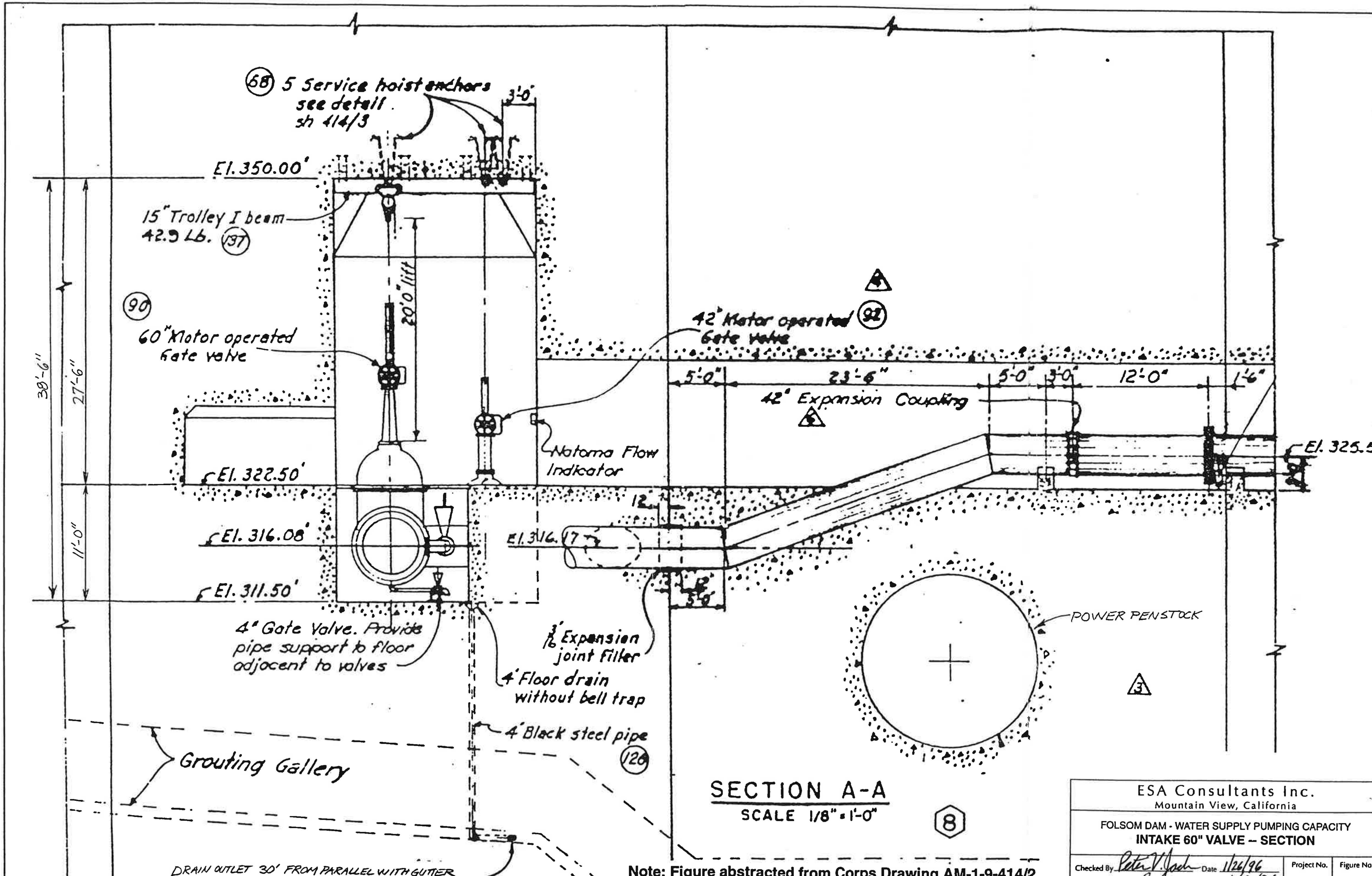
FOLSOM DAM - WATER SUPPLY PUMPING CAPACITY
INTAKE 60" VALVE – PROFILE

Checked By	Date 1/26/96	Project No.	Figure No.
Approved By	Date 1/31/96	043.9501	6-2



Note: Figure abstracted from Corps Drawing AM-1-9-414/2.

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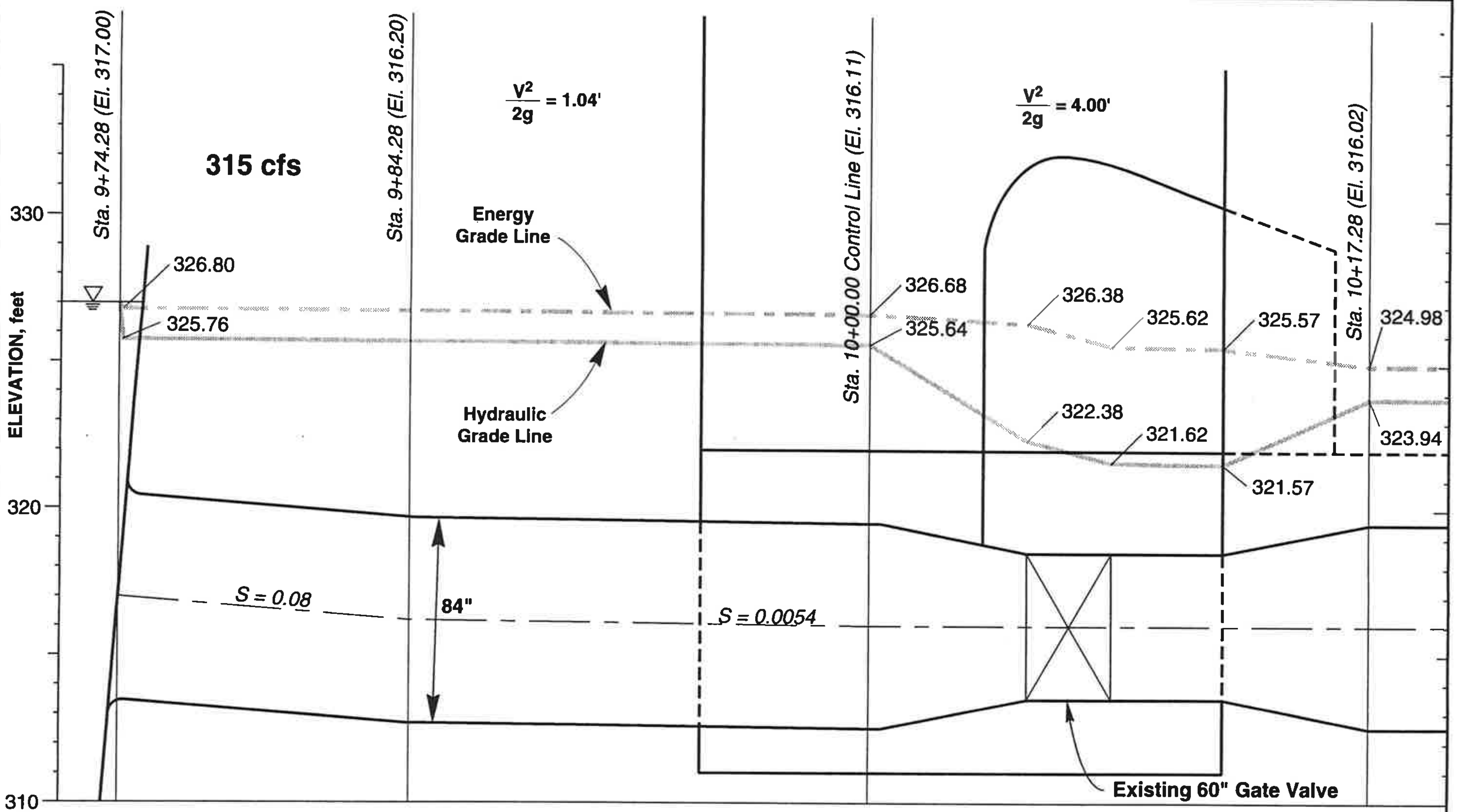
FOLSOM DAM - WATER SUPPLY PUMPING CAPACITY
INTAKE 60" VALVE – SECTION

Checked By Peter V. Jach	Date 1/26/96	Project No. 043.9501	Figure No. 6-3
Approved By B	Date 1/31/96		



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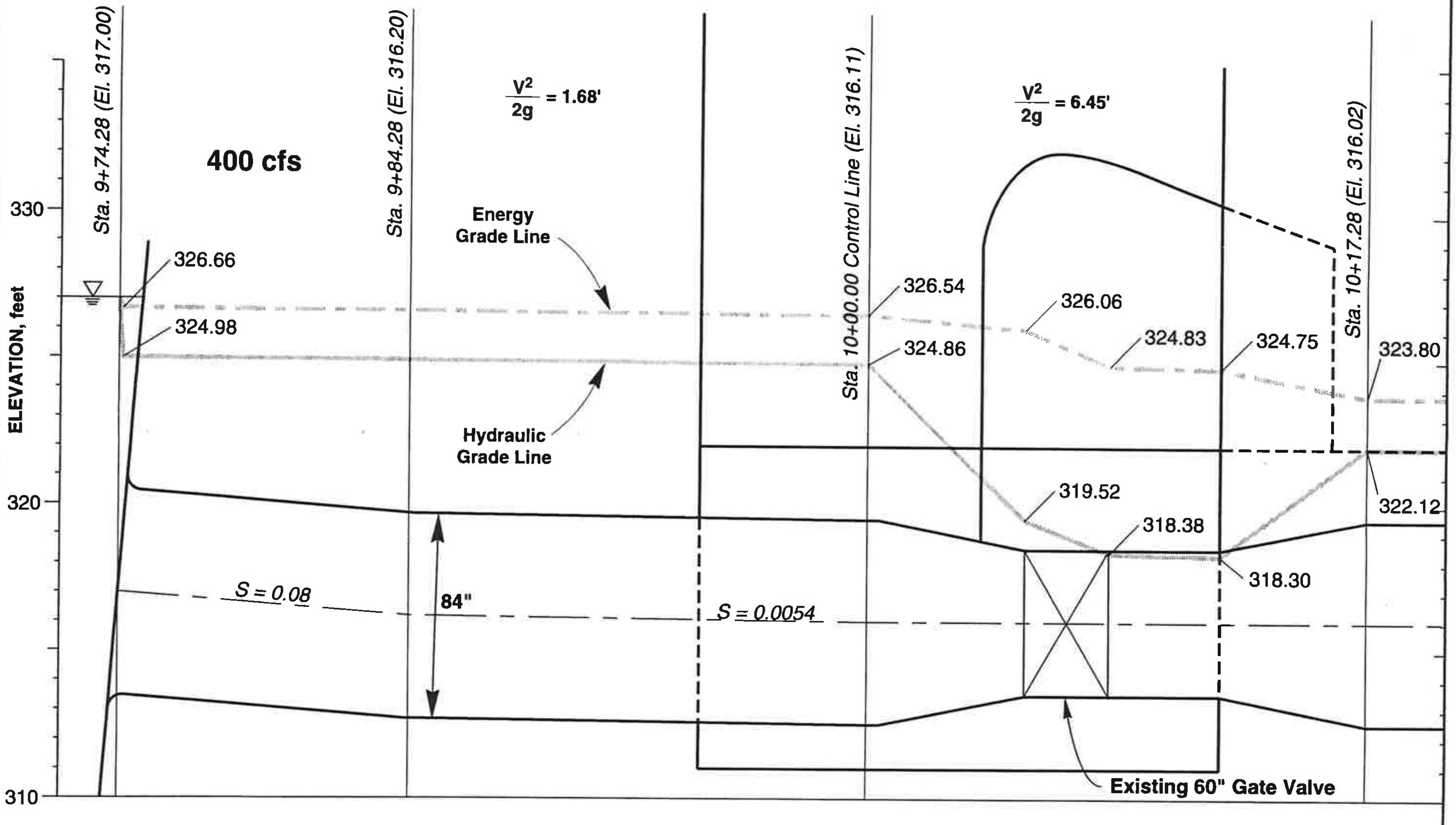
FOLSOM DAM - WATER SUPPLY PUMPING CAPACITY
INTAKE ENERGY AND HYDRAULIC GRADE LINES FOR 315 CFS

Checked By Peter V. Jache	Date 1/26/96	Project No.	Figure No.
Approved By B	Date 1/31/96	043.9501	6-4



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FOLSOM DAM - WATER SUPPLY PUMPING CAPACITY
INTAKE ENERGY AND HYDRAULIC GRADE LINES FOR 400 CFS

Checked By Peter V. Jacobs	Date 1/26/96	Project No.	Figure No.
Approved By B	Date 1/31/96	043.9501	6-5

7. VENTURI METER IN THE NORTH FORK LINE

The original equipment for flow measurements in the North Fork Line is an 84" x 49" Venturi meter located a short distance downstream of the pumping plant at El. 327.05 (Corps, 1951; see Figure 7-1). With an increase of maximum flow in the North Fork Line from 250 cfs to 335 cfs (recognizing the diversion of 65 cfs to the Natoma line), the question arises whether this flow meter becomes a significant restriction for flow. The Corps (1951) calculations show the following flow conditions for the originally visualized 250 cfs:

- 84" Velocity: 6.5 ft/sec
- 84" Velocity Head: 0.66 ft
- 49" Velocity: 19.09 ft/sec
- 49" Velocity Head: 5.66 ft
- Differential Velocity Head: 5.00 ft
- Head Loss: 0.2 (Differential) = 0.2 x 5.0 = 1.0 ft

With a flow of 335 cfs, the following flow characteristics will pertain:

- 84" Velocity: 8.70 ft/sec
- 84" Velocity Head: 1.18 ft
- 49" Velocity: 25.58 ft/sec
- 49" Velocity Head: 10.16 ft
- Differential Velocity Head: 8.98 ft
- Head Loss: 0.2 (Differential) = 0.2 x 8.98 = 1.80 ft

Thus, with an increase of only 0.8 feet in head loss, the venturi meter is not a significant restriction from a head loss viewpoint.

The next question is whether the higher velocity in the venturi throat will lead to flow separation within the downstream flow expansion. The expansion length is approximately 14 feet. A useful rule of thumb is that the expansion section should have a length equivalent to 3 times the Froude Number (for the smaller diameter section) for each unit increase in radius. The following calculations apply:

- Froude Number (49" diameter @ 335 cfs):

$$= \frac{V}{\sqrt{gd}}$$

$$= \frac{25.58}{\sqrt{32.2 \times \frac{49}{12}}}$$

$$= 2.23$$

- Safe Expansion Length:

$$= 3Fr \times 0.5\left(7 - \frac{49}{12}\right)$$

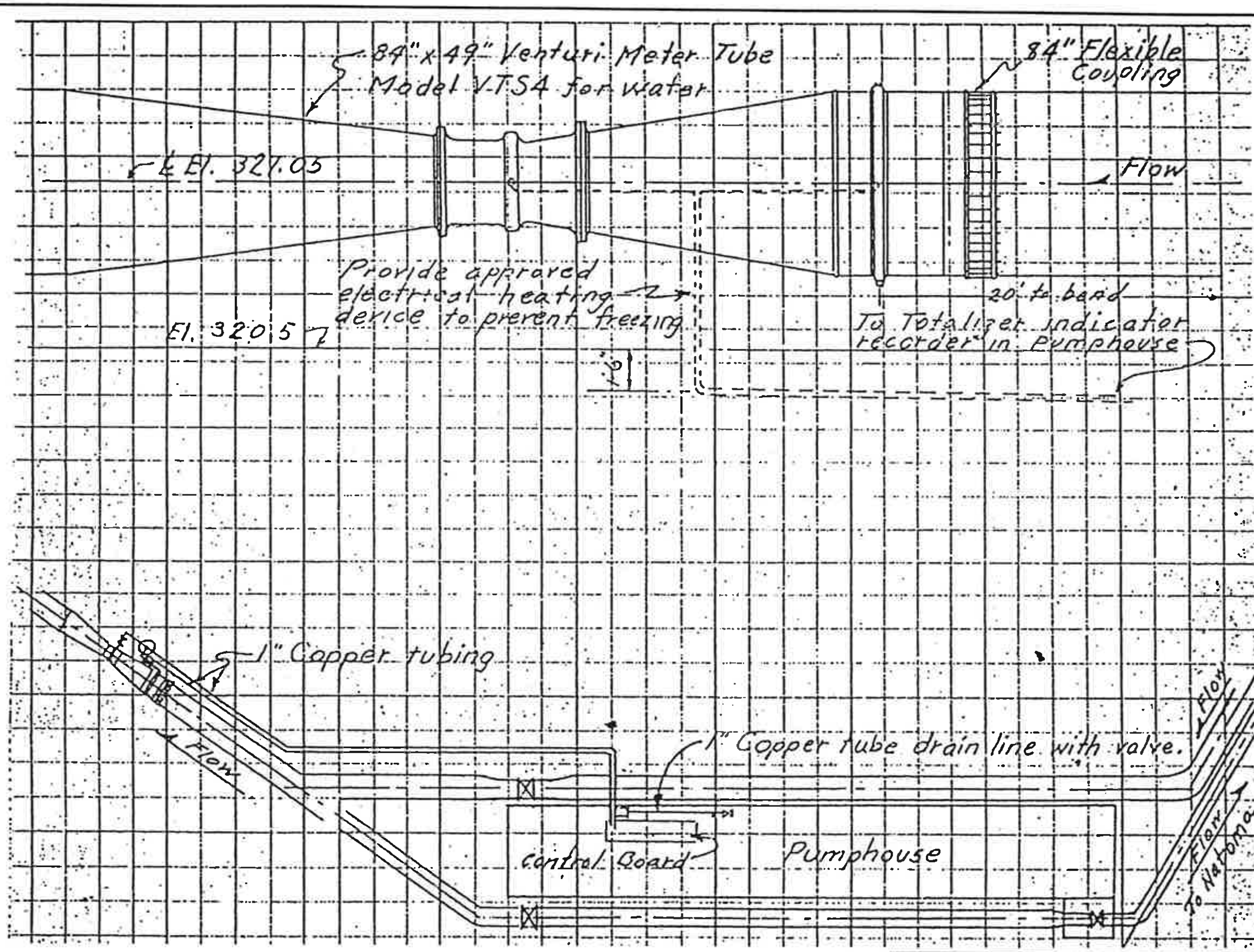
$$= 6.69 \times 1.46$$

$$= 9.76 \text{ feet}$$

Thus, with a 14 foot expansion length, no flow separation is expected.

Finally, there is the question of whether the venturi will still perform effectively its function as a flow meter. All that will be required is to extend the effective range of the transducers and to recalibrate or replace the conversion functions in the rest of the instrumentation system.

In summary, except for the need to extend the range of the instrumentation capability, no action is foreseen relative to the venturi.



Note: Figure from Corps 1951 Calculations.

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FOLSOM DAM - WATER SUPPLY PUMPING CAPACITY
NORTH FORK 84" LINE VENTURI METER

Checked By Peter V. Jacke Date 1/26/96	Project No.	Figure No.
Approved By B Date 1/31/96	043.9501	7-1

8. VORTEX FORMATION

The USBR has expressed concern about the possibility of vortex formation as reservoir water surface elevations fall below El. 340 to 335 and then approach minimum pool (El. 327). They have installed a special pump at a tap in one of the penstocks to provide an alternative to the normal water intake, in case vortex problems become so severe as to drastically limit pumping capabilities. They have indicated that modeling studies may be required to fully characterize vortex formation potential at the pool elevations of concern. Although the USBR has expressed concern about vortex formation, no instance of vortex formation has yet been observed, even under the relatively low reservoir conditions that prevailed during the late 1980's and early 1990's drought (Sanford and Joye, 1995).

Vortex formation potential is a function of intake geometry and flow (or velocity), as well as reservoir water surface elevation (or intake submergence). At very low flow rates (e.g. 50 cfs), and the 6.5 feet of submergence available for the Folsom water supply intake at minimum pool, no vortex formation would be expected, even recognizing the unsymmetric geometry of the approaching flow.

Figure 8-1 presents various definitions of the approximate boundary between the zones where vortex formation is likely versus unlikely, based on hydraulic conditions. Many of these relationships have been developed in terms of Froude Number (Fr). Some of these definitions are more conservative than others and some assume very severe geometries. For example, Reddy and Pickford (1972) indicated the submergence (s) over diameter (d) relationship $s/d = 1+Fr$ to define an envelope line above which vortexes would not be expected even in rectangular sumps. Although the Folsom intake approach is unsymmetric, it is not as confined as a sump. The two relationships that are likely to best represent the Folsom water supply intake are those by Gordon (1970, unsymmetric) and Knauss (1987). They have been given bolder lines in Figure 8-1.

Various pumping capacities for the Folsom Project water supply system are also shown on the diagram-- the present capacity, the proposed capacity with two additional pumps, and the prospective ultimate capacity of 400 cfs at minimum pool. The relationships generally indicate the following:

- Vortex formation is extremely unlikely with the existing pumping facility, even at minimum pool (El. 327), because of the limited delivery capacity (about 108 cfs).

- The potential for vortex formation is slightly greater when the two new pumps are added, but a vortex still may not occur (even at minimum pool) . This is because delivery capacity (at about 208 cfs) is still relatively modest when compared to the intake diameter and submergence.

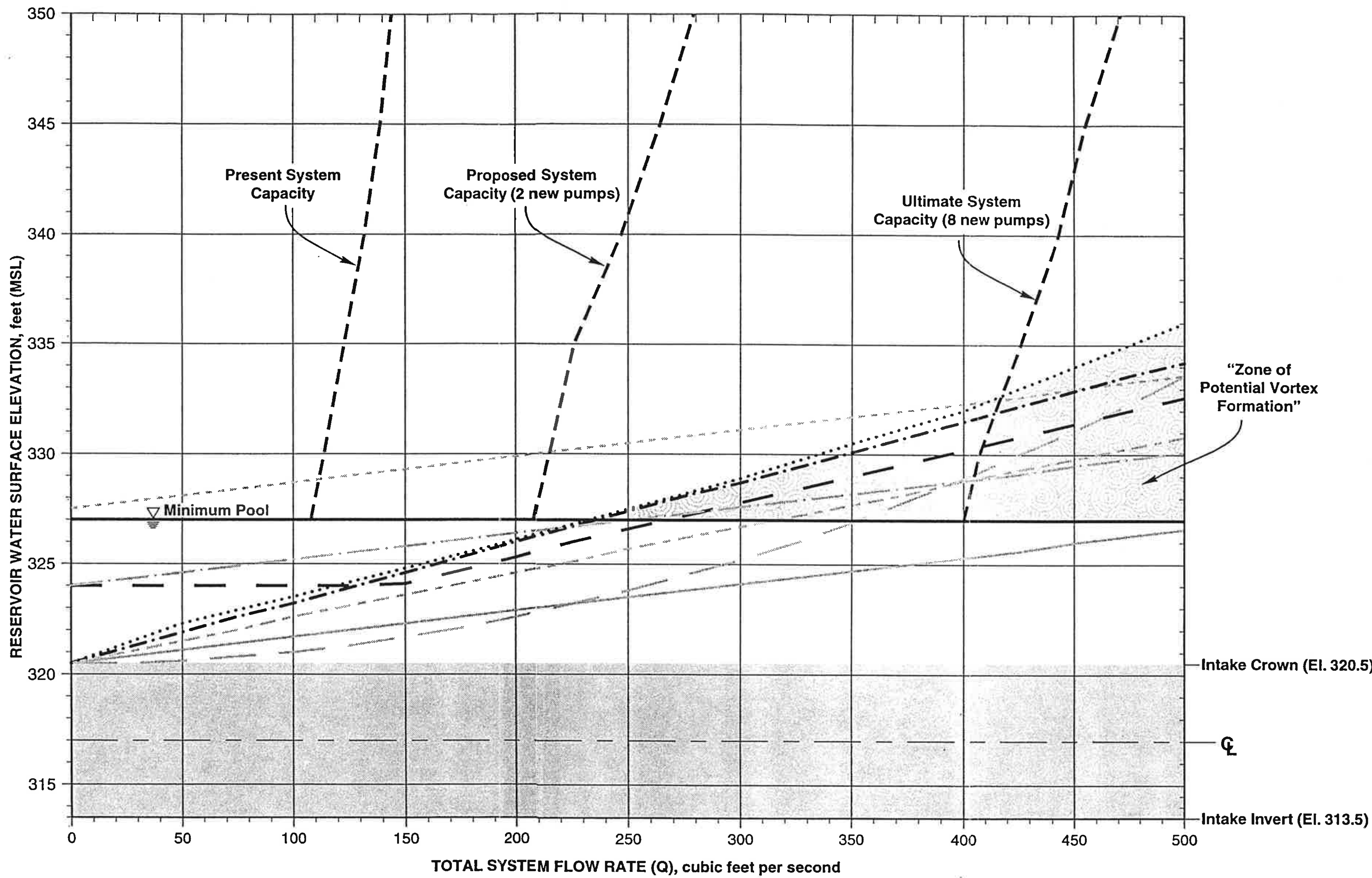
- Vortex formation is likely to occur if additional pumping capacity is added and delivery of 400 cfs is attempted at minimum pool. Indeed, critical (air entraining) vortex formation conditions should be expected to develop at approximately El. 332 when pumping at 400 cfs.

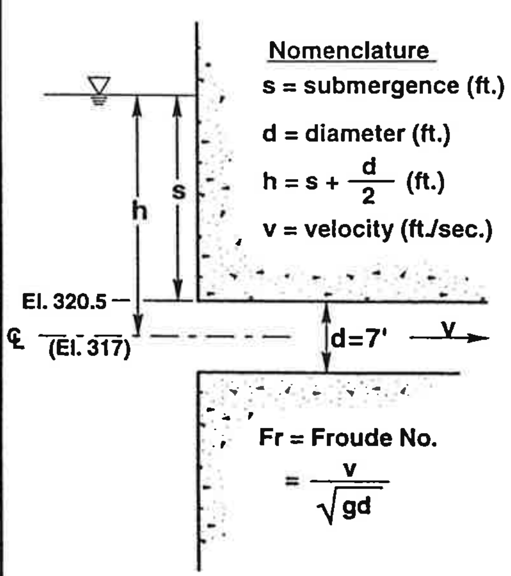
A zone of high vortex formation potential has been indicated for the Folsom Dam water supply intake based on the technical literature reviewed. This zone assumes no vortex defeating actions are taken, other than to limit pumping sufficiently to prevent formation of air entraining vortexes. The constraint that such an approach places on water supply deliveries is quite modest. In the final 5± feet of pool drawdown a gradual decrease of pumping rate from 400± cfs down to 235± cfs would be required. More severe restrictions might well be expected due to rationing programs.

The above pumping restrictions due to vortex formation might be considered unacceptable--e.g., in case of an emergency situation such as a large fire. If so, various actions could be considered to defeat the vortexes. A large floating (probably wooden) raft could be constructed over the intake area to resist the swirl and impede the vortex access to air. Guide vanes, air traps, air release valves and vacuum pumps could be installed between the intake and the pumping plant to combat detrimental vortex impacts. Many of these actions are relatively economical and could be implemented relatively quickly if necessary.

The USBR has indicated that a hydraulic model study could be conducted to better characterize vortex formation potential at the Folsom Dam water supply intake. The USBR would likely insist on such a study if water supply agencies wanted assurance of a full 400 cfs pumping capacity at minimum pool (El. 327). The USBR Denver hydraulic laboratory has estimated that such a study could be performed for approximately $63,000 (April, 1995).

Based on the modest likelihood of vortex formation at existing and proposed pumping capacities (with two additional pumps) it seems that such a modeling study can be postponed. Similarly any action to develop vortex defeating facilities would seem premature. The issue of vortex formation potential can and should be reconsidered when the next project to increase Folsom Project pumping capacity is initiated.





EXPLANATION

- Estimated upper boundary of vortex formation zone.
- $^s/_d = 1 + Fr$ (Ready and Pickford, 1972; most conservative).
- $s = 0.4\ vd^{0.5}$ (per Gordon, 1970, unsymmetric).
- $(h/d)_{critical} = 2Fr + {}^1/_2$ (Knauss, 1987).
- $s = 5\frac{v^2}{2g}$ (Namikas, 1995).
- $s = 0.3\ vd^{0.5}$ (per Gordon, 1970, symmetric).
- $^s/_d = 0.5 + Fr$ (after Ready and Pickford).
- $^s/_d = Fr$ (Ready and Pickford, 1970).

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Mountain View, California

FOLSOM DAM - WATER SUPPLY PUMPING CAPACITY
VORTEX FORMATION BASED ON WATER SURFACE ELEVATION AND FLOW

Checked By Peter V. Jack Date 1/26/96	Project No.	Figure No.
Approved By B Date 1/31/96	043.9501	8-1

9. PUMPING FLOOR LAYOUT

Based on the discussions of hydraulic factors in the previous sections, improvements to increase the Folsom Project pumping capacity can be limited to changes in the pumping plant. Accordingly, this section focuses on the physical arrangement of the hydraulic conduits and machinery on the pumping floor of the pumping plant.

USBR Drawing 485-208-980 was obtained from the Bureau as a CAD file and was modified to show the addition of Pumps 7 and 8 as presented in Figure 9-1. Pump dimensions were obtained from Ingersoll-Dresser for the 750-LNE-1050 and those dimensions result in the indicated layout.

The most important design/constructibility consideration is the indicated enlargement of the suction piping to 36" diameter (from the existing 30") and the discharge piping to 30" diameter (from the existing 24"). This involves cutting back the cones that provide transitions from the headers into the pumping plant. Around its circumference, the cone wall diverges from the center line of the pumping plant piping at an angle of 6 degrees. Thus, in order to gain 3" of piping radius (or 6" of diameter) the cones must be shortened by 2.38 feet. This is possible while still leaving sufficient working room between the headers themselves and the new weld required to install the larger diameter pipes. On the suction cone, 7.42" of clearance remains (less pipe thicknesses) at the tightest location and on the discharge cone, 10.52" of clearance remains (less pipe thicknesses).

Implementing the proposed cone shortenings will require:

- Removal of the concrete that surrounds the pipe/cone where it now penetrates the pump house wall.
- Removing the concrete embedment outside the wall, between the wall and the header, as necessary to establish acceptable working space.
- Draining the header.
- Precision cutting the cone to receive a new larger (36" or 30") diameter pipe.
- Precision welding the pipe to achieve the alignment needed to reach the pump.
- Renewal of the interior and exterior pipe coatings.
- Installation of the needed valve at the end of the new pipe.
- Refilling of the header.
- Replacement of the piping's concrete embedment and the concrete wall surrounding the pipe.

Since the headers must be drained, it is necessary to coordinate this operation with the system operating needs to maintain water deliveries. For the discharge piping, a period of gravity flow (usually a month or more) can be used to accomplish the needed modification. For the suction side (which is also the gravity bypass), the opportunities are more restricted. However, there may be a period during the winter when adequate gravity flow can be achieved through the Natoma gravity intake line and back flow to the pumping plant discharge header to feed the North Fork line. This could provide a several day period to accomplish the needed work. Also, the work might be scheduled in conjunction with some other need for draining the suction side of the system.

If the logistics for scheduling the suction side modification of the cones are simply unworkable, the installation can adopt an expansion to 36" diameter inside the pumping plant. It is noted however, that the suction side header will have to be drained for a short time to accomplish the needed piping modifications inside the plant, even if the cones are not cut back.

Other aspects of the layout are straightforward:

- The indicated valves will likely be AWWA C504 Class 150B butterfly valves. On the suction side a manual actuator will be sufficient. On the the discharge side a motorized actuator (AWWA C540) with sufficient torque capability to work during transitions in pump operation will be required. In checking conformance to AWWA C504, the water velocity at maximum pump discharge was found to be high. Therefore the discharge side valve was increased to 36" diameter to prevent valve actuation difficulties.

- The indicated flexible couplings will, at the least, conform to AWWA C219.

- The pump, motor base, and anchorage system will be designed in accordance with the manufacturer's recommendations, working (to the extent possible) with the dimensions of the removable pump floor slabs that are built into the existing structure.

Detailed design will need to address several additional issues such as the pump/motor/valve response to a power outage. The pump and motor will have to be capable of tolerating backflow or the valve system will have to be designed to prevent backflow.

Installation of the equipment presents an important constructibility issue relative to the capacity of the existing crane, which is rated at 7.5 tons. Both the pump and the 1500 hp motor will exceed this limit, so both will need to be disassembled and installed in portions. This will require supervision of manufacturer's representatives in order to protect warranties.

Table 9-1 presents the cost estimate (in 1995 dollars) for the hydraulic and mechanical components of the pumping plant improvements, including the needed concrete demolition and replacement for the cone work and pump base.

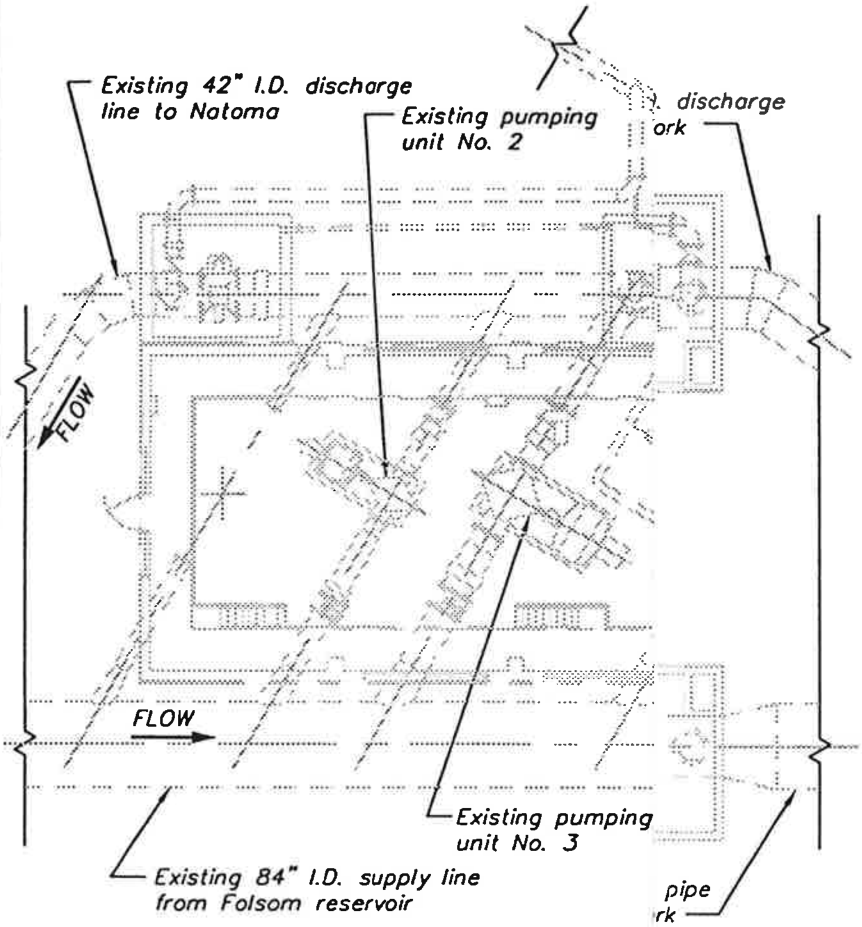
TABLE 9-1
COST ESTIMATE FOR
PUMPS, VALVES, PIPING & INSTALLATION*

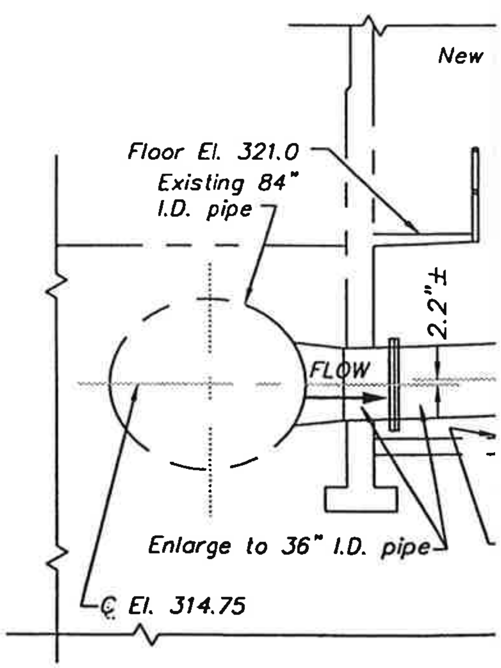
For each unit:

Pump (ID 750-LNE-1050)	$ 110,000
Valve (36"; 150B; Manual)	8,000
Valve (36"; 150B; Motor)**	13,000
Flexible couplings (2 @ 36"; $500 each)	1,000
Expander, reducer, and other piping	8,000
Subtotal materials	$ 140,000
Cutting back cones (labor & mat.; 2 @ $17,500)	35,000
Other installation (labor & mat., incl. pump and motor base)	60,000
Subtotal (for each unit)	$ 235,000
Taxes, mobilization, clean up, etc. (15%)	35,000
Total (for each unit)	$ 270,000

* Estimate is given at mid-1995 price levels. Note that the building modification costs for accommodating electrical equipment are included in the electrical estimates in Section 10.

** The motor-operated valve for the variable frequency drive alternative has been chosen based on the assumption that no long-term, high head throttling will be required.





REFERENCE BUREAU DRAWINGS

PLAN, ELEVATIONS AND DETAILS -- 485-D-1415
REINFORCEMENT DETAILS -- 485-D-1416
EQUIPMENT ARRANGEMENT -- 485-D-1417
POWER CONDUIT PLAN -- 485-D-1420
LIGHTING DIAGRAM AND CONDUIT PLAN -- 485-D-1421
VALVE SUPPORT DETAIL -- 40-D-5913

485-208-980

ESA CONSULTANTS INC.
MOUNTAIN VIEW, CALIFORNIA

FOLSOM DAM – WATER SUPPLY PUMPING CAPACITY

ADDITION OF UNITS 7 AND 8
GENERAL ARRANGEMENT

DRAWN BY	DATE	PROJECT NO.	DRAWING NO.
DMO	1/31/96	043.9501	9-1
CHECKED BY B	DATE 1/31/96		

ARWA-202

10. ELECTRICAL

10.1 Summary

The purpose of this electrical study is to analyze different types of motors and motor drives/controllers and recommend the most suitable and cost effective electrical system to operate the pumps. Discussions on the pumps and selection of their sizes and quantity are covered in the preceding sections of this report.

This study based on the pumping plant's operational requirements, considers three alternatives for selection of motors and motor drives/controllers; (1) variable speed motors with variable frequency drives (VFD's), (2) two speed motors with two speed controllers and, (3) single speed motors with single speed controllers.

Induction motors are more suitable for the pumping plant applications. Also, they are considerably less expensive and offer comparable efficiency and power factor as the synchronous motors.

The single speed and two speed motor alternatives would provide a variety of water delivery options and also would be less expensive. These two alternatives, however, would not provide the versatility and energy efficiency of a water delivery system that can be achieved with the variable speed system. Some flexibility in water delivery service, however, may be realized by selecting different pump combinations in response to the varying water demands.

The variable speed system would meet the pumping plant's water delivery service and operational requirements more closely than the single and two speed systems as discussed in the report. As such, the variable speed induction motors with VFD's using the pulse width modulated (PWM) technology are recommended.

10.2 Motors and Motor Drives/Controllers

Variable speed, two speed and single speed systems offer three alternatives to handle the varying requirements of water deliveries by the plant. The variable speed system can automatically control the water deliveries by change of the motor speeds in response to the preset water demands. The two speed and single speed systems in conjunction with the existing pumping plant motors could, also, provide a variety of water delivery schedules in response to the water demands by manually operating a pre-selected group of pumps for any given pumping conditions.

The state-of-the-art changes in motor designs have introduced newer induction motors which compare favorably with synchronous motors in regard to higher efficiency and power factor for pumping plant applications. The selection of induction motors results in significant cost savings.

Variable speed motors can be operated by magnetic drives or VFD's. Such drives would continuously control motor speeds based on the requirements of the water deliveries. Magnetic drives have been in use for many years. They provide satisfactory performance and are less expensive. However, the magnetic drives have significantly lower efficiencies at lower speeds. These magnetic drives are coupled together with the motor and pump and are installed as one unit at the pump location. This poses a major drawback in use of the drives due to the limited space at the pump location. VFD's on the other hand can be installed remotely from the pump and motor. As such, they do not cause similar space problems as the magnetic drives.

Modern VFD's use current source inverter (CSI) power structures and/or full pulse width modulated (PWM) switching pattern technologies. These technologies have provided more than one choice for solid state VFD selection. The PWM technology provides the best power quality output and efficiency at all speeds, and enormous diversity in operation with near perfect sinusoidal output waveform. Medium voltage (4160 V) VFD's with sophisticated modern technology (PWM) to control current and voltage harmonics at all load levels and speeds are more expensive and complex compared to the VFD's which only use the CSI technology and offer much less harmonics control and power quality.

The VFD's which use PWM technology are considered to be more suitable for the larger size motors such as those (1500HP) being considered for this pumping plant. The CSI technology which offers less harmonics control, less power quality and lower efficiencies at lower speeds could be a major concern in this application. Besides the low power quality and efficiency, the harmonics generated by this type of VFD's could have a serious impact on the utility grid system and on the operation of solid state (computer) loads.

Two speed (590 rpm and 505 rpm) motors with less than a factor of two difference in the speeds, as required for this pumping plant, would require two windings, one for each speed. This would result in a larger diameter and comparatively more expensive motor than the single winding motor. A two speed system would provide a better control of water deliveries compared to the single speed system by operating the pumps at different speeds and in different combinations with other pumps. Two speed motor controllers are compact in size compared to VFD's. VFD's require significantly larger foot print in the electrical control area than the single or two speed motor controllers.

Single speed motors and controllers would provide the simplest form of pumping plant system similar to the existing system. Also, this system would be less expensive in the initial installed costs compared to the other systems. However, the operating cost of the system considering the power consumption would outweigh the initial cost savings advantage. Also, this system would not provide the same degree of flexibility in water deliveries as the two and variable speed systems described above.

10.3 Power System Description and Arrangement

a. Description

The pumping plant switchgear is presently served by two redundant feeder lines from the main substation. The present cable capacity of each of the two feeders is 260 A (Amps;1-350 kcMil/ phase). These feeders are planned for replacement by the next larger size cables. The cable capacity of the new feeders after replacement by the new cables (500 kcMil/phase) will become 465 A.

Existing system loading is depicted in Table 10-1. This table shows that the total load on the existing switchgear busses and cables serving the existing switchgear is 257 A. The existing switchgear busses which are rated at 1000 A are adequately sized to serve the present loads. The cables serving the switchgear, however, are rated at 260 A (1-350 kcMil/ phase) and they are considered marginally sized to serve the present loads.

The new loading with the addition of the two new pumps, each rated at 1500 HP will become 617 A (see Table 10-2). This loading is significantly over the rating of the existing cables (1-350 kcMil/phase) and also about 25% over the rating of the new cables (465 A; 1-500 kcMil/phase).

In order to keep the new and existing loading of the pumping plant within the capacity of the new cables, only one of the new motors should be added to the existing switchgear. The second new motor should be served from a new switchgear which should be powered from one of the redundant (second) power feeders presently serving the existing switchgear. This arrangement of the new loads would result in a total loading of 437 A on one feeder and 180 A on the other feeder.

Some of the existing motor loads should also be shifted to the new switchgear to divide the total load between the two switchgear equally. This would make the system more reliable and flexible in operation. A separate load study to redistribute the existing and new loads on the two switchgear to provide improved system reliability should be considered during the design stage of the project.

b. Arrangement

The power supply and equipment arrangements covered in this section are for the recommended variable speed motor system (VFD's with PWM technology).

The power supply to the existing and new loads is shown on the Single Line Diagram, Figure 10-1. The arrangement of the existing and new switchgear is shown on the Electrical Equipment Plan Drawing, Figure 10-2.

The addition of the VFD switchgear for the new variable speed drives would require expansion and remodelling of the existing pumping plant building as shown in Figure 10-2. This arrangement should also be reviewed again at the design stage of the project if redistribution of the existing and new loads is considered for improved system reliability.

10.4 Electrical Cost Estimate

The electrical cost estimates for three alternatives (in mid-1995 dollars) are presented in Tables 10-3, 10- 4, 10- 5 and 10-6. The total costs for the electrical alternatives are as follows:

a. Variable Speed System:

(1) The total cost to install variable speed (CSI Technology) system for two new motors (Table 10-3) = $ 970,500

(2) The total cost to install variable speed (PWM Technology) system for two new motors (Table 10-4) = $ 1,120,500

b. Two Speed System:

The total cost to install two speed system for two new motors (Table 10-5) = $ 829,600

c. Single Speed System:

The total cost to install single speed system for two new motors (Table 10-6) = $ 462,100

TABLE 10-1
EXISITING LOADS

Pump No.	Speed (rpm)	Horsepower	Full Load Amps
2	1200	250	30
3	720	600	69
4	900	400	48
5	900	400	48
6	450	550	62
	Total	2,200	257

TABLE 10-2
EXISITING AND NEW LOADS

Pump No.	Speed (rpm)	Horsepower	Full Load Amps
2	1200	250	30
3	720	600	69
4	900	400	48
5	900	400	48
6	450	550	62
7 (new)	600	1500	180
8 (new)	600	1500	180
	Total	5,200	617

TABLE 10-3
ELECTRICAL COST ESTIMATE
VARIABLE SPEED MOTORS AND DRIVES (CSI TECHNOLOGY)

Description	Qty	Unit	Manhours /Unit	Total Manhours	Labor Cost/ Manhour	Material unit cost	Total Labor	Total Material	Total Cost
Variable Speed Motor	2	EA	180	360	$45	$155,000	$16,200	$310,000	$326,200
Variable Speed Drive	2	EA	80	160	$45	$165,000	$7,200	$330,000	$337,200
4 kV Cables, Cond. & Connect.	—	LS	—	—	—	—	—	—	$10,000
4 kV Switchgear Modifications	—	LS	—	—	—	—	—	—	$15,000
480 V Power Modifications	—	LS	—	—	—	—	—	—	$8,000
Building Modifications	—	LS	—	—	—	—	—	—	$80,000
								Total Direct Cost =	$776,400
								Bond, Insurance, Taxes, Profit (25%) =	$194,100
								TOTAL COST =	$970,500

ESA Consultants

TABLE 10-4
ELECTRICAL COST ESTIMATE
VARIABLE SPEED MOTORS AND DRIVES (PWM TECHNOLOGY)

Description	Qty	Unit	Manhours /Unit	Total Manhours	Labor Cost/ Manhour	Material unit cost	Total Labor	Total Material	Total Cost
Variable Speed Motor	2	EA	180	360	$45	$155,000	$16,200	$310,000	$326,200
Variable Speed Drive	2	EA	80	160	$45	$225,000	$7,200	$450,000	$457,200
4 kV Cables, Cond. & Connect.	—	LS	—	—	—	—	—	—	$10,000
4 kV Switchgear Modifications	—	LS	—	—	—	—	—	—	$15,000
480 V Power Modifications	—	LS	—	—	—	—	—	—	$8,000
Building Modifications	—	LS	—	—	—	—	—	—	$80,000
								Total Direct Cost =	$896,400
								Bond, Insurance, Taxes, Profit (25%) =	$224,100
								TOTAL COST =	$1,120,500

ESA Consultants

TABLE 10-5
ELECTRICAL COST ESTIMATE
TWO SPEED MOTORS AND STARTERS

Description	Qty	Unit	Manhours /Unit	Total Manhours	Labor Cost/ Manhour	Material unit cost	Total Labor	Total Material	Total Cost
Two Speed Motor	2	EA	180	360	$45	$290,000	$16,200	$580,000	$596,200
Motor Starter	2	EA	50	100	$45	$15,000	$4,500	$30,000	$34,500
4 kV Cables, Cond. & Connect.	—	LS	—	—	—	—	—	—	$10,000
4 kV Switchgear Modifications	—	LS	—	—	—	—	—	—	$15,000
480 V Power Modifications	—	LS	—	—	—	—	—	—	$8,000
								Total Direct Cost =	$663,700
								Bond, Insurance, Taxes, Profit (25%) =	$165,900
								TOTAL COST =	$829,600

TABLE 10-6
ELECTRICAL COST ESTIMATE
SINGLE SPEED MOTORS AND STARTERS

Description	Qty	Unit	Manhours /Unit	Total Manhours	Labor Cost/ Manhour	Material unit cost	Total Labor	Total Material	Total Cost
Single Speed Motor	2	EA	180	360	$45	$150,000	$16,200	$300,000	$316,200
Motor Starter	2	EA	50	100	$45	$8,000	$4,500	$16,000	$20,500
4 kV Cables, Cond. & Connect.	—	LS	—	—	—	—	—	—	$10,000
4 kV Switchgear Modifications	—	LS	—	—	—	—	—	—	$15,000
480 V Power Modifications	—	LS	—	—	—	—	—	—	$8,000
								Total Direct Cost =	$369,700
								Bond, Insurance, Taxes, Profit (25%) =	$92,400
								TOTAL COST =	$462,100

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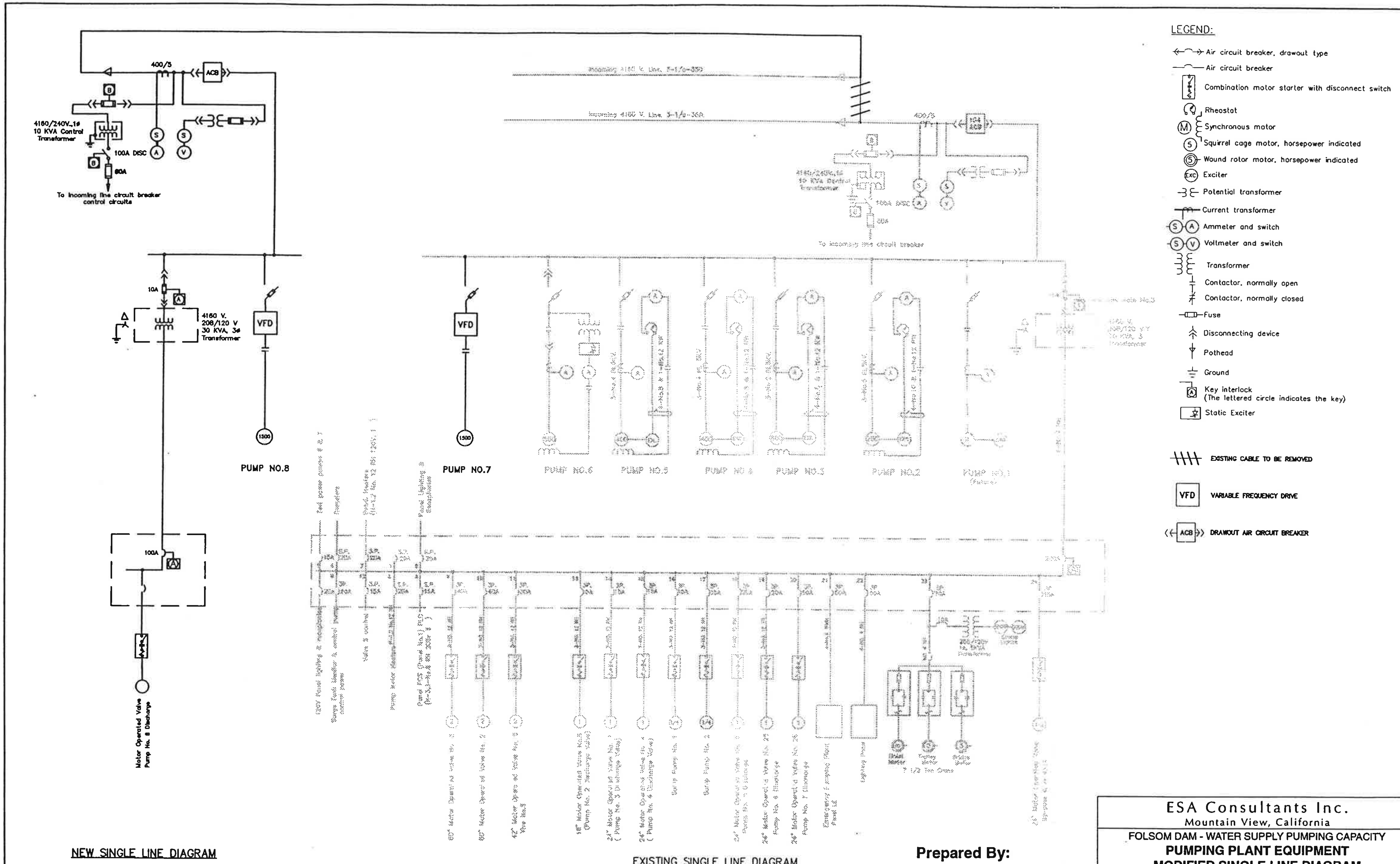


Prepared By:

SAI ENGINEERS, INC.
295 Santa Ana Court, Sunnyvale, CA 94086

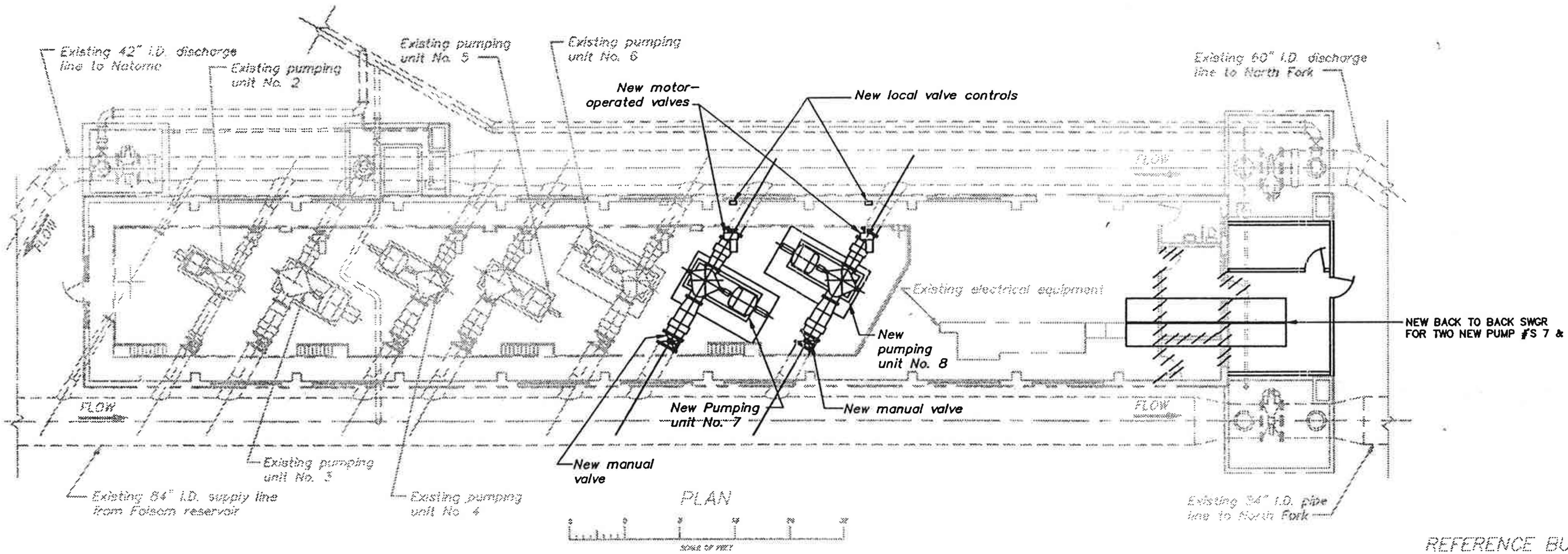
ESA Consultants Inc.
Mountain View, California

FOLSOM DAM - WATER SUPPLY PUMPING CAPACITY

PUMPING PLANT EQUIPMENT MODIFIED SINGLE LINE DIAGRAM

Checked By	Date	Project No.	Figure No.
Approved By B	Date 1/31/96	043.9501	10-1

ARWA-202



REFERENCE BUREAU DRAWINGS

PLAN, ELEVATIONS AND DETAILS ---	485-D-1415
REINFORCEMENT DETAILS ---	485-D-1416
EQUIPMENT ARRANGEMENT ---	485-D-1417.
POWER CONDUIT PLAN ---	485-D-1420
LIGHTING DIAGRAM AND CONDUIT PLAN ---	485-D-1421
VALVE SUPPORT DETAIL ---	40-D-5913

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FOLSOM DAM - WATER SUPPLY PUMPING CAPACITY
PUMPING PLANT EQUIPMENT
ELECTRICAL EQUIPMENT PLAN

Checked By	Date	Project No.	Figure No.
Approved By ß	Date 1/31/96	043.9501	10-2

TABLE 11-1
OVERALL COST ESTIMATE FOR
TWO VARIABLE SPEED PUMPS
(PWM TECHNOLOGY)
(at mid-1995 price levels)

Hydraulic / Mechanical* (2 units @ $270,000)		$540,000
Electrical** (variable speed; PWM technology; 2 units)		1,120,000
	Subtotal	1,660,000
	Contingency (15%)	250,000
	Total Construction Cost	1,910,000
Engineering / Design (8%)		150,000
Bureau of Reclamation Design Review (2%)		40,000
	Subtotal	2,100,000
Construction Supervision (5%)		110,000
Bureau of Reclamation Supervision (3%)		60,000
	Total	$2,270,000
	Say	**$ 2.3 million**

* Includes civil work for pump, motor, and piping (see Section 9)

** Includes civil work for related building modification (see Section 10)

11. COST ESTIMATE

The overall cost estimate for the pumping capacity improvements are presented in Table 11-1 in mid-1995 dollars. This cost estimate is for two pumps with variable speed motors and the (more expensive) PWM variable frequency drive technology. The cost information is drawn from the more detailed estimates for the hydraulic/mechanical portion (Section 9) and the electrical portions (Section 10). Civil work costs for the respective portions of the project were included within those estimates. The subtotal of the aggregated estimate for two new pumping units is $1.66 million. Inclusion of a 15% contingency brings the total to $1,910,000. This is a construction cost estimate including equipment, materials and installation. The estimate of total project costs needs to include allowances for design, coordination with the USBR, construction management, and administration. Such allowances have been indicated in Table 11-1, resulting in a total project cost estimate of $2,270,00 which has been rounded upward to $2.3 million. It is noted that the intensity of USBR review is not predictable and the amount of effort required to coordinate with the Bureau and respond to their comments and concerns is likewise unknown. Thus, the allowances indicated are initial estimates.

12. REFERENCES

ESA Consultants Inc. 1994. Increasing Peak Water Supply Flows from Folsom Dam. Prepared for City of Roseville Department of Environmental Utilities. ESA Consultants Inc., Mountain View, CA.

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USBR, 1995b. Bill Joye: personal communication

U.S. Corps, 1951. Hydraulic Calculations for Natomas and North Fork Pipelines.

APPENDIX A
FOLSOM DAM WATER SUPPLY PUMPING TEST
ON NOVEMBER 18, 1994

APPENDIX A
FOLSOM DAM WATER SUPPLY PUMPING TEST ON NOVEMBER 18, 1994

A pumping test of the Folsom Dam water supply facilities was conducted on November 18, 1994. The primary purpose of the test was to confirm or refine the calculated delivery capacity and head loss findings reported in an earlier study (ESA Consultants, November 1994).

Preparatory work for the pump test included identification of measurement locations, installation of needed gages, development of data sheets and coordination with the affected water supply agencies to arrange for participation and establish appropriate operating conditions during the test. The test data are included as Attachment 1. Some clarifying questions and answers regarding test conditions are set forth in Attachment 2.

The test was performed in two distinct portions. The first portion of the test was performed while both Roseville and San Juan had their throttling valves fully open. The second portion was conducted while San Juan was fully open and Roseville was closed. Thus, Roseville head loss can be characterized only from the first portion of the test. The second portion gives an additional data point for San Juan.

There were numerous slight inconsistencies in the recorded data. These were resolved by using the results of both tests, the initial Corps (1951) head loss calculations, and considerable engineering judgment. The adjusted data that were adopted as an adequate, internally consistent representation of the test data are set forth in Table A-1. This table also provides the unadjusted field data for comparison. The following observations are provided:

- There is a flow rate discrepancy between the USBR North Fork venturi reading and the sum of the Roseville and San Juan treatment plant readings. The difference is approximately 4 percent and is likely due to slight miscalibration of one or more of the flow meter transducers.

- Two of the pressure gages appear to give slightly high results-- Point C (PC) appears high by 1 psi and the USBR stand pipe appears high by 1.5 feet (approximately 1 psi).

- The water surface elevations in the Roseville and San Juan rapid mix chambers are the ideal hydraulic grade measurement in each case, assuming that no throttling is occurring. Thus these elevations were inferred based on the plant hydraulic regimen and flow and they were used instead of PE and PF.

- Steady state conditions were not reached in the second test, but the inferred steady state numbers are not unreasonable compared to the readings available.

The indicated steady state readings translate into hydraulic grade lines for the system as plotted in Figure A-1 and A-2 and further tabulated in Table A-2.

Initial calculations of segment head losses using the measured pressures indicated substantial internal inconsistencies; i.e., one segment would have higher head losses than expected based on the Corps 1951 calculations and another would have lower head loss than expected. In some cases negative head losses were found and in others, seemingly significant discrepancies could be rationalized away based on the limited precision of the measurements.

Ultimately, by accepting the pressure measurements at the Hinkle "Y" (PD) and adopting the compromise system flow rates indicated in Table A-1, application of the Corps (1951) head loss factors gave reasonable results for all segments. This sequence of analyses is presented in Table A-3 and A-4 for the North Fork line. The calculations of head loss from PB to PD for the initial test using the Corps head loss factor showed agreement with measured pressures within 0.1 psi. It was primarily because of this result that PD was accepted, PB was slightly adjusted and the Corps head loss figures were then used everywhere possible.

From the Hinkle "Y" to San Juan and Roseville, no relevant Corps calculations were available; the piping systems were designed and constructed after 1951. A preliminary assessment of San Juan head loss was available from the initial ESA study but it required confirmation. Accordingly, head loss factors were calculated for San Juan and Roseville using the pressure measurement at Hinkle "Y" and the estimated water surface elevations in the first open tanks at each water treatment plant-- the rapid mix chambers. Those calculations are documented in Table A-5.

Finally, the head loss factors used in this study for each existing segment of the water supply pumping system are shown in Table A-6.

TABLE A-1
FOLSOM DAM WATER SUPPLY SYSTEM
PUMPING TEST DATA

		Pressures							Flows					
	Time	PA (psi)	PB (psi)	PC (psi)	PD (psi)	PE (ft. of H2O)	PF (psi)	USBR Stand Pipe (ft. elev.)	QN (cfs)	QS (cfs)	QR (cfs)	QT [a] (Sum N,S,R) (cfs)	USBR North Fork [a] (cfs)	QT [a] (Sum N,NF) (cfs)
FIRST TEST	10:40	19	53.5	32	20	17.99	9	440	10	125	62.2	197.2	180	190
	10:45	19	53.5	32	20	17.86	9	440	10	125	63.3	198.3	180	190
	10:50	19	53.5	32	20	17.95	9	440	10	125	63.3	198.3	180	190
	10:55	19	53.5	32	20	17.90	9	440	10	125	63.3	198.3	180	190
	11:00	19	53.5	32	20	19.44	9	440	10	125	63.5	198.5	180	190
Steady State (b)	**10:45 to 11:00**	**18.6**	**53.6**	**30.9** [c]	**20.0**	**15.2** [d]	**8.5** [d]	**438.3** [e]	**10**	**120.6**	**61.2**	**191.8**	**181.8**	**191.8**
SECOND TEST														
	11:10	19	57.0		23	51.92			10	164	0	174	158	168
	11:15	19	57.0	35.4	23	59.28		445/448	10	164	0	174	158	168
Steady State (b,f)	**If Test Continued to 11:45 +/-**	**18.7**	**56.3**	**33.6** [c]	**23.0**	**55.94**	**8.6** [d]	**444.4**	**10**	**160**	**0**	**170**	**160**	**170**

Test Parameters:
Date: 11/18/94
Lake Level: 366.44
Pumps Running: 2, 3, 4, 5 & 6

Explanation:

QN = Flow to Natoma (City of Folsom & Prison)
QS = Flow to San Juan Water District
QR = Flow to City of Roseville
QT = Total Flow

PA: EL. 322.05 (Suction Header)
PB: EL. 314.52 (Jct. Grav. & Pump Disch.)
PC: EL. 365.95 (50 ft. downstream of the first Stand Pipe)
PD: EL. 388.36 (just upstream of Hinkle "Y")
PE: EL. 385.5 (Roseville)
PF: EL. 404.39 (San Juan)

Notes:

a There is a difference in flow reading between San Juan & Roseville versus USBR of around plus or minus 8.3 cfs. QT may be 190 cfs based on USBR reading of 180 cfs at North Fork Venturi during the first test. The discrepancy decreases to plus or minus 6 cfs during the second test.

b Steady state conditions adopted based on reconciling measurement inconsistencies.

c The pressure gauge at point C appears to read high by 1 psi.

d Since both Roseville and San Juan had their throttling valves wide open, the preferred data from the test is the water surface elevation in their rapid mix chambers (the initial, open-air tank in their treatment plants). These elevations were inferred (See Attachment 2) and used. The elevations used are consistent with some additional head loss from the point of pressure measurement to the rapid mix chambers.

e The USBR Stand Pipe readings appear to be high by approximately 1.5 feet.

f During the second test, it appears that points B, C, the USBR Stand Pipe, and Point E (Roseville) have not yet reached steady state. Inferred values represent an estimated ultimate steady state condition.

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TABLE A-2
FOLSOM DAM WATER SUPPLY SYSTEM
PUMPING TEST PRESSURES
CONVERTED TO ELEVATIONS

	Measured Pressure			Used Pressure		
	Pressure Reading		Head elevation	Head elevation	Adjusted Pressure	
First Test	(psi)	(ft of H2O)	(ft)	(ft)	(ft of H2O)	(psi)
PA (Suction Header)	19	43.85	365.90	364.98	42.93	18.6
PB (Jct. Gravity & Pump Disch.)	53.5	123.46	437.98	438.27	123.75	53.6
PC (50 ft. downstream of Stand Pipe)	32	73.85	439.80	437.27	71.32	30.9 [c]
PD (upstream of Hinkle "Y")	20	46.15	434.51	434.51	46.15	20.0
PE (Roseville)	N/A	17.90	403.40	400.70 [b]	15.20 [b]	N/A
PF (San Juan)	9	20.77	425.16	424.00 [b]	19.61	8.5 [b]
USBR Stand Pipe [a]	N/A	N/A	440	438.27 [c]	N/A	N/A
Second Test						
PA (Suction Header)	19	43.85	365.90	365.29	43.24	18.7
PB (Jct. Gravity & Pump Disch.)	57	131.54	446.06	444.35	129.83	56.3
PC (50 ft. downstream of Stand Pipe)	35.4	81.69	447.64	443.57	77.62	33.6 [c]
PD (upstream of Hinkle "Y")	23	53.08	441.44	441.44	53.08	23.0
PE (Roseville)	N/A	51.9/59.3	437.40/444.80	441.44	55.94	N/A
PF (San Juan)	-	-	-	424.35 [b]	19.96	8.6 [b]
USBR Stand Pipe [a]	-	-	445/448	444.35 [c]	N/A	N/A

a The USBR "Stand Pipe" reading is really located at PB, and reflects the hydraulic pressure at the junction of the gravity 84" line and the pumping discharge line when pumping operations are underway.

b The pressure measurements for Roseville and San Juan were not used. The estimated water surface elevation in the first open-air tanks were used instead. These changes to downstream measurement locations would show slightly lower pressures due to additional head losses.

c The adjusted pressure readings used are within the precision of the gage readings except for the following: (1) The pressure gage at PC appears to yield readings that are high by 1 psi; and (2) The USBR Stand Pipe transducer appears to yield readings that are high by approximately 1.5 feet.

TABLE A-3
NORTH FORK LINE
HEAD LOSS AND "k" VALUE

1. Corps of Engineers (1951) head loss calculation from junction of gravity feed and pumping plant discharge lines to Hinkle "Y":

   - Flow: 250 cfs
   - Head loss: 7.10 feet
   - Head loss factor: $k = \frac{\sqrt{\Delta h}}{Q}$

   k = 0.010658*

2. Pumping test measurements from junction of gravity feed and pumping plant discharge lines to Hinkle "Y":

   - First Test

	k from measurements		k from adjustments
Q (cfs)	PB - PD (Δh = 3.47 ft)	Stand Pipe - PD Δh = 5.49 ft	PBadj - PD (Δh = 3.76 ft)
180	0.010349	0.013017	0.010773
181.8	0.010246	0.012888	**0.010666***
188.3	0.009893	0.012443	0.010298

   - Second Test

	k from measurements		k from adjustments
Q (cfs)	PB - PD (Δh = 4.62 ft)	Stand Pipe - PD (Δh = 5.06 ft)	PBadj - PD (Δh = 2.91 ft)
158	0.013604	0.014237	0.010797
160	0.013434	0.014059	**0.010662***
164	0.013106	0.013716	0.010402

* Note that compromise flows and adjusted pressures yield "k" values that are essentially the same as the Corps' values.

TABLE A-4
NORTH FORK LINE
HEAD LOSS AND "k" VALUES USING POINT C (PC)

1. Corps of Engineers (1951) Head Loss Calculation:

A. From junction of gravity feed and pumping plant discharge (PB) to PC

Flow 250 cfs
Head Loss 1.90 feet
Head Loss Factor: $k=(\Delta h)^{0.5}/Q=$ **0.005514***

B. From PC to Hinkle "Y" (PD)

Flow 250 cfs
Head Loss: 7.10-1.90= 5.20 feet
Head Loss Factor: $k=(\Delta h)^{0.5}/Q=$ **0.009121***

2. Pumping Test Measurements

A. From junction of gravity feed and pumping plant discharge (PB) to PC

First Test

	k From Measurements		k From Adjustments
	PB - PC	Stand Pipe - PC	PBadj. -PCadj.
Q (cfs)	(Δh=-1.82 ft)	(Δh= 2.02 ft)	(Δh= 1.00 ft)
180	calculation	0.007896	0.005556
181.8	is not	0.007818	**0.005501***
188.3	sensible	0.007548	0.005311

Second Test

	k From Measurements		k From Adjustments
	PB - PC	Stand Pipe - PC	PBadj. -PCadj.
Q (cfs)	(Δh=-1.58 ft)	(Δh=-1.14 ft)	(Δh= 0.78 ft)
158	calculation	calculation	0.00559
160	is not	is not	**0.005520***
164	sensible	sensible	0.005385

B. From PC to Hinkle "Y" (PD)

First Test

	k From Measurements	k From Adjustments
	PC - PD	PCadj. - PD
Q (cfs)	(Δh= 5.29 ft)	(Δh= 2.76 ft)
180	0.0012778	0.009230
181.8	0.012657	**0.009138***
188.3	0.012215	0.008823

Second Test

	k From Measurements	k From Adjustments
	PC - PD	PCadj. - PD
Q (cfs)	(Δh= 6.20 ft)	(Δh= 2.13 ft)
158	0.015759	0.009273
160	0.015562	**0.009122***
164	0.015183	0.008899

* Note that compromise flows and adjusted pressures yield "k" values that are essentially the same as the Corps' values.

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TABLE A-5
HEAD LOSSES FROM HINKLE "Y"
TO ROSEVILLE AND SAN JUAN

1. Roseville; First test

    - Flow: 61.2 cfs

    - Head loss: $PD + \frac{v^2}{2g}$ - Roseville Rapid Mix
      $= 434.51 + 0.35 - 400.70$
      $= 34.16$ ft

    - Head loss factor: $k = \frac{\sqrt{\Delta h}}{Q}$
      $= 0.0955$

2. San Juan

    a. First test

    - Flow: 120.6 cfs

    - Head loss: $PD + \frac{v^2}{2g}$ - San Juan Rapid Mix
      $= 434.51 + 0.35 - 424.00$
      $= 10.86$ ft.

    - Head loss factor: $k = \frac{\sqrt{\Delta h}}{Q}$
      $= 0.027325$

    b. Second test

    - Flow: 160 cfs

    - Head loss: $PD + \frac{v^2}{2g}$ - San Juan Rapid Mix
      $= 441.44 + 0.27 - 424.35$
      $= 17.36$ ft.

    - Head loss factor: $k = \frac{\sqrt{\Delta h}}{Q}$
      $= 0.026041$

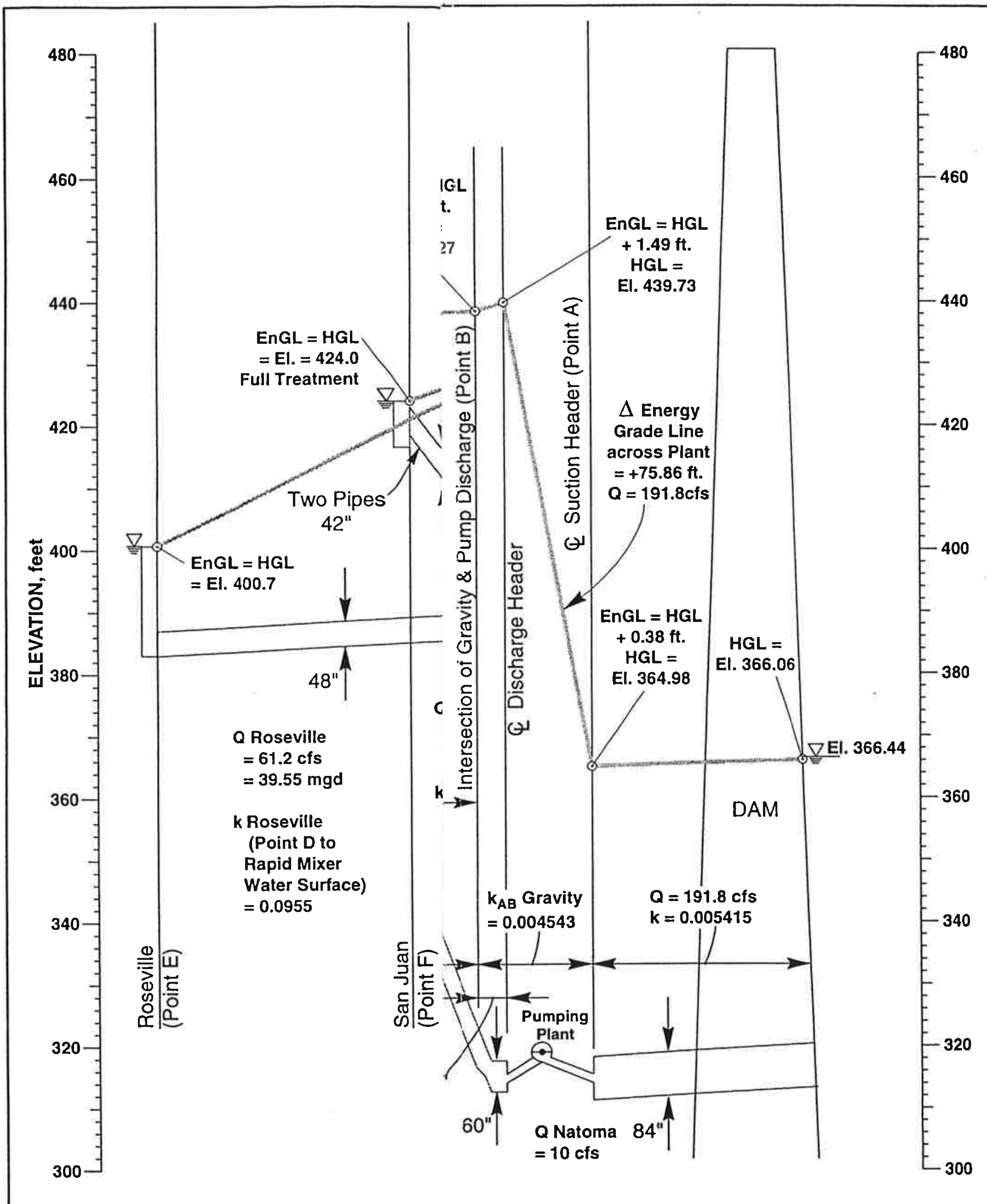
    c. Use $k = 0.0266$

TABLE A-6
HEAD LOSS FACTORS USED

Reservoir to centerline of pumping plant suction header (PA)*:	k = 0.005415
Centerline of pumping plant discharge header to junction of pump discharge line and gravity feed line (PB)*:	k = 0.006657
Centerline of pumping plant suction header to junction of gravity feed line with pump discharge line*:	k = 0.004543
Junction of pump discharge line and gravity feed line (PB) to Hinkle "Y" (PD)*:	k = 0.010658
Junction of pump discharge line and gravity feed line (PB) to stand pipe (PC)*:	k = 0.005514
Hinkle "Y" (PD) to Roseville Rapid Mix**:	k = 0.0955
Hinkle "Y" (PD) to San Juan Rapid Mix**:	k = 0.0266

* Based on Corps 1951 calculations which use the following head loss assumptions:
- Mannings "n" = 0.013
- Gate Valve = $0.19v^2/2g$
- Contraction Loss = 0.1 to $0.5v^2/2g$
- Expansion Loss = 0.2 to $0.5v^2/2g$
- Venturi loss = $0.2(\Delta v^2/2g)$

** Based on November 18, 1994 pumping test



ESA Consultants Inc.
Mountain View, California

FOLSOM DAM - WATER SUPPLY PUMPING CAPACITY
NOVEMBER 1994 PUMPING TEST
FIRST TEST

Checked By Peter V. Jach Date 1/26/96	Project No.	Figure No.
Approved By B Date 1/31/96	043.9501	A-1

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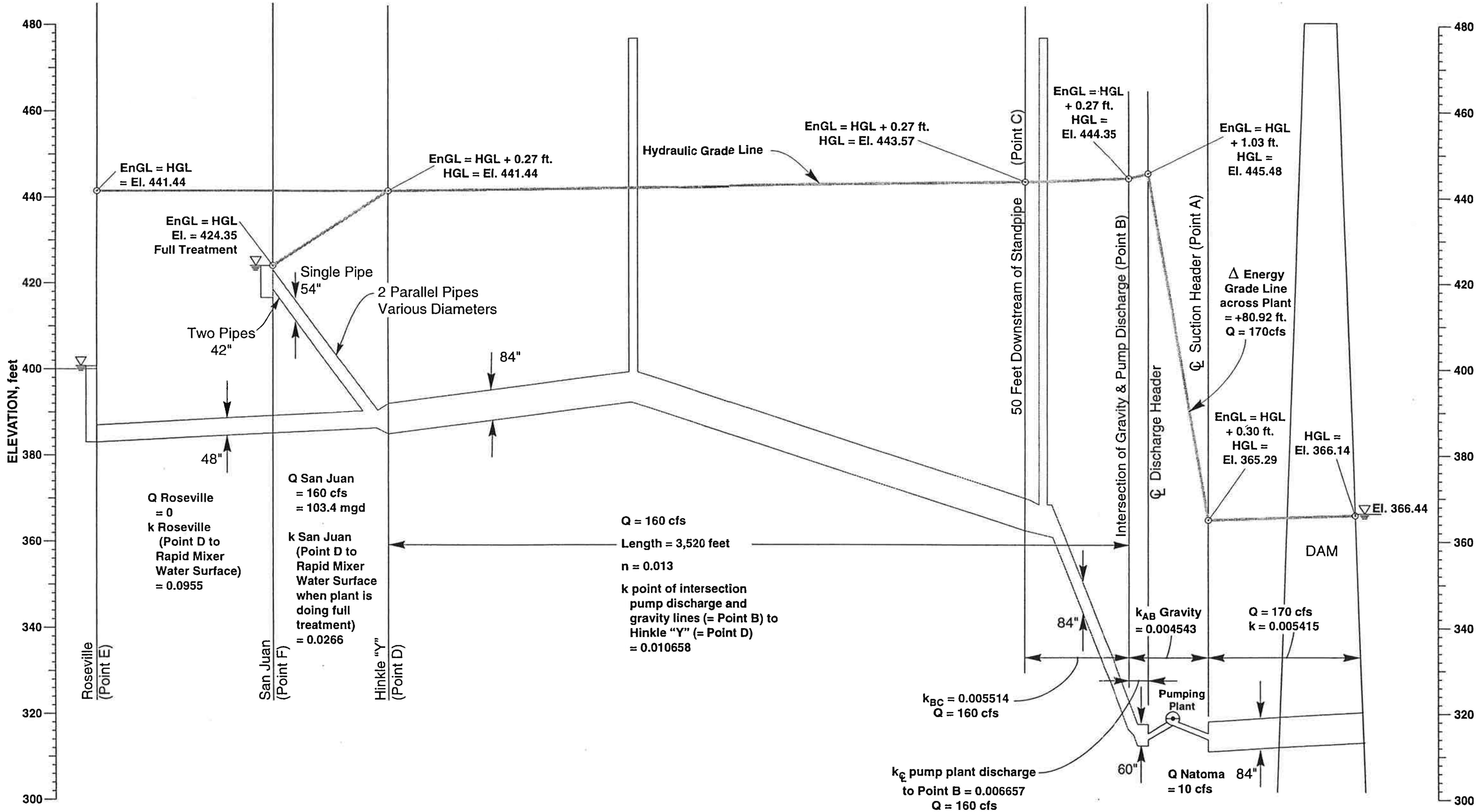


Notes: 1) $\sqrt{\Delta h} = kQ$
2) HGL = Hydraulic Grade Line
3) EnGL = Energy Grade Line
4) Values of HGL Shown here were calculated using Q values from the pumping test and k values for the Corps' 1951 calculations

ESA Consultants Inc. Mountain View, California		
FOLSOM DAM - WATER SUPPLY PUMPING CAPACITY **NOVEMBER 1994 PUMPING TEST** **SECOND TEST**		
Checked By Peter V. Jache Date 1/26/96 Approved By B Date 1/31/96	Project No. 043.9501	Figure No. A-2

APPENDIX A
ATTACHMENT 1

PUMPING TEST DATA
AND PREPARATORY CORRESPONDENCE

File

MURRAY, BURNS AND KIENLEN

A Corporation
1616 29th Street, Suite 300
Sacramento, California 95816
Telephone (916) 456-4400
FAX (916) 456-0253

MEMORANDUM

TO: City of Roseville File — December 2, 1994

FROM: Mark Fortner

SUBJECT: Folsom Dam Water Supply Pump Test on November 18, 1994

Attached are the results of a pump test performed on November 18, 1994. The test was performed to verify the calculations for the report to the City of Roseville, Increasing Peak Water Supply Flows From Folsom Dam. The capacity curve developed in the report included pump #7. Pump #7 has been moved to the penstock tap and therefore the test did not include pump 7.

It should be noted that all the gages used in the test were calibrated with the exception of the Roseville gage. The accuracy of the gages is 1%±. Gage "B" appears to be reading low compared to the other gages and standpipe.

The results show that pump #6 does not provide a large benefit at high heads. The test verifies that the capacity curve developed for the Roseville report is reasonable. Should pump 7 be moved back to the pumphouse, another pump test is recommended.

Mark Fortner

MF:bl
Attachments

FOLSOM DAM WATER SUPPLY SYSTEM
PUMPING TEST

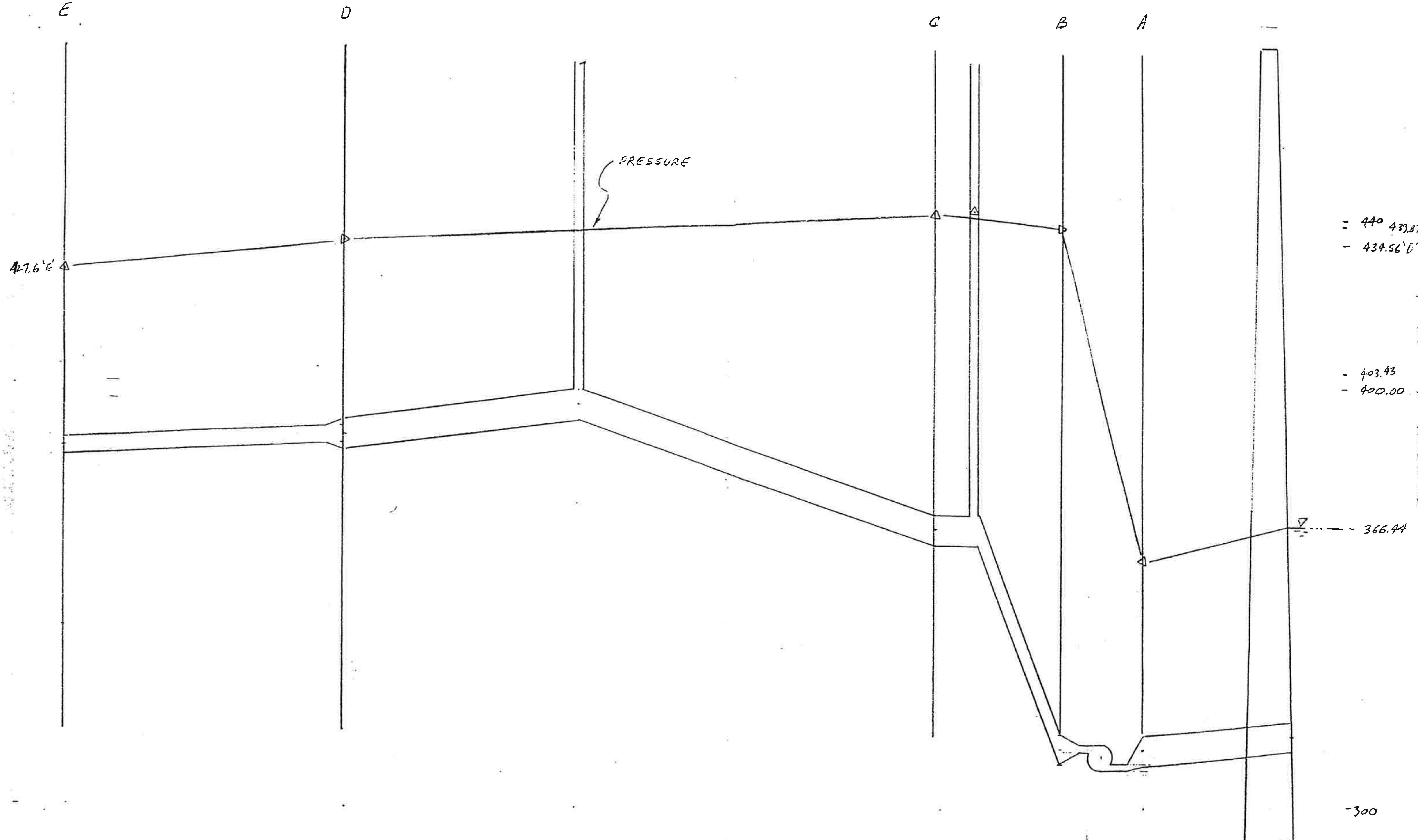
Date: 11/18/94
Lake Level: 366.44
Pumps Running: 2,3,4,5 & 6

PA:El. 322.05
PB:El. 314.52
PC:EL. 365.95
PD:El. 388.36
PE:El. 385.5
PF:El. 404.39

Note: There appears to be a difference in flow reading between San Juan & USBR of around plus or minus 7 cfs. QT may be 190 cfs based on USBR reading of 180 cfs at North Fork ventur~~e~~i.

	Pressures(psi)						Flows(cfs)				
Time	PA	PB	PC	PD	PE (ft of H_2O)	PF	QN Natoma	QS	QR	QT	USBR Stand-pipe (ft ele
10:40	19	53.5	32	20	17.99	9	10	125	62.2	197.2	
10:45	19	53.5	32	20	17.86	9	10	125	63.3	197.3	-8
10:50	19	53.5	32	20	17.95	9	10	125	63.3	197.3	440
10:55	19	53.5	32	20	17.90	9	10	125	63.3	197.3	
11:00	19	53.5	32	20	19.44	9	10	125	63.5	197.5	
-----ROSEVILLE CLOSED-----											
11:10	19	57.0		23	51.92		10	164*	--	174	445/448
11:15	19	57.0	35.4	23	59.28		10	164*	--	174	
*USBR reading of 158 cfs at North Fork Venturi											

QN = Flow to ~~M~~Natomas (City of Folsom, Folsom Prison)
QS = Flow to San Juan SWD
QR = Flow to City of Roseville
QT = Total Flow

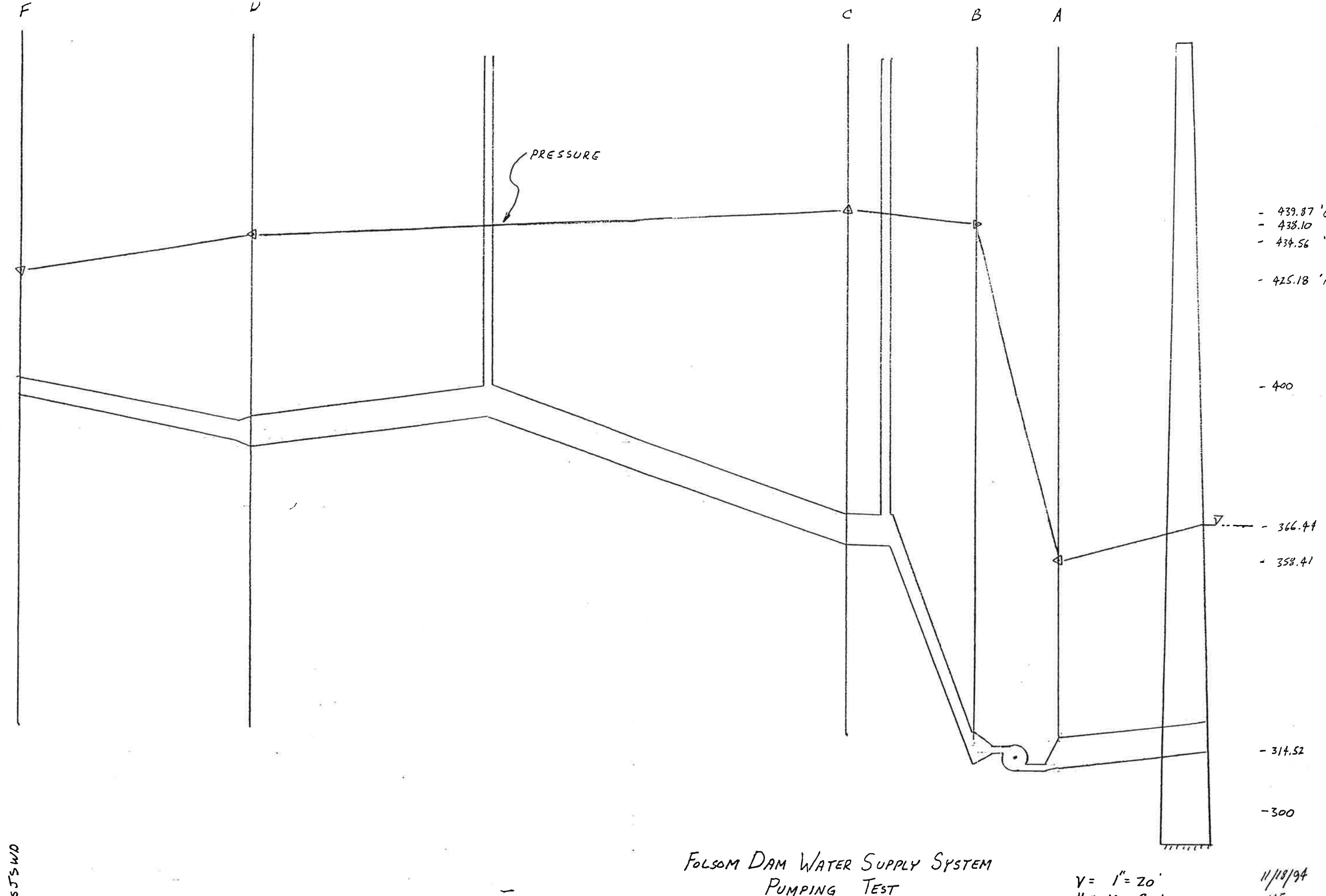


ROSEVILLE

FOLSOM DAM WATER SUPPLY SYSTEM
PUMPING TEST

V = 1" = 20'
H = No Scale

ARWA-202



FOLSOM DAM WATER SUPPLY SYSTEM
PUMPING TEST

V = 1" = 20'
H = No Scale

11/18/94
MF

ARWA-202

Memo to City of Roseville, City of Folsom, November 17, 1994
San Juan Water District, U. S. Bureau of Reclamation
Re: Pump Test of Folsom Dam Water Supply system

City of Roseville

- Pressure at PE, upstream of throttling valve downstream of venturi. Measured in Roseville operation room.
- Flow to Roseville measured in Roseville operation room.
- Level of water in sedimentation basin.

San Juan Water District

- Pressure at PF on single feed line just upstream of treatment plant. This will be measured manually by SJWD personnel.
- Flow to SJWD, measured in SJWD operation room.
- Level of water in sedimentation basin.

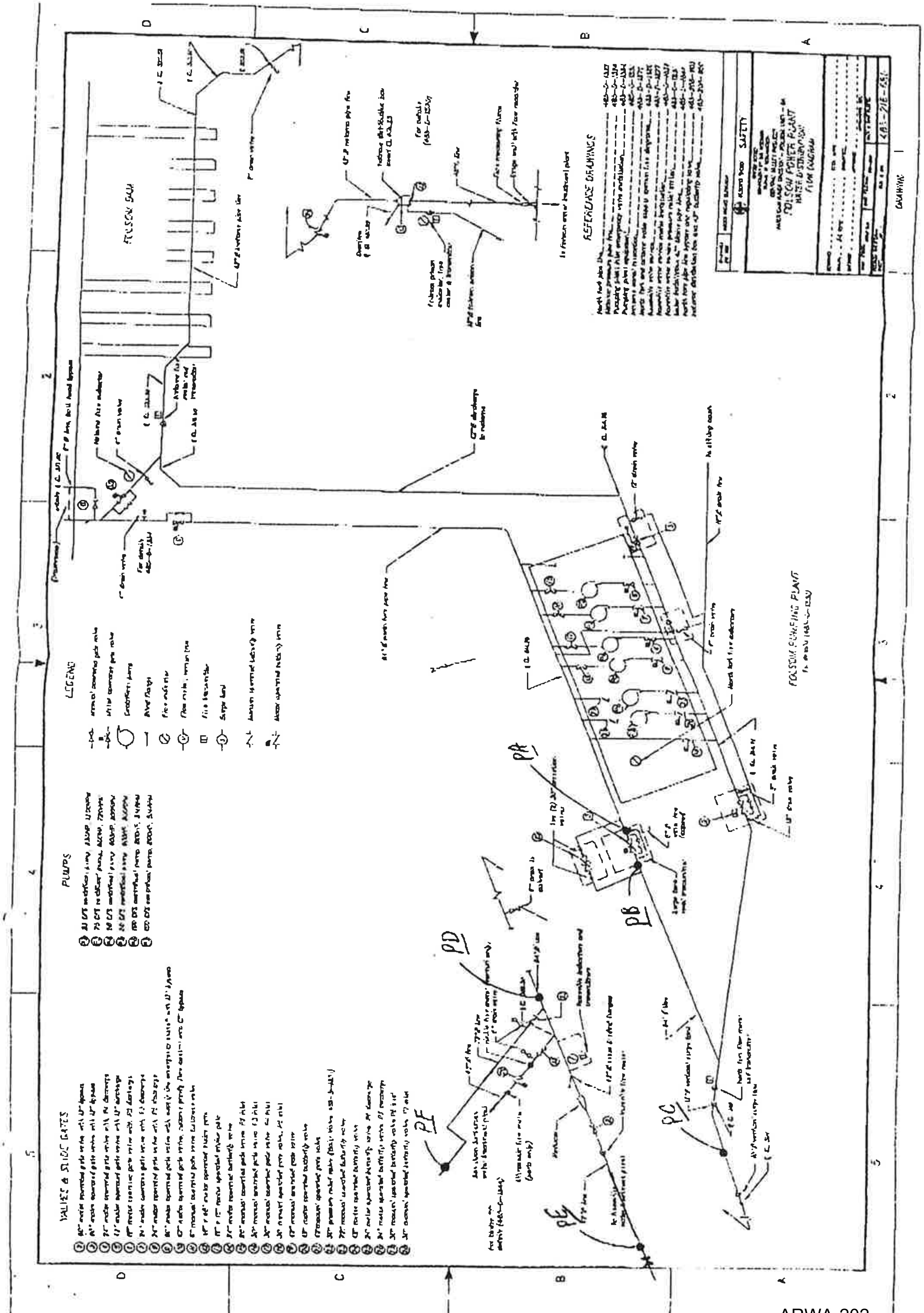
Gages will be in place by Thursday evening. Radios will be checked sometime Thursday. MBK and USBR personnel will meet at 9:30 a.m. on the day of the test to review any last minute items and call operators to synchronize watches.

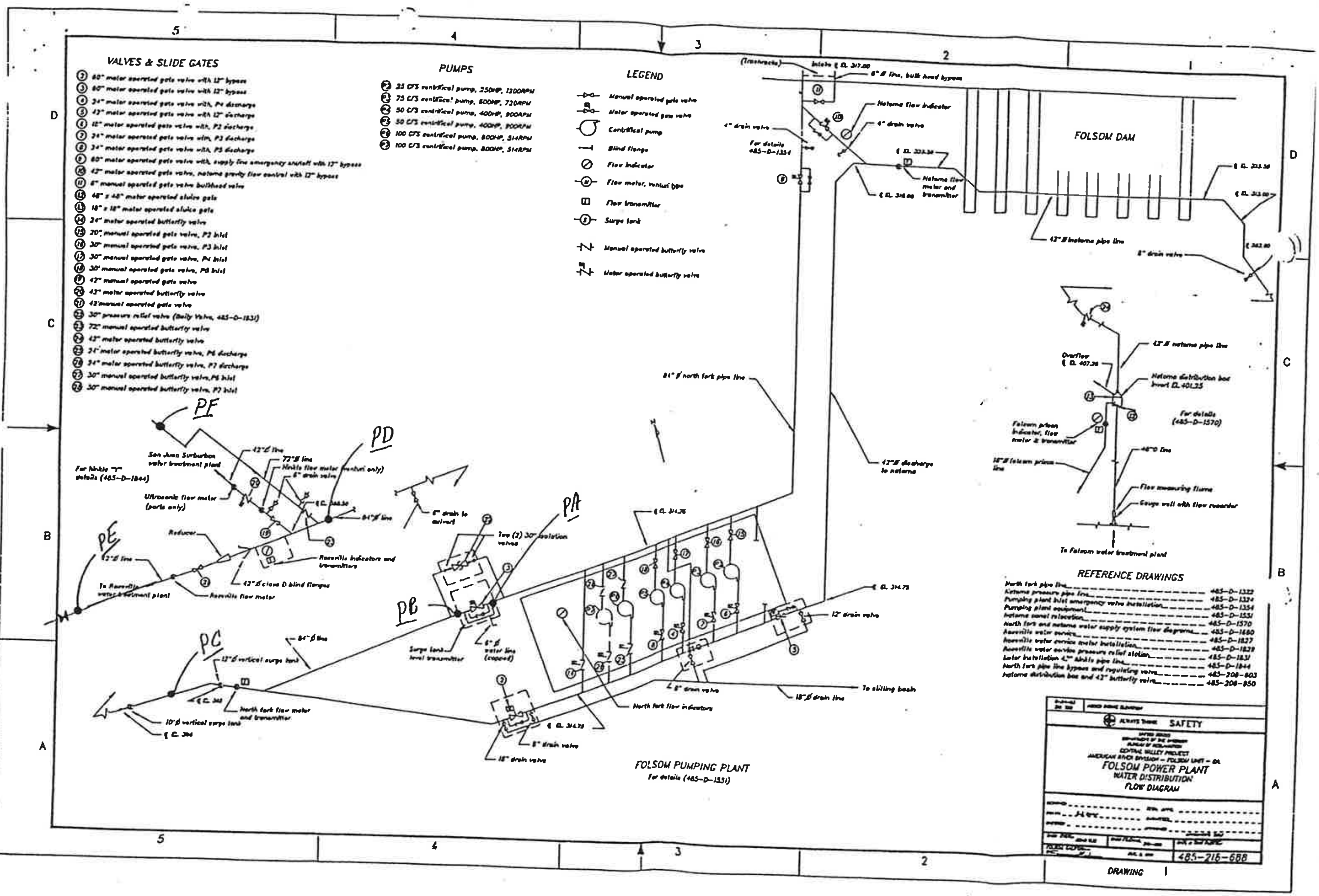
It is proposed that the USBR operation room will verify when a steady state condition has been reached by coordinating with the other operators by telephone. When the steady state condition has been reached, the USBR operator (probably Ed Dempsey) will call down to the pump plant and notify a manual gage reader, who will then relay to other gage readers, by radio, that at time X:XX we will begin reading. The readings will be made at every five minutes (i.e. 10:05, 10:10, 10:15, etc.) for the five readings. Ed will notify the pump plant when the test is complete or if problems arise. Attached is a data collection sheet, and a location map.

Please call if you have any questions or recommendations.

Mark Fortner

MF:bl
Attachments


FOLSOM DAM
REFERENCE DRAWINGS
FOLSOM PUMPING PLANT
LEGEND
PUMPS
VALVES & SLIDE GATES
PA
PB
PC
PD
PE
PF


VALVES & SLIDE GATES
PUMPS
LEGEND
Manual operated gate valve
Motor operated gate valve
Centrifical pump
Blind flange
Flow indicator
Flow meter, venturi type
Flow transmitter
Surge tank
Manual operated butterfly valve
Motor operated butterfly valve
FOLSOM DAM
42" Ø natoma pipe line
81" Ø north fork pipe line
42" Ø discharge to natoma
FOLSOM PUMPING PLANT
For details (485-D-1351)
PA
PB
PC
PD
PE
PF
REFERENCE DRAWINGS
North fork pipe line — 485-D-1322
Natoma pressure pipe line — 485-D-1324
Pumping plant inlet emergency valve installation — 485-D-1354
Pumping plant equipment — 485-D-1351
Natoma canal relocation — 485-D-1551
North fork and natoma water supply system flow diagrams — 485-D-1570
Roseville water service — 485-D-1680
Roseville water service meter installation — 485-D-1827
Roseville water service pressure relief station — 485-D-1829
Meter installation 42" hinkle pipe line — 485-D-1831
North fork pipe line bypass and regulating valve — 485-D-1844
Natoma distribution box and 42" butterfly valve — 485-208-603
485-208-850
ALWAYS THINK SAFETY
CENTRAL VALLEY PROJECT
AMERICAN RIVER DIVISION – FOLSOM UNIT – CA
FOLSOM POWER PLANT
WATER DISTRIBUTION
FLOW DIAGRAM
485-216-688
DRAWING

ARWA-202

FOLSOM DAM WATER SUPPLY SYSTEM
PUMPING TEST

Date: 11-18-94
Lake Level: ______
Pumps Running: ______

PA:El. 322.05
PB:El. 314.52
PC:EL. 365.95
PD:El. 388.36
PE:El. 385.5
PF:El. 404.39

ville
ed valve

	Pressures(psi)						Flows(cfs)			
Time	PA psi	PB	PC	PD	PE	PF	QN	QS	QR	QT
~~[illegible]~~	~~[illegible]~~									
10:40	19									
10:45	19									
10:50	19									
10:55	19									
11:00	19									
11:05	19									
11:10	19									
11:15	19									

5

FOLSOM DAM WATER SUPPLY SYSTEM
PUMPING TEST

Date: 11-18-94
Lake Level: ____________
Pumps Running: ____________
____

PA:El. 322.05
PB:El. 314.52
PC:EL. 365.95
PD:El. 388.36
PE:El. 385.5
PF:El. 404.39

	Pressures(psi)						Flows(cfs)			
Time	PA	PB	PC	PD	PE	PF	QN	QS	QR	QT
1040		53.5								
1045		53.5								
1050		53.5								
1055		53.5								
1100		53.5								
1105		57.0								
1110		57.0								
1115		57.0								

FOLSOM DAM WATER SUPPLY SYSTEM
PUMPING TEST

Date: 11-18-94
Lake Level:
Pumps Running:

ASHCROFT 4" LIQUID FILLED ~~GAGE~~ 0-100 PSI
CALIBRATED 12/17/91
S.N. PG003

PA:El. 322.05
PB:El. 314.52
PC:EL. 365.95
PD:El. 388.36
PE:El. 385.5
PF:El. 404.39

Gary

		Pressures(psi)						Flows(cfs)			
	Time	PA	PB	PC	PD	PE	PF	QN	QS	QR	QT
	10:10:30			21.0							
	10:15:00			27.1							
	10:17:20			35.0							
	10:19:00			31.0							
	10:20:30			34.0							
	10:25:00			34.8							
	10:30:00			33.8							
	10:35:00			32.6							
	10:40:00										
BEGIN TEST	10:40:00			32.2							
MF 1st READ	41:20			32.0							
	45:00			32.0							
	46:20			32.0							
	50:00			32.0	Needle steady						
	51:20			32.0							
	55:00			32.0							
	56:20			32.0							
	11:00:00			32.0							
SVL OFF	01:20			32.2							
	05:00			34.6							
	06:20			35.2							
	10:00			35.5	Needle bouncing ± 02 PSI						
	11:20			35.4							
	15:00			35.8							
END TEST	16:20			35.8							
	20:00			27.0							

FOLSOM DAM WATER SUPPLY SYSTEM
PUMPING TEST

939-7263

791-1715 SJS

Date: 11/18/96

Lake Level: __________

Pumps Running: __________

PA:El. 322.05
PB:El. 314.52
PC:EL. 365.95
PD:El. 388.36
PE:El. 385.5
PF:El. 404.39

Bill, Paul, Gary, Me, Roseville, SJS

Pressures(psi) — Flows(cfs)

	Time	PA	PB	PC	PD	PE	PF	QN	QS	QR	QT
1	10:40				20						
2	10:45				20						
3	10:50				20						
4	10:55				20						
5	11:00				20						
	11:02				21						
	11:04:30				22						
6	11:05				22						
	11:07				23						
7	11:10				23						
8	11:15				23						

ARWA-202

FOLSOM DAM WATER SUPPLY SYSTEM
PUMPING TEST

San Juan Suburban

Date: 11/18/94
Lake Level:
Pumps Running:

PA:El. 322.05
PB:El. 314.52
PC:EL. 365.95
PD:El. 388.36
PE:El. 385.5
PF:El. 404.39

Time	Pressures(psi) PA	PB	PC	PD	PE	PF	Flows(cfs) QN	QS	QR	QT
10:40						9				
10:45						9				
10:50						9				
10:55						9				
11:00						9				

ARWA-202

+E
66.44

FOLSOM DAM WATER SUPPLY SYSTEM
PUMPING TEST

Date: 11-18-94
Lake Level: 366.44
Pumps Running: 2, 3, 4, 5, + 6

Q Natoma 10 cfs

PA:El. 322.05
PB:El. 314.52
PC:EL. 365.95
PD:El. 388.36
PE:El. 385.5
PF:El. 404.39

	Pressures(psi)						Flows(cfs)				
Time	PA	PB	PC	PD	PE	PF	North Fork QN	QS	QR	QT	SP
1040											
1045							180	127	65		440
1050							180				440
1055							180				440
1100							180				440
							180				440
1110							158	164	0		448

ARWA-202

FOLSOM DAM WATER SUPPLY SYSTEM
PUMPING TEST

Date: 11/18/94
Lake Level: ______________
Pumps Running: ______________
____

PA:El. 322.05
PB:El. 314.52
PC:EL. 365.95
PD:El. 388.36
PE:El. 385.5
PF:El. 404.39

San Juan

	Pressures(psi)						Flows(cfs)			
Time	PA	PB	PC	PD	PE	PF	QN	QS	QR	QT
10:40 am								125		
10:45 am								125		
10:50 am								125		
10:55 am								125		
11:00 am								125		
11:10 AM	ROSEVILLE OFF LINE/SAN JUAN @ 100%							164		

CITY OF ROSEVILLE
WATER TREATMENT PLANT
9342 BARTON RD.
ROSEVILLE, CA 95746

TELEPHONE (916) 791-4586 • FAX (916) 791-4671

FAX TRANSMITTAL

DATE 11-21-94

TO COMPANY Murry, Burns & Kremlin FAX NO. 456-0253

ATTENTION Mark Fortner

SUBJECT Attached chart

FROM: NAME Jim Jesse

TOTAL NO. OF PAGES INCLUDING THIS PAGE

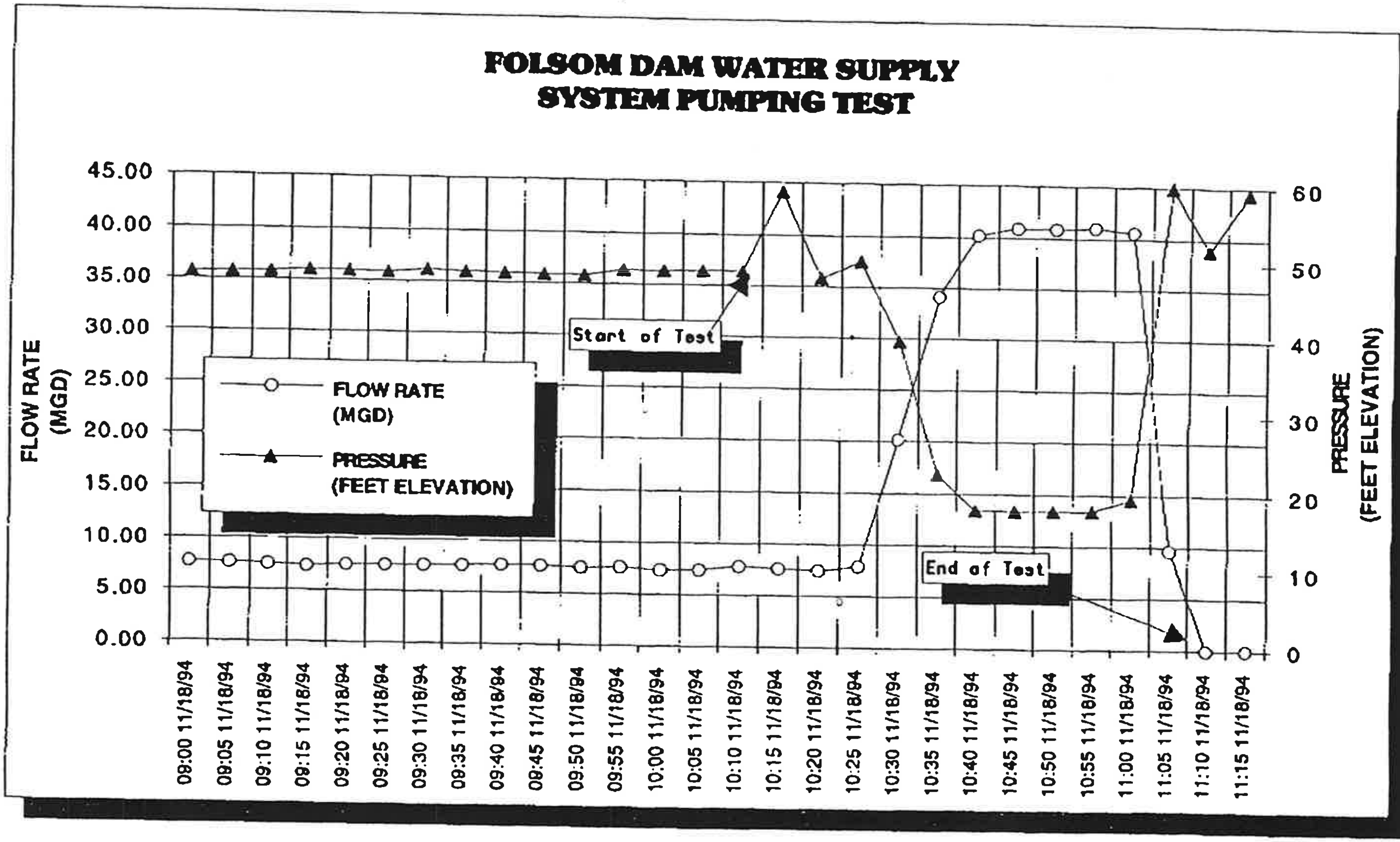
2

MESSAGE:

this chart replaces the one sent to you on Friday. The chart on Friday was labled wrong. throw it away.

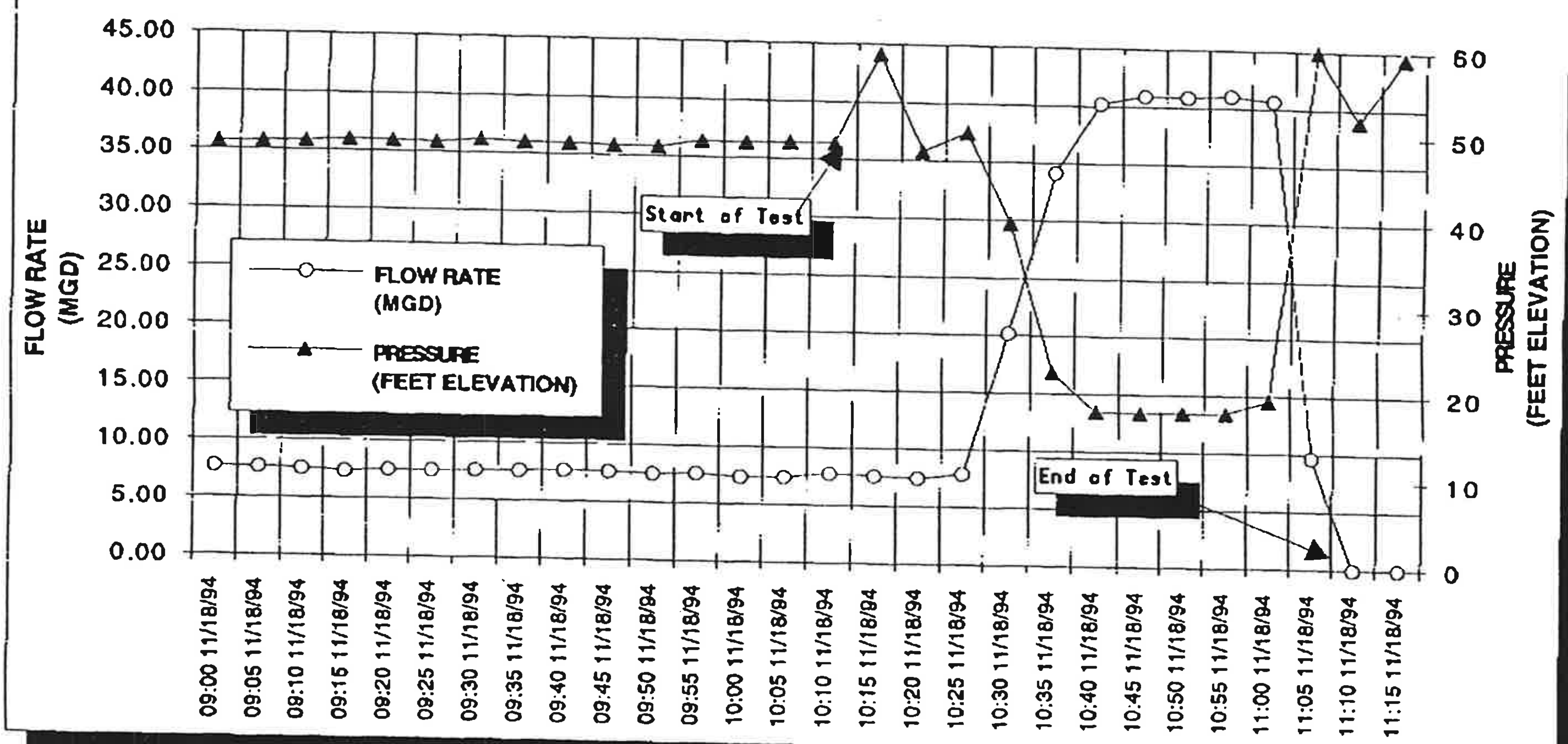
Jim

CITY OF ROSEVILLE WATER TREATMENT PLANT



CITY OF ROSEVILLE WATER TREATMENT PLANT

FOLSOM DAM WATER SUPPLY SYSTEM PUMPING TEST



FOLSOM DAM WATER SUPPLY SYSTEM
PUMPING TEST

DATA FROM CITY OF ROSEVILLE
WATER TREATMENT PLANT

11/18/94

TIME AND DATE	FLOW RATE (MGD)	PRESSURE (Feet Elevation)
09:00 11/18/94	7.67	47.56
09:05 11/18/94	7.65	47.64
09:10 11/18/94	7.53	47.73
09:15 11/18/94	7.34	47.98
09:20 11/18/94	7.51	47.89
09:25 11/18/94	7.51	47.81
09:30 11/18/94	7.55	48.18
09:35 11/18/94	7.61	47.98
09:40 11/18/94	7.69	47.93
09:45 11/18/94	7.65	47.77
09:50 11/18/94	7.51	47.69
09:55 11/18/94	7.59	48.52
10:00 11/18/94	7.34	48.43
10:05 11/18/94	7.40	48.56
10:10 11/18/94	7.76	48.56
10:15 11/18/94	7.61	58.95
10:20 11/18/94	7.47	47.73
10:25 11/18/94	7.88	49.97
10:30 11/18/94	20.17	39.63
10:35 11/18/94	34.11	22.47
10:40 11/18/94	40.23	17.99
10:45 11/18/94	40.93	17.86
10:50 11/18/94	40.89	17.95
10:55 11/18/94	41.02	17.9
11:00 11/18/94	40.63	19.44
11:05 11/18/94	9.70	59.98
11:10 11/18/94	0.00	51.92
11:15 11/18/94	0.00	59.28

$\bar{x}$ = 17.93

MURRAY, BURNS AND KIENLEN
A Corporation
1616 29th Street. Suite 300
Sacramento. California 95816
Telephone (916) 456-4400
FAX (916) 456-0253

MEMORANDUM

TO: City of Roseville, City of Folsom, San Juan Water District, U. S Bureau of Reclamation — November 17, 1994

FROM: Mark Fortner

SUBJECT: Pump Test of Folsom Dam Water Supply System

To clearly define the losses of the Folsom water supply system, a pump test is scheduled for Friday, November 18, 1994, between 10:00 and 10:30 a.m. This test will be with the system operating wide open.

The following data will be collected by the respective entity:

U. S. Bureau of Reclamation

- Pressure at PA upstream of pumping plant (upstream of 60" gate valve), on top of vault. This will be manually read by USBR personnel. See attached drawing.
- Pressure at PB downstream of pumping plant (downstream of 60" gate valve) in vault. This will be manually read by USBR personnel.
- Flow at North Fork flowmeter, measured in USBR operation room.
- Flow to Folsom, measured in USBR operation room.
- Folsom Lake level, measured in USBR operation room.
- Note which pumps are operating.

Murray, Burns and Kienlen

- Pressure at PC, downstream of venturi and surge tank. This will be manually read by MBK personnel.
- Pressure at PD, upstream of Hinkle Wye. This will be manually read by MBK personnel.

FOLSOM DAM WATER SUPPLY SYSTEM
PUMPING TEST

Date:____________________

Lake Level:________________

Pumps Running:____________

____

PA:El. 322.05
PB:El. 314.52
PC:EL. 365.95
PD:El. 388.36
PE:El. 385.5
PF:El. 404.39

	Pressures(psi)						Flows(cfs)			
Time	PA	PB	PC	PD	PE	PF	QN	QS	QR	QT

ARWA-202

DRAFT

APPENDIX A
ATTACHMENT 2

QUESTIONS/ANSWERS REGARDING THE FOLSOM DAM PUMPING TEST DATA

ESA Consultants

APPENDIX A
ATTACHMENT 2

QUESTIONS / ANSWERS REGARDING THE FOLSOM DAM PUMPING TEST DATA

1. Regarding Roseville pressure readings, are they in psi or feet of water?
   Ans: Feet of water.

2. Was Roseville throttling during the first test?
   Ans: No, they were wide open.

3. What was the water surface elevation in Roseville's rapid mix chamber?
   Ans: Assume 400.7 based on plant hydraulic grade line for 41 mgd (63 cfs).

4. Regarding the San Juan pressure readings, where were they taken?
   Ans: In the chemical feed vault, above the pipeline.

5. Was San Juan throttling during the tests?
   Ans: No, they were wide open for both parts of the test.

6. What was the water surface elevation in the San Juan rapid mix chamber?
   Ans: For the first test, assume it was El. 424.0; for the second test assume it was El. 424.35. These numbers are inferred from operator observations of water surface versus Q and assumption of full treatment (including coagulation and sedimentation).

7. Where is Gage C relative to the first stand pipe?
   Ans: Approximately 50 feet downstream (toward Hinkle"Y").

APPENDIX B
COMMENTS RECEIVED ON
THE DRAFT OF THIS REPORT

- **U.S. Bureau of Reclamation**
- **Robert W. Miles for City of Folsom**

ESA Consultants

United States Department of the Interior

BUREAU OF RECLAMATION
North-Central California Area Office
7794 Folsom Dam Road
Folsom, California 95630

IN REPLY REFER TO:

CC-600
PRJ-22

DEC 0 7 1995

RECEIVED
DEC - 8 1995

Mr. Joseph D. Countryman, P.E.
Murray, Burns, and Kienlen
1616 29th Street, Suite 300
Sacramento, California 95816

Subject: Review of Draft Report - Folsom Pumping Plant and Pipeline Flow Enhancement, Central Valley Project

Dear Mr. Countryman: Joe

We have reviewed the draft report "Increasing Water Supply Pumping Capacity at Folsom Dam," dated October 20, 1995, and have the following comments:

1. The head-capacity curves with pump no.6 use the presently installed 450 RPM motor configuration. The 514 RPM motor can be installed to give added capacity at the higher head situations.

2. We agree with your assessment that the variable speed pumps will reduce the throttling and energy waste while providing the future pumping needs.

3. The use of PWM technology for the variable frequency drive (VFD) would be necessary because it would not impact the utility grid system and operation of solid state equipment; therefore, the CSI technology would not be acceptable.

4. The study recommends installing two new pumps which will raise the delivery capacity to 400 cfs at reservoir El. 392 and provide the 150 cfs peak flow necessary for Roseville. Since this is well above the minimum pool reservoir level of El. 327, we concur that the modeling study can be postponed. The intake water surface should be monitored if there are reservoir elevations that approach the minimum pool. If additional large pumps are installed then the modeling study will be required.

5. We concur that we should perform a full review of the emergency closure capability of the 60 inch and 42 inch intake gate valves located in Folsom Dam.

If you have any questions or concerns, contact Bill Joye of my staff at 988-1707 (TDD 989-7285).

Sincerely,

Tom

Thomas J. Aiken
Area Manager

ROBERT W. MILES
CONSULTING CIVIL ENGINEER
RCE 20595

November 9, 1995

Mr. Derrick H. Whitehead
Manager, Environmental Utilities
City of Roseville
1800 Booth Road
Roseville, CA 95747

Subject: Folsom Dam Conveyance Facilities
City of Folsom Comments on Draft Report
File: 3.0110

Dear Mr. Whitehead:

This letter conveys the City of Folsom comments on the draft report entitled "Increasing Water Supply Pumping Capacity at Folsom Dam," dated September, 1995. This report has been distributed for review and comment by the project participants and Reclamation. We appreciate the efforts you, your staff, and your consultants have made to facilitate review of this work.

In general, we've found that the draft report has been well prepared, and we agree with the key finding that the pumping plant should be expanded by installation of two pumps with variable speed drivers. Beyond the selection of the pumps there are several issues that deserve some attention by the project team. We have itemized these points in the following paragraphs.

PUMPING CAPACITY

Table 4-1 of the draft report presents a summary of system flow rate and a distribution of the flows to the respective agencies. To review Table 4-1 we have assembled the information in the enclosed Table A.

Table A contains a summary of the various water contracts, amounts, and flows that the pumping plant may be expected to respond to now and in the future. Some of the information in the table has been estimated, such as the entries for "future" Roseville and Folsom water amounts and flows. The column entitled "Source/Priority" contains the three types of water to be conveyed; water rights water, Central Valley Project (CVP) water, and non-project water. These types of water are listed in our estimated order of priority, with water rights water being the highest priority. The last column contains estimates for the maximum flowrates necessary to convey the annual amounts. Table A shows that conveyance of the water rights water will require about 175 cfs of pumping capacity. Similarly, CVP water will require about 138 cfs, and non-project water will require an additional 173 cfs of pumping capacity.

Table B shows the pumping capacities required when the flowrates from Table A are tabulated and summed in order of priority. As the table shows, the water rights require 175 cfs of capacity. After the first contract for CVP water is added for San Juan WD, the pumping capacity becomes 206 cfs. Similarly, note that the required pumping capacity to implement Roseville's 1989 conveyance contract for non-project water is 438 cfs. For this reason, we believe that the pumping plant should be expanded to a capacity of 438 cfs, not 400 cfs as proposed in the draft report.

PUMPING CRITERIA

In the Summary, on page 1-1, it is proposed that the expanded pumping plant be able to pump 400 cfs at a reservoir elevation of 392 feet. We have reviewed this criterion and have an alternative to propose based upon the above information. Table B can be used to develop the following criteria.

1. The pumping plant should be able to pump the water rights water, 175 cfs, at a minimum pool elevation of 327.

2. The pumping plant should be able to pump the water rights plus CVP water, even during a critical dry year. If we assume that the CVP water is cut back on a flow basis to 75 percent of the contract amounts during a critical dry year, the total of water rights and CVP water would be 279 cfs. A rough estimate for the reservoir elevation during a critical dry year would be 340, which would occur during a repeat of August 1977, according to Table 5-5.

3. The pumping plant should be able to pump a combination of water rights, CVP, and non-project water of 438 cfs during a non-critical dry year at a reservoir pool elevation of about 395. The elevation of 395 would represent the reservoir level in August of a dry year, according to Table 5-5.

4. In the future, the pumping plant should be able to pump all three categories of water, 486 cfs, during a non-critical dry year, probably at a reservoir elevation of about 395, which would represent the reservoir level in August of a dry year.

A copy of Figure 1-1 from the draft report has been marked to show the above criteria. The proposed pumps are slightly undersized to meet the second and third criteria. However, it may not be necessary or desirable to provide complete pumping capacity to meet the above criteria demands immediately because it will take a considerable period of time before they actually develop. In general, pumping plants should be sized for requirements that will occur during a reasonable planning period. Pumping plants that have significant overcapacity tend to present operating problems or operate with less flexibility or efficiency than desired. It may be appropriate to cut back on the capacity of the two proposed pumps slightly and then install a third pump as the pumping requirements development over time.

The fourth criterion should be met with an expansion at some point in the future, as necessary.

STANDBY PUMPING CAPACITY

From Figure 5-12, if either Pump 7 or 8 is out of service the pumping plant cannot meet a criterion of 400 cfs at reservoir elevation 392. A standby pumping unit would be necessary to firm up the capacity.

SPACE FOR FUTURE ELECTRICAL SWITCHGEAR AND CONTROLS

Has space for future electrical equipment been designated? This should be done to avoid limitations for future expansions.

CONSTRUCTION PLAN

The sequence of construction activities should be evaluated to confirm that the pumping plant can be modified with reasonable lengths of downtime and disruption to the plant operations.

SCHEDULE

A schedule should be established for the project. It may be necessary to pre-order critical equipment.

COST ESTIMATE

Modification-type projects should have a contingency greater than 15 percent at this stage of project development. A more suitable value would be 25 percent. As the design develops in the next phase, the contingency can be reduced appropriately.

REVIEW AND DISCUSSION

Gordon Tornberg and I will be available to review these issues with you and the project team as required to promote a complete understanding of all the factors in this project.

Sincerely,

Robert W. Miles

Robert W. Miles

cc: Mr. Gordon F. Tornberg
Mr. Joseph D. Countryman

TABLE A
FOLSOM DAM CONVEYANCE FACILITIES
Water Contracts, Amounts, and Flows

SOURCE/ PRIORITY	AGENCY	YEAR OF CONTRACT	AMOUNT, acre-feet/year	FLOW RATE, cfs	CALCU-LATED FLOW RATE, cfs
Water Rights	Folsom	1971	22,000	--	61
	Folsom	1971	5,000	--	14
	San Juan WD	1954	33,000	75	91
	Prison	1958	4,000	9	9
Subtotals			59,000		175
CVP	San Juan WD	1962	11,200	--	31
	Roseville	1967	32,000	65/150	88
	Folsom (Fazio)	--	7,000	--	19
Subtotals			50,200		138
Non-Project	San Juan WD	1972	25,000	--	69
	Roseville	1989	20,000	--	56
	Roseville (Future)	--	10,000	--	28
	Folsom (Future)	--	7,200	--	20
Subtotals			62,200		173

TABLE B
FOLSOM DAM CONVEYANCE FACILITIES
Order of Priority for Pumping Capacity

SOURCE/ PRIORITY	PUMPING CAPACITY, cfs	San Juan WD, cfs	Roseville cfs	Folsom cfs	Prison cfs
Water Rights	175	91		75	9
CVP	206	31			
	294		88		
	313			19	
Non-Project	382	69			
	438		56		
	458			20	
	486		28		
Totals	486	191	172	114	9

KRIZ

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FINAL REPORT

FOLSOM PUMPING PLANT
SYSTEM CAPACITY EVALUATION
(Task Order 06A1204097M, Contract 06CS204097M)

PREPARED FOR

US BUREAU OF RECLAMATION
CENTRAL CALIFORNIA AREA OFFICE
FOLSOM, CALIFORNIA

PREPARED BY

WRE Water Resources Engineering, Inc.

July 2011

ARWA-202

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PREPARED BY
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Water Resources Engineering, Inc.'s work on this report was performed by Principal Engineer Gustavo Arboleda, PE, and Staff Engineer Cynthia Cano. To the best of our knowledge, the data contained herein are true and accurate and satisfy the scope of work for the Task 5 deliverable under Task Order 06A1204097M. The data, findings, recommendations, specifications, or professional opinions were prepared solely for the use of our client in accordance with generally accepted professional engineering practice. We make no other warranty, either expressed or implied, and are not responsible for the interpretation by others of the contents herein.

FOLSOM PUMPING PLANT
SYSTEM CAPACITY EVALUATION

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EXECUTIVE SUMMARY

This report presents the results of studies performed to evaluate system capacity as well as operational and energy usage issues associated with the Folsom Pumping Plant raw water delivery system. Studies included:

- Evaluation of hydraulic performance
- Evaluation of the effects of higher demands on the water delivery system
- Evaluation of power and control systems
- Identification of possible corrective measures and their costs

Evaluation of Hydraulic Performance

The evaluation identified:

- *Physical deficiencies:*
  - ❖ The geometry of pump intakes at the Folsom Pumping Plant generates adverse approach flow conditions (swirl and skewed flow distributions) that result in a phenomenon known as "recirculation," characterized by loud crackling sounds around the pump suction and/or discharge. Suction and discharge recirculation can be very damaging to pump operation and should be avoided for continuous operation.[1]
  - ❖ Four of seven valves on pumping plant discharge piping (after pumps numbered 2, 3, 4, and 5) are gate valves not suitable for partially-open operation (i.e., throttling); the other three valves (after pumps numbered 6, 7, and 8) are of the butterfly type and can be safely used in a partially-open position. Valve throttling can be necessary at times to control pump head (i.e., lift) and keep pumps within the range of heads recommended by the manufacturer for safe and efficient operation.
  - ❖ Five of the seven pumps at the pumping plant (pumps numbered 2, 3, 4, 5, and 6) are of the constant-speed type; that is, the motors that drive the pumps maintain a steady rate of revolutions per minute (rpm) from no load to full load (the other two pumps, 7 and 8, have variable frequency drives that allow them to perform efficiently at different rpm). The constant speed pumps at the plant were designed to generate lifts ranging from 84 feet (pumps 4, 5, and 6) to 100 feet (pumps 2 and 3) when running at peak efficiency (as shown in Table 3-3 of this report). At the minimum operating efficiency recommended by manufacturers (as shown in Table 3-4 of this report), the pumps are designed to generate lifts ranging from 50 feet (pumps 4, 5, and 6) to about 60 feet (pumps 2 and 3). Available heads during periods of pump operation at the Folsom Pumping Plant are frequently under 50 feet; when constant speed pumps are operated continuously below the efficiency levels recommended by manufacturers, they are likely to develop

[1] Karassik, I. J. et al, "Pump Handbook," Second Edition, McGraw-Hill, New York, 1986, pp. 2.267. The handbook indicates that the cavitation damage produced by discharge recirculation is generally invisible from the suction side, as it occurs on the underside of the impeller vanes; if discharge recirculation is occurring, this might explain why impeller damage has not been detected during pump impeller inspections at the Folsom Pumping Plant.

premature wear of the impeller vane tips, failure of the pump mechanical seal and bearings, and under extreme conditions breaking of the impeller shaft.

- *Operational deficiencies*:
  - ❖ Current operating procedures do not take into account the characteristics (i.e., "pump curves") of the constant speed pumps and their acceptable operating ranges, resulting in operation of the pumps well outside of manufacturer-recommended ranges. When operated outside manufacturer-recommended ranges, the pumps can suffer damage and may deliver less water than indicated by pump performance curves.
  - ❖ Discharge valves are slowly brought to full open position after pump startup without regard to operating pumps at the manufacturer-recommended total dynamic head (TDH). As previously indicated, operating pumps at lower-than-recommended heads can be detrimental to the pumps and generate less-than-expected flow rates.
  - ❖ The two pumps with variable frequency drives or "VFDs" (pumps numbered 7 and 8) are only operated between 50 and 75 percent of full speed; this constraint was imposed by operators based on observed deficient operation (noise and vibration) outside of this range of speeds. The VFD pumps are operated with the discharge valve fully open, although the pump operation training document prepared by Will Betchart in March 2000 indicates that valves should be throttled to control pump head. Valve throttling would likely allow operation of VFD pumps through their normal operating range, which is generally from about 30 to 100 percent of full speed.
- *Operational limits:*
  - ❖ Gravity flows - The raw water delivery system is capable of satisfying current and anticipated future demands by gravity when the reservoir water level is high enough. Based on raw water demand and reservoir level data provided by Reclamation for the years 2000 to 2007, the water level in the reservoir was high enough to allow deliveries by gravity to the North Fork Pipeline about 28 percent of the time.
  - ❖ Pumped flows - The 7 pumps in the Folsom Pumping Plant have a combined capacity of 404 cubic feet per second (cfs) when operated at peak efficiency. Current typical summer demands are approximately 309 cfs (see Table 2-1 of this report). Maximum current demands based on treatment capacities at the end points are about 361 cfs; maximum future demands based on anticipated treatment plant expansions would be 474 cfs (see Table 2-2 of this report). If the pumps were operated at peak efficiency, they would be able to meet current typical and maximum demands; they would not be able, however, to meet future maximum demands. Operation of the pumps at peak efficiency would generally require valve throttling: by partially closing the discharge valves, additional head would artificially be created that would bring the pumps to their most efficient operating level.

Effects of Higher Demands

Increased delivery volumes would:

- Raise the gravity flow threshold for both the North Fork and Natoma pipelines, thereby increasing the need to pump.

- Cause the pumping plant's constant speed pumps to be used more frequently, as demands would exceed the capacity of the variable speed pumps more often than now.
- Substantially increase power consumption; energy usage would more than double for a 25 percent increase in water demand.
- Require increased pump maintenance by accelerating the degradation of equipment that is already operating at low efficiencies under adverse hydraulic conditions.

Evaluation of Power and Control Systems

The plant's power supply could be upgraded as follows:

- Modernizing plant switchgear and using microprocessor-based, multi-functional relays would significantly improve the reliability of power supply to the pumping plant.
- New cable feeders from Switchgear UHA to pumping plant main switchgear would improve overall system reliability at a relatively moderate cost and with little disruption to plant operations.
- The configuration of the pumping plant's main switchgear 1 and switchgear 2 should be changed to provide redundancy and improve power supply reliability; under the existing system configuration, pumps would lose power upon failure of breakers or interconnecting cables.

The plant's controls appear to have adequate reliability. Since pump 7 and 8 share a control power supply, however, a malfunction or even a blown fuse can cause loss of control power to both pumps. To improve reliability, a separate power supply should be provided for each pump control circuit.

Corrective Measures and Their Cost

The following five corrective actions were identified that could be implemented individually or in various combinations:

- *Development and adoption of new Standard Operating Procedures (SOP):* The current SOP could be revised to operate pumps at their proper TDH; this would require throttling discharge valves on pumps 6, 7, and 8 as necessary, and operating constant speed pumps only when the TDH is within acceptable ranges, since existing gate valves would not allow throttling. If new valves and/or pumps are to be installed, development of a new SOP should be delayed to incorporate details of the operation of the new equipment.

  Costs: In-house preparation assumed, no external costs, and no equipment purchase involved.
- *Installation of butterfly valves on pump discharge pipes that lack them:* The new valves would include automated controls to operate pumps within acceptable TDH ranges. The existing SOP would have to be revised upon valve installation.

  Costs: Five new butterfly valves of appropriate sizes would cost approximately $315,000, including automated controls.

- *Power supply upgrades:* Could range from replacement of power cables to installation of new, modern switchgear with microprocessor-based, multi-functional relays. Improving the reliability of plant's main switchgear 1 and switchgear 2 is recommended to provide redundancy of power supply to the pumps.

  Costs: Cable replacement could be done for about $200,000; switchgear UHA could be upgraded for about $600,000. Refurbishing the plant's main switchgear would cost around $2M.

- *Installation of new variable speed pumps:* Options to install three, four, or five pumps were assessed. New valves are assumed with the new pumps. Power supply upgrades would be necessary as well, as the new pumps would increase the total power demand at the plant. A new SOP would be needed.

  Costs: New pumps, valves, and associated controls would cost from $5.1M dollars (for three pumps) to $8.4M dollars (for five pumps). A major overhaul of the power supply system would require an additional expenditure of about $2M dollars.

- *Pump intake reconfiguration:* A new intake configuration would improve the efficiency of the existing pumps only if they are operated within acceptable ranges; new discharge valves and a new SOP would be required along with the intake reconfiguration; pumps could remain as they are, and minor power supply upgrades would suffice.

  Costs: A physical model study (approximate cost $150,000 to $200,000) is recommended to design the reconfiguration of the pump intakes. Cost of the reconfiguration would depend on the design developed through the model tests. Minor modifications to the intake piping could cost under $1M. Major restructuring of the pumping plant intake, if required, could cost upwards of $5M.

1 INTRODUCTION

1.1 Background

The Bureau of Reclamation ("USBR" or "Reclamation") operates a pumping plant and several pipelines that supply water from Folsom Reservoir to the City of Folsom, Folsom Prison, the City of Roseville, and the San Juan Water District (SJWD). Projected increases in water demand will increase the burden on pumps, pipes, and power supplies, with possible adverse effects on system performance and maintenance needs.

The pumping plant and pipelines have experienced some operational problems at current delivery volumes. Standpipes have been overtopped a few times. Variable speed pumps are not operational through their full range. Constant speed pumps exhibit noises typically associated with cavitation (the rapid formation and collapse of bubbles), which can damage pumps and shorten their useful life.

The power supply to the pumping plant lacks redundancy, which could result in a halving of the pumping plant capacity if one of its two power sources were lost. In that case, only four of the eight pumps in the plant would remain operational until an alternate power source could be brought on line.

Energy usage is impacted by reservoir water levels and other factors: operation of pumps at low efficiencies, for example, increases power requirements; the settings in the programmable logic controller at the pumping plant affect pump performance and energy consumption; pump selection can also affect power consumption.

1.2 Scope

This report presents the results of studies performed to evaluate system capacity as well as operational and energy usage issues associated with the Folsom Pumping Plant and water delivery pipelines. The scope of the studies included:

- Collection and analysis of system configuration and operational data.
- Evaluation of the hydraulic performance of the pumping system (pumps, pipes, valves, fittings, surge tanks), including an assessment of variable frequency drive (VFD) operation.
- Evaluation of the potential impacts that sustained deliveries at higher-than-current volumes would have on system components.
- Evaluation of power supply and control systems.
- Development of recommended changes to the pumping plant and their estimated costs.

Technical memoranda were prepared at the end of each project phase to summarize results of evaluations of hydraulic performance, power/control systems reliability, and impacts of increased demands. This final report consolidates project findings, conclusions, and recommendations.

1.3 Objectives

The objectives of the tasks addressed in this report were to:

- Identify physical and operational deficiencies in the pumping system at current delivery volumes and recommend corrective measures.
- Assess the impacts of higher delivery volumes on system components and plant operations.
- Define current operational limits of pumping plant, pipelines, and electrical system.

1.4 Changes

There were two significant changes to the water delivery system since this project was started in September 2007:

- A new pipeline, along with a surge protection standpipe and associated valves, was added in 2010 as part of the Raw Water Bypass Pipeline Project.
- Parts of the Natoma Pipeline were reconfigured and re-aligned in 2010-2011.

The effects of these changes, if any, on hydraulic calculations and performance evaluations are noted where appropriate.

1.5 Report Organization

The remainder of this report is organized as follows:

- *Section 2 – System Configuration and Operational Data*
    - ❖ System description based on available drawings and field inspections
    - ❖ Operational data based on available documentation and interviews with operators and water customers
- *Section 3 - Hydraulic Performance Evaluation*
    - ❖ Basis of hydraulic performance evaluation
    - ❖ Ability to satisfy demands through gravity flows
    - ❖ Ability to satisfy demands through pumping
    - ❖ Conclusions about hydraulic performance
- *Section 4 – Evaluation of the Effects of Higher Demands*
    - ❖ Basis of evaluation
    - ❖ Impact of higher flow velocities
    - ❖ Raised gravity flow thresholds
    - ❖ Increased frequency of pump use
    - ❖ Increased power consumption
    - ❖ Conclusions about impacts of higher demands

- *Section 5 – Evaluation of Power and Control Systems*
  - ❖ Basis of electrical and control systems evaluation
  - ❖ Configuration of electrical and control systems
  - ❖ Reliability of electrical and control systems
  - ❖ Conclusions about power and control system reliability
- *Section 6 – Recommended Actions and Their Approximate Costs*
  - ❖ Development and adoption of new SOP
  - ❖ Discharge valve replacement
  - ❖ Upgrades of power supply
  - ❖ Pump replacement
  - ❖ Pump intake reconfiguration
  - ❖ Combinations of recommended actions
  - ❖ Impacts of recommended actions
- *Appendices*
  - ❖ References
  - ❖ Field tests
  - ❖ Hydraulic model
  - ❖ Pump curves
  - ❖ Referenced electrical/control drawings
  - ❖ Proposed pump selection schedule
  - ❖ Reclamation comments on first draft of report and WRE responses

1.6 Units and Datum

Flow rates and pressures can be reported in a variety of units. This report uses cubic feet per second (cfs) for flow rates and "feet of head" to indicate the height of the water column in pipelines or the lift provided by pumps. Commonly used conversions are listed below:

- 1 cfs = 448.8 gallons per minute (gpm) = 0.65 million gallons per day (MGD)
- 1 MGD = 1.55 cfs
- 1 foot of head = 0.43 pounds per square inch (psi)
- 1 psi = 2.31 feet of head

Elevations are reported in feet, and are referenced to the North American Vertical Datum of 1988 (NAVD88), unless otherwise noted. NAVD88 is the datum used by the California Data Exchange Center to report water surface elevations in reservoirs, including Folsom Reservoir. Some Reclamation drawings referenced in this document use the National Geodetic Vertical Datum of 1929 (NGVD29) which was used prior to the 1980s and is also referred to as the Mean Sea Level datum. NAVD88 and NGVD29 are related as follows at the Folsom Dam:

NGVD29 elevation + 2.34 feet = NAVD88 elevation

2 SYSTEM CONFIGURATION AND OPERATIONAL DATA

This section of the report describes the water delivery system associated with the Folsom Pumping Plant and outlines operations data. The water delivery system is broken down into the following components:

- Water source
- Water transmission pipelines
- Pumps
- Appurtenances
- Flow control valves
- Electrical power supply
- Control system

Operational data for the water delivery system include:

- Current and future water demands
- Water surface elevations that impact water deliveries
- VFD and constant speed pump operation

2.1 System Description

The water delivery system associated with the Folsom Pumping Plant is considered a "municipal and industrial" (M&I) system. That designation indicates that the system delivers untreated (raw) water to end users.

The conveyance of raw water from the Folsom Reservoir to four end users (SJWD, City of Roseville, Folsom Prison, and City of Folsom) requires a complex system of pipes, valves, flow meters, surge protection towers, and electric-motor-driven pumps. The approximate alignment of pipelines and locations of system end points are shown in Figure 2-1. A flow diagram for the raw water delivery system is presented in Figure 2-2. System components are described below.

2.1.1 Water Source

Folsom Reservoir is the water source for the raw water delivery system associated with the Folsom Pumping Plant. Folsom Dam regulates runoff from about 1,875 square miles of drainage area. The reservoir has a normal full-pool storage capacity of 975,000 acre-feet with a minimum seasonally designated flood control storage space of 400,000 acre-feet. Roughly 100,000 acre-feet of raw water are delivered annually to the larger customers, SJWD and the City of Roseville. About 40,000 acre-feet are delivered to the City of Folsom and Folsom Prison per year.

Figure 2-1 Raw Water Delivery System Layout

* North Fork Pipelines include an above-ground 84-inch diameter pipe and an underground 72-inch diameter pipe that extend in parallel from a point roughly 100 feet downstream of the Folsom Pumping Plant to the Hinkle Y

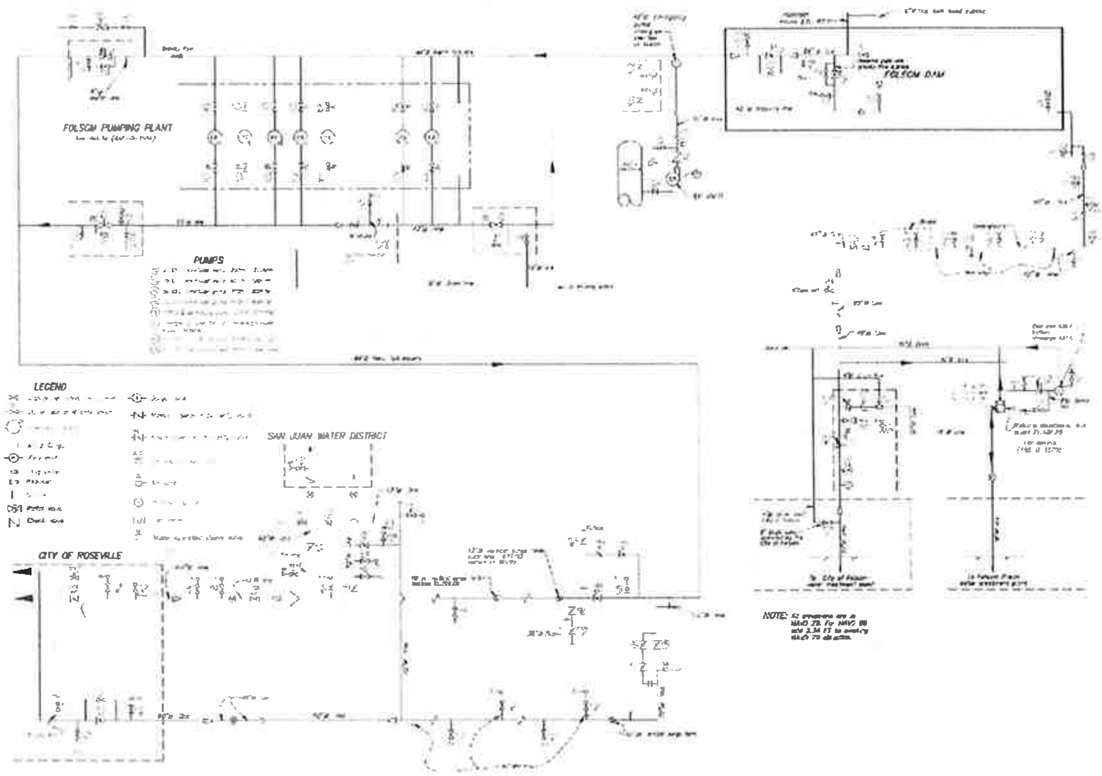

Figure 2-2 Folsom Pumping Plant Water Distribution Flow Diagram
Source: Drawing 485-218-688, Folsom Pumping Plant Water Distribution Flow Diagram, USBR, August 8, 1991, Last Revision November 4, 2010

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Water surface elevation in the reservoir has fluctuated between 366.8 (winter 2008) and 465.4 (spring 2005) feet since 2001, as shown in Figure 2-3. When the reservoir water level is high, typically in the springtime, water can flow by gravity to the four end points. The threshold at which deliveries can be made by gravity depends on the total system demand (i.e., the higher the demand, the higher the reservoir water level needs to be).

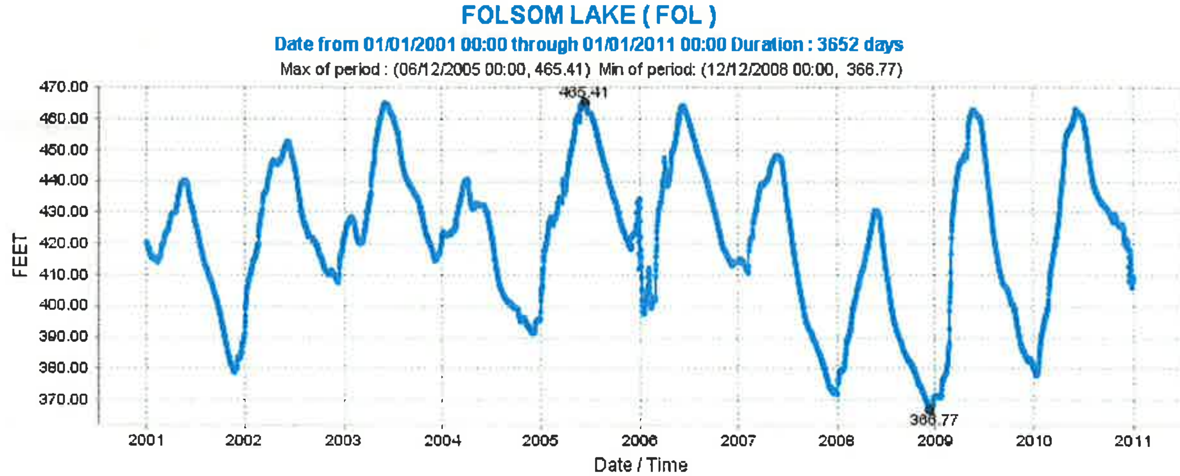

Figure 2-3 Folsom Reservoir Water Levels 2001-2010
Source: California Data Exchange Center

2.1.2 Water Transmission

Water is conveyed from Folsom Reservoir to four end users through four pipelines:

- *The North Fork Pipelines:* The original pipeline is an 84-inch-diameter above-ground steel pipe with two surge-protection standpipes (Figure 2-4). The pipeline originates at the Folsom Dam intake structure and extends about 4,000 feet above ground to the "Hinkle Y." A parallel 72-inch-diameter underground steel pipeline was added in 2010 (Figure 2-5) to provide redundancy in case the original pipeline failed or needed maintenance. The North Fork Pipelines can deliver gravity or pumped flows to the SJWD and City of Roseville pipelines, which originate at the Hinkle Y.
- *SJWD Pipelines:* Two above-ground parallel steel pipes, 42 and 72 inches in diameter, originate at the Hinkle Y. The respective diameters change to 54 and 66 inches about 850 feet downstream of the Hinkle Y, at the location of crossover valves that interconnect the two pipes. About 750 feet further downstream, the two pipes combine into a single 54-inch-diameter pipe that conveys gravity and pumped flows to the SJWD Water Treatment Plant
- *City of Roseville Pipelines:* Two underground pipelines, 48 and 60 inches in diameter, deliver Folsom Reservoir water to the City of Roseville's water treatment plant. The two pipelines originate from a 60-inch-diameter pipeline that extends 434 feet from Reclamation's metering facility at the Hinkle Y toward the Auburn Folsom Road. The 48- and 60-inch-diameter lines are roughly 9,000 feet long each.

Figure 2-4 North Fork Pipeline and Main Standpipe, Looking East (Picture on Left, with "Old" [2009] Natoma Pipeline in Background), and 84-inch North Fork Pipeline and 10-foot-diameter Standpipe, Looking West Toward Hinkle Y (Picture on Right)

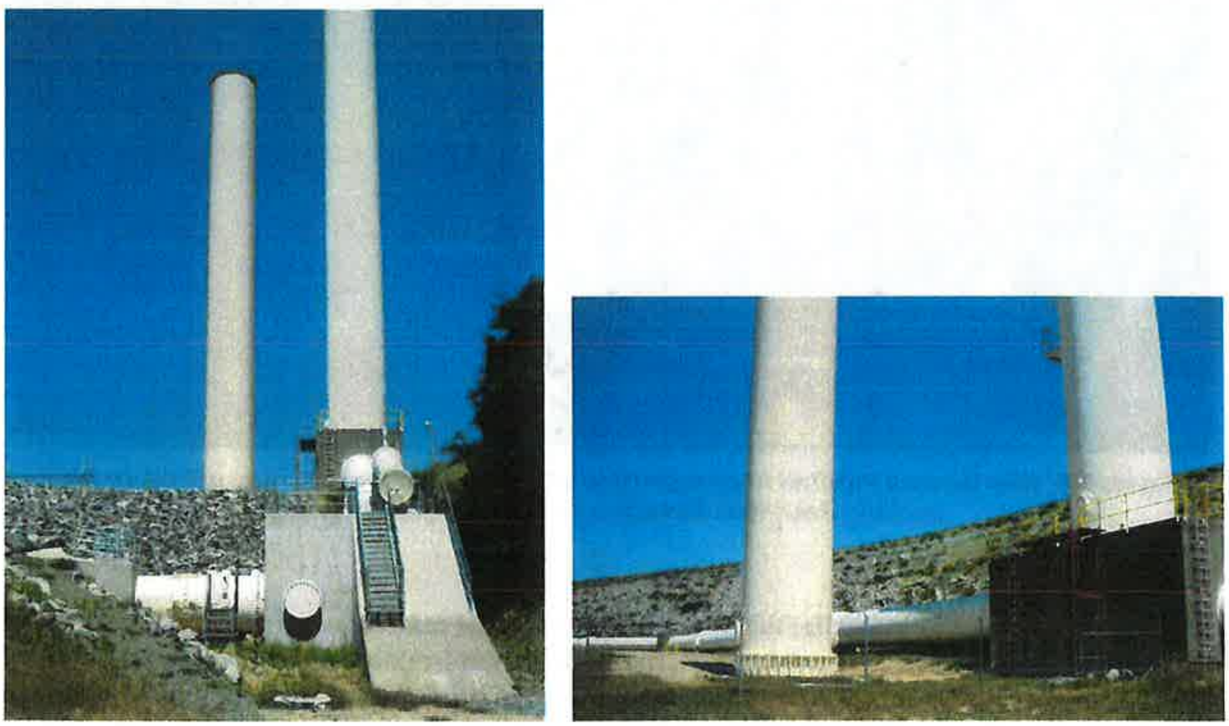

Figure 2-5 Connection to New 72-inch-diameter Pipe (Picture on Left); Standpipes on Above-ground and Underground Pipes (Picture on Right)

- *The Natoma Pipeline:* This 42-inch-diameter steel pipe branches off the North Fork Pipeline roughly 50 feet downstream of the Folsom Dam raw water intake. The pipeline is also connected to the Folsom Pumping Plant discharge manifold, which allows it to convey pumped flows to the City of Folsom and Folsom Prison water treatment plants. A construction project initiated by the City of Folsom replaced parts of the Natoma Pipeline with new 48- and 60-inch-diameter pipes (Figure 2-6). This project included the addition of a new 18-inch-diameter pipe originating at the Natoma Pipeline isolation valve structure and extending to Folsom Prison's water treatment plant; the project also included replacement of the pipeline's surge protection standpipe with a new 10-foot diameter standpipe with an overflow elevation of 436 feet.[2]

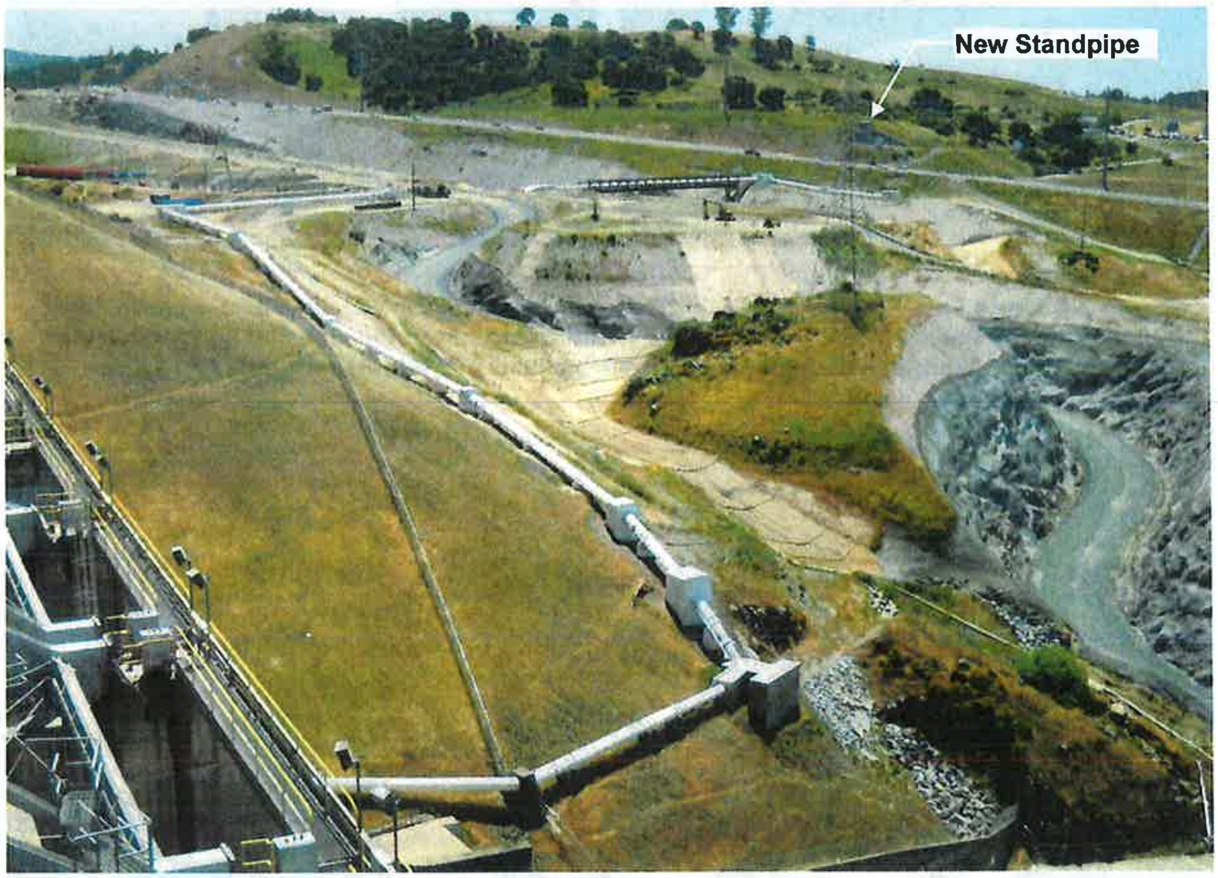

Figure 2-6 New Natoma Pipeline Alignment (May 2011), as Seen from the Top of Folsom Dam Looking East; New Standpipe in Upper Right of Picture

2.1.3 Pumps

Eight pumps are available to raise the hydraulic grade line of reservoir water to satisfy downstream demands. Seven of the pumps, five with constant velocity and two with variable frequency drives, are within the Folsom Pumping Plant (Figure 2-7); pump suction piping is connected to the 84-inch-diameter North Fork Pipeline (centerline elevation of 317.1 feet). The

[2] Per Sheet C-10, "Standpipe Plans, Section, and Details," City of Folsom "Natoma Standpipe Relocation" drawings, March 2007.

eighth pump, designated the "emergency pump," is located in a separate enclosure adjacent to Penstock No. 1 (Figure 2-8); the pump's 36-inch diameter suction line taps the penstock at elevation 261.34 feet. Pump capacities[3] range from roughly 20 to 90 cfs for lifts ranging from 84 to 126 feet. Motor horsepower range is from 250 to 1,500.

Figure 2-7 Folsom Pumping Plant Pumps, Looking South from Entrance Nearest Dam

[3] At points of maximum efficiency on pump curves.

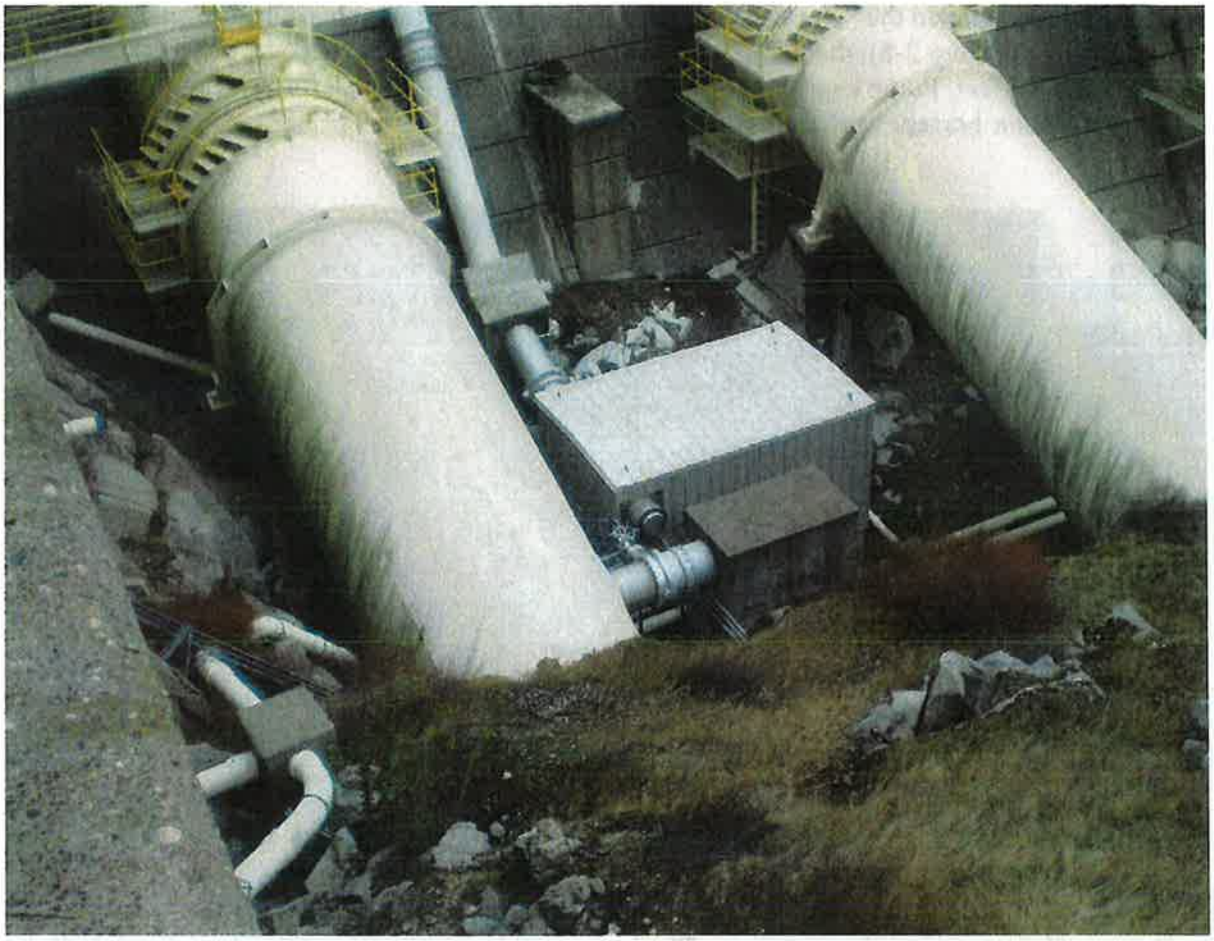

Figure 2-8 Emergency Pump Enclosure Adjacent to Penstock No. 1

2.1.4 Appurtenances

Appurtenances include surge protection towers, valves, and flow meters. Additional appurtenances include air relief and drain valves, overflow piping, and instrumentation and controls.

Surge Protection

The North Fork Pipelines have three surge protection towers:

- Two towers are located about 200 feet downstream of the pumping plant: a 12-foot diameter tower on the 84-inch-diameter pipe and a 10-foot-diameter tower on the 72-inch-diameter pipe. The towers overflow at elevation 479.34 feet.[4]
- A 10-foot-diameter standpipe on the 84-inch-diameter pipe, with overflow at elevation 479.34 feet, is located about 2,200 feet downstream of the pumping plant.

[4] As previously indicated, elevations are consistently referenced to NAVD88 datum.

The Natoma Pipeline has a 10-foot diameter standpipe with roof at elevation 440 feet (overflow at elevation 436 feet and maximum operating level at 434 feet). A 30-inch diameter pipe connects the pipeline to the standpipe.[5]

The 36-inch-diameter emergency pump discharge line has a 48-inch-diameter standpipe that rises along the outside face of Folsom Dam. The standpipe overflows at elevation 452.34 feet.

Valves

The system includes over 80 valves of various types and sizes, both manual- and motor-operated (see Figure 2-2). Most of the valves are of either the gate or butterfly type.

Flow Meters

There are flow meters on the North Fork, Natoma, SJWD, and City of Roseville pipelines, as indicated in Figure 2-2.

2.1.5 Flow Control

Flow rates are controlled by throttling valves at three of the system's four end points. Operators at the water treatment plants at SJWD, City of Roseville, and City of Folsom set a target flow rate and their automated valves open or close as needed to maintain the target flow rate. The flow rate to Folsom Prison is partially controlled by an overflow weir in a distribution box. The box is located a short distance upstream of the prison's pump station wet well, which is the raw water delivery point.

Current practice is not to throttle any of the valves in the pumping plant. Valves on the pumps' discharge pipes are programmed to open slowly as the pumps are turned on and operate fully open. Pumps 6, 7, and 8 have butterfly valves, which would allow throttling. The other pumps have gate valves, which are not designed for and could be damaged if continuously operated partially open.

2.1.6 Electrical Power Supply

A double-ended substation supplies electrical power to the pumping plant's 4.16kV switchgear (labeled "UHA"). Switchgear UHA receives power from two transformers, designated KZ4A and KV9A. Switchgear UHA has two main breakers, 52-A and 52-B; the first connects to transformer KV9A and the second to both transformer KZ4A and a tie breaker designated UHA5. The main breakers and the tie breaker are electrically interlocked.

Switchgear UHA is connected to the pumping plant's switchgears 1 and 2. Switchgear 1 provides power to 208/120V panel CPC through transformer KPA. Switchgear 2 provides power to 208/120V panel CPB through transformer KPB. Both transformers KPA and KPB provide power to 208/120V panels CPA and CPD through an automatic transfer switch.

The electrical supply to the pumping plant is discussed in greater detail in Section 5 of this document.

[5] According to Sheet C-10, "Standpipe Plans, Section, and Details," City of Folsom "Natoma Standpipe Relocation" drawings, March 2007.

2.1.7 Control System

Information about the configuration of the control system was derived from two drawings. Pumping Plant Expansion Project drawing E-3 shows that the VFD pumps (7 and 8) and their motor-operated discharge valves (V14 and V26) are hard-wired for start/stop or open/close control, interlock, and remote monitoring. The VFD pumps share an 115VAC-24VDC control power supply. The VFD control and the PLC in remote panel 1101 are mentioned in the drawing, but no details are provided.

Reclamation drawing No. 485-218-1461 is a partial representation of the Pumping Plant Central Start/Stop Control Schematic. The drawing indicates that a loss of 24V control power device 27CPC was installed (the device, however, is not shown on the drawing). How the Central Start/Stop Control is connected to each pump's start/stop control circuit is likewise not shown. The physical protection of these control circuits from central control to local pump could not be determined from available information.

2.2 Operational Data

The raw water delivery system is expected to convey water continuously to SJWD, City of Roseville, City of Folsom, and Folsom Prison water treatment plants, at the rates they individually require. Water deliveries are preferably made by gravity, when the water level in Folsom Reservoir allows it. When the water level in the reservoir is too low to satisfy demands by gravity, pumps are turned on to provide the necessary lift. VFD pumps are used before constant speed pumps. Information about water demand, water surface elevations that impact water deliveries, and VFD and constant speed operation is presented below.

2.2.1 Current and Future Water Demands[6]

Typical winter and summer demands from the four water purveyors supplied through the Folsom Pumping Plant are presented in Table 2-1. Seasonal fluctuations are illustrated in Figures 2-9 and 2-10.

Table 2-1 Typical Winter and Summer Demands

	Flow Rates (cfs)				
Demand Condition	SJWD	City of Roseville	City of Folsom	Folsom Prison	Total
Typical winter demands	40	32	25	3	**100**
Typical summer demands	170	77	57	5	**309**

Raw water demands are limited by treatment capacities. The Folsom Reservoir water goes directly into the treatment trains at the SJWD, City of Roseville, City of Folsom, and Folsom Prison treatment plants. The maximum flow rate each of these purveyors can request at any one time, therefore, is the maximum flow rate that their plants can treat. Current and future treatment capacities (i.e., maximum demands) are summarized in Table 2-2.

[6] Based on information provided in 2009 by Bill Sadler (SJWD), Shawn Barnes (City of Roseville), Todd Eising (City of Folsom), and Pedro Reyes (Folsom Prison).

Table 2-2 Current and Future* Maximum Demands

	Flow Rates (cfs)				
Purveyor	SJWD	City of Roseville	City of Folsom	Folsom Prison	Total
2009 Treatment Capacity	186	93	77	5	**361**
Planned Future Capacity	232	155	77	10	**474**

* San Juan Water District: Timing of future treatment plant upgrades is uncertain, as they depend on development within their service area and consequent demand increases.

City of Roseville: Already increased its treatment capacity from 93 cfs to what it considers the "ultimate" treatment capacity of 155 cfs, but demand will likely stay under the 93 cfs range for the next several years.

City of Folsom: No treatment planned upgrades currently planned.

Folsom Prison: Changes to increase capacity are under way in 2011.

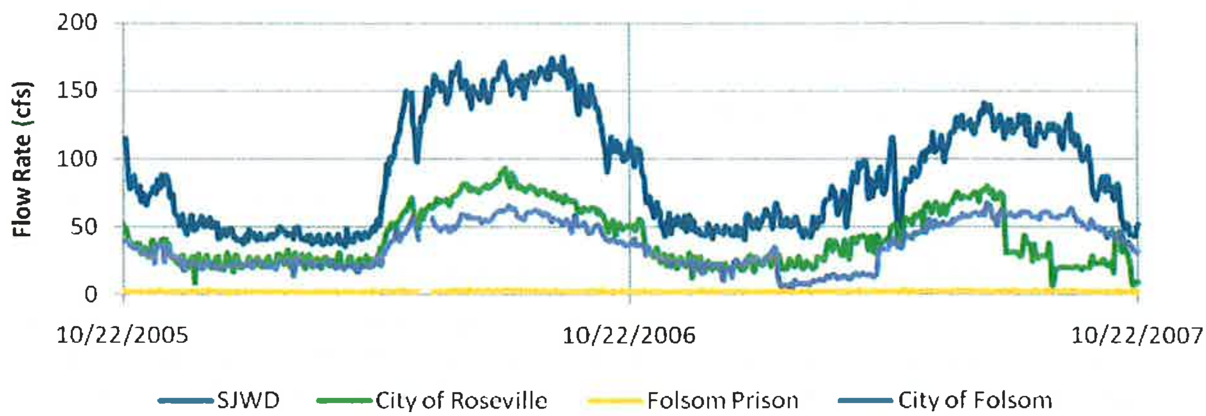

Figure 2-9 Average Daily Flows to Each of Four Purveyors, 2005-2007

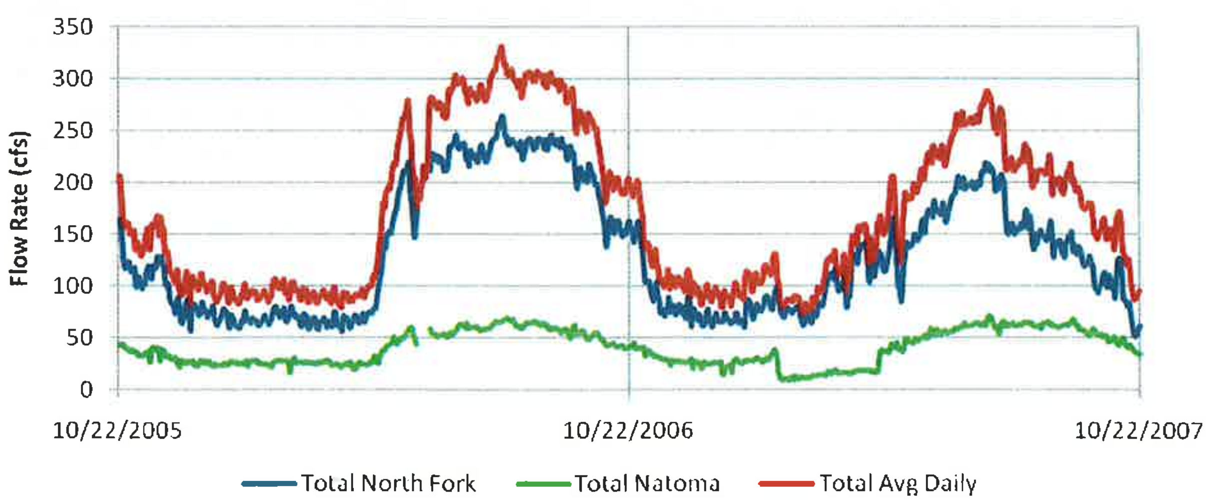

Figure 2-10 Total Average Daily Flows from 2005 to 2007

2.2.2 Elevations Relevant to Water Deliveries

Elevations relevant to water deliveries from the Folsom Pumping Plant system are listed below; most are illustrated graphically in Figure 2-11

- *Folsom Reservoir water level:* The water level in the reservoir changes continuously throughout the year, as shown in Figure 2-3. Gravity flows to the North Fork Pipelines are possible when the reservoir level is above 425 feet; summer demands would require higher reservoir levels, typically above 430 feet. Gravity flows to the Natoma Pipeline are possible when the reservoir level is above 410 feet.
- *End point water levels:* Water levels at the end points, shown graphically in Figure 2-11, remain largely unchanged over time, as they depend on process elevations in each water treatment plant.
- *Elevations of raw water intakes:* Very low water levels in the reservoir, at or below elevation 332 feet, render the intake to the pumping plant unusable.[7] The intake to the emergency pump would remain usable for reservoir water levels as low as 310 feet; the emergency pump draws water from one of the power penstocks, located roughly 20 feet lower than the raw water intake. The actual intake point from the penstock to the pump suction pipe is at elevation 261.34 feet.
- *Overflow level in North Fork Pipelines standpipes:* The overflow level in the standpipes is higher than the maximum lake level and would therefore not be reached under gravity flow conditions; it could be exceeded, however, during pumping operations.
- *Overflow level in Natoma Pipeline standpipe:* The overflow level in the new standpipe is at elevation 436 feet; the water level in the reservoir goes above that elevation almost every year (see Figure 2-3); selected valves on the Natoma Pipeline are closed or throttled to prevent overflows at reservoir levels above 436 feet.
- *Overflow level in emergency pump standpipe:* The emergency pump standpipe overflows above elevation 452.34 feet. Water levels need to be monitored and the 36-inch butterfly valve on the discharge line possibly throttled to prevent standpipes from overflowing when the emergency pump is activated. If the emergency pump is operated in accordance with its Standard Operating Procedure no valve throttling would be required, as the pump would not be operated for reservoir levels above 330 feet; the pump has a 100-foot lift and would therefore not be able to reach the overflow level when operated at reservoir levels of 330 feet and lower.

[7] At least 10 feet of water depth above the crown of the intake are needed to prevent air entrainment, which could lock the pumps and/or result in cavitation.

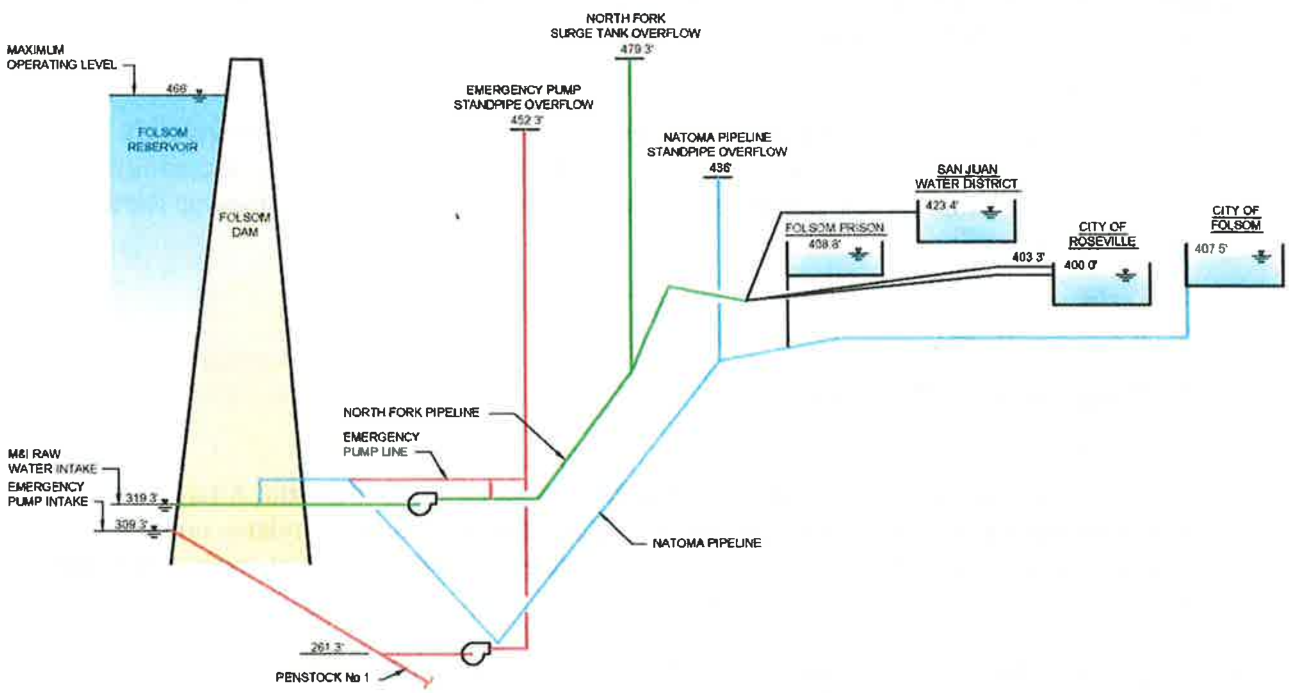

Figure 2-11 Water Surface Elevations that Impact Water Deliveries

ARWA-202

2.2.3 Pump Operation

Pumps can be operated from the Folsom Power Plant control room or from the pumping plant. On and off controls are available at both the control room and the pumping plant. Operators prefer to operate pumps from the pumping plant in order to visually confirm proper operation. Standpipe set points for variable speed pump operation can be changed only at the pumping plant.

Variable speed pumps are generally turned on before constant speed pumps. Operators limit operation of the VFDs to speeds between 50 and 75 percent of full speed and pump controls are locked to prevent operation above 75 percent speed. Operators also avoid running the two VFDs together for reasons explained in Section 3 of this document. Under current operating procedures, pumps are activated as follows:

1. Operators assess demand based on requests from four purveyors.
2. Operators select pumps to satisfy total demand. Either Pump 7 or Pump 8 (pumps with VFDs) is selected to satisfy demands up to 85 cfs; if demand exceeds 85 cfs, one or more constant speed pumps are turned on along with one VFD, based on labels on pump startup buttons which read:
    - Pump 2 = 25 cfs
    - Pump 3 = 75 cfs
    - Pump 4 and 5 = 50 cfs each
    - Pump 6 = 100 cfs
3. Operators activate pumps, generally from the controls in the pumping plant. A target North Fork Pipeline surge tank level is selected by looking it up in a table that relates surge tank levels to SJWD demand. A setting for the VFD is determined by looking it up on a table that relates surge tank level to VFD set point.

Pump operation and pump capacities are discussed in detail in Section 3 of this document.

3 HYDRAULIC PERFORMANCE EVALUATION

The hydraulic performance of the raw water delivery system was evaluated to identify physical and operational deficiencies in the pumping system at current delivery volumes and recommend corrective measures. The evaluation also helped define current operational limits of the pumping plant and pipelines. This section of the report presents:

- Basis of hydraulic performance evaluation
- Ability to satisfy demands through gravity flows
- Ability to satisfy demands through pumping
- Conclusions about hydraulic performance

3.1 Basis of Hydraulic Performance Evaluation

The hydraulic performance of the raw water delivery system associated with the Folsom Pumping Plant was evaluated on the basis of:

- Available data
  - ❖ Document review
  - ❖ Inspections
  - ❖ Interviews
- Field tests to measure actual head losses
- Computer simulations

3.1.1 Available Data

Document Review
A variety of drawings and documents were reviewed, including (see Appendix A for a full list):

- Pumping plant expansion drawings prepared by SAI Engineers for USBR in 1997.
- Roseville 60" Raw Water Pipeline Project drawings prepared by Boyle Engineering in 2001.
- Natoma Standpipe Relocation drawings prepared by Robert W. Miles for the City of Folsom in 1997.
- Construction of Natoma Pipeline Phase A drawings prepared by Robert W. Miles for the City of Folsom in 1998.
- Natoma Raw Water Pipeline Phase B drawings prepared by CDM for the City of Folsom in 2000.
- Folsom Pumping Plant Training for Pumps 7 & 8 Operation, Preliminary Session, March 13, 2000 Agenda, prepared by Will B. Betchart in March 2000.
- Folsom Pumping Plant Capacity Evaluation, Attachment B-1: Maximum Bypass Capacity through the Discharge Header Assuming Maximum Flood Control Water Surface, prepared by Will B. Betchart in December 2004.

- Folsom Pumping Plant Emergency Pump Test, prepared by Will B. Betchart in December 2004.
- Designer's Operating Criteria and Standard Operating Procedure, Folsom Dam Emergency Pumping Plant, prepared by USBR.
- Pump Test Data from Ingersoll-Dresser Pump Company, May 1998.
- Daily Flow Data for 2004-2006 provided by SJWD and City of Roseville.
- Daily Flow and Pump Operation Data for 2004-2006 provided by USBR.

Inspections
Above-ground system components were visually inspected several times over the course of the project. The inspections served to confirm and supplement information on drawings and other documents.

Interviews
Interviews were conducted with:

- Reclamation's mechanical and electrical engineering staff at the Folsom office, to discuss system design and operating criteria.
- Folsom Pumping Plant operators, to review current operating practices and discuss system limitations.
- Treatment plant operators at SJWD, City of Roseville, City of Folsom, and Folsom Prison, to discuss raw water demands, flow controls, future expansions, and raw water delivery details.

3.1.2 Field Tests to Measure Actual Head Losses

Field test were conducted to measure actual energy losses during system operation. Tests were performed on April 29, 2009, following a previously devised and approved test plan. Data inconsistencies prompted a topographic survey to verify key pipeline elevations; WRE conducted a simple survey to verify North Fork Pipeline elevations from the pumping plant to the Hinkle Y. Once the data inconsistencies were resolved, a memorandum presenting the results of the head loss tests was prepared (attached in Appendix B). Test results were used to calibrate the computer model that was developed to evaluate hydraulic performance.

3.1.3 Computer Simulations

A hydraulic model of the system was developed to simulate current and future operating conditions. InfoWater, a geospatial water distribution system modeling tool, was used to model the raw water delivery system. The attributes of system components (elevations of junctions and valves, lengths and diameters of pipes, pump characteristics, reservoir water levels) were coded into the InfoWater model.

The computer model was initially calibrated using calculated head losses and later re-calibrated using data from field tests conducted in April and September, 2009. The calibrated version of the model closely reproduces head losses measured during field tests.

Additional details of model development and a listing of the system characteristics entered into the model are included in Appendix C.

3.2 Ability to Satisfy Demands through Gravity Flows

The ability of the system to satisfy demands through gravity flows was assessed by evaluating:

- North Fork Pipeline gravity flows.
- Natoma Pipeline gravity flows.
- Frequency of gravity flows.

3.2.1 North Fork Pipeline Gravity Flows

The threshold water levels at which typical and maximum water deliveries become possible via gravity flow were calculated using the hydraulic model. Computer simulations were performed assuming simultaneous delivery to four end users. Valves at each end point were throttled as needed to achieve the combination of demands under consideration. Typical demands were derived from historical data; current and future treatment plant capacities were provided by Chief Operators at each site.

The reservoir water levels at which deliveries can be made through the North Fork Pipeline are controlled by SJWD. The target water level at the SJWD treatment plant is 425.4 feet. The target water level at the City of Roseville treatment plant is 400 feet. Reservoir water levels higher than 400 feet would make possible gravity flows to the City of Roseville, but only water levels higher than 425.4 would make possible gravity flow to SJWD. In order to deliver to the two purveyors simultaneously, the higher reservoir water levels must be used, hence the intake valves at the City of Roseville treatment plant must be throttled accordingly.

The computed threshold water levels are listed in Table 3-1. The simulations assumed all flow through the (original) 84-inch-diameter pipeline. If the 72-inch pipeline alone were used, the threshold levels would be higher. If the two pipelines were used together, the threshold levels would be slightly lower than those presented in Table 3-1.

Table 3-1 Reservoir Water Levels Required to Meet North Fork Pipeline Demands by Gravity

Demand Condition	Flow Rates (cfs) SJWD	Roseville	Total	Min. Water Level in Folsom Reservoir (ft)
Typical winter demands	40	32	72	426
Typical summer demands	170	77	247	439
Current maximum demands (Existing capacity of treatment plants)	186	93	279	441
Future maximum demands (Projected treatment plant capacities)	232	155	387	455

3.2.2 Natoma Pipeline Gravity Flows

The threshold water levels for typical and maximum demands were similarly computed for the Natoma Pipeline. Since it is possible to deliver water by gravity to Natoma Pipeline end users while pumping to North Fork Pipeline end users (closing valve V5 while V10 is open), the target levels for SJWD do not control threshold levels for Natoma Pipeline flows.

The reservoir water levels at which deliveries can be made through the Natoma Pipeline are controlled by target water levels at the City of Folsom treatment plant (elevation 407.5 feet as shown in Figure 2-11) and Folsom Prison (elevation 408.8 feet). Reservoir water levels above elevation 436 feet require valve throttling to prevent overflows at the standpipe.

The computed threshold water levels are listed in Table 3-2. The simulations assumed the pipe lengths and diameters corresponding to the new Natoma Pipeline, as reflected in *Construction of Natoma Pipeline Phase A* drawings.

Table 3-2 Reservoir Water Levels Required to Meet Natoma Pipeline Demands by Gravity

Demand Condition	Flow Rates (cfs) City of Folsom	Folsom Prison	Total	Min. Water Level in Folsom Reservoir (ft)
Typical winter demands	25	3	28	413
Typical summer demands	57	5	62	424
Current maximum demands (Existing capacity of treatment plants)	77	5	82	435
Future maximum demands (Projected treatment plant capacities)	77	10	87	436

3.2.3 Frequency of Gravity Flows

The raw water delivery system cannot satisfy demands year-round by gravity alone. Between January 1, 2000 and November 30, 2007 gravity flows were possible between 28 and 55 percent of the time:

- *North Fork Pipeline:* 698 days or 28 percent of the total number of days.
- *Natoma Pipeline:* 1,412 days or 55 percent of the total number of days.

These frequencies are based on actual reservoir water levels, actual water demands, and calculated threshold levels generated by the computer model. The frequencies do not necessarily reflect actual system operation during that period.

Future reservoir water levels are difficult to predict. If they remain at approximately the same levels observed in the past decade, demand increases will shorten the amount of time that raw water deliveries can be made with gravity flows.

3.3 Ability to Satisfy Demands through Pumping

The ability to satisfy demands through pumping was assessed by evaluating:

- Pump capacities
- Pump operating ranges
- Operational constraints
- System's pumping capacity

3.3.1 Pump Capacities

Pump curves - graphical representations of the relation between flow rate, total dynamic head (TDH), pump efficiency, and brake horsepower - are available for all pumps in the Folsom Pumping Plant as well as for the emergency pump. Pump curves are typically based on flow tests conducted at the manufacturer's site before pump delivery. The curves are generally verified after the pumps are installed before an owner accepts the pumps and puts them into operation.

Pump characteristics at their highest efficiency point are summarized in Table 3-3. The actual pump curves are included in Appendix D. The emergency pump (not listed in Table 3-3) has the same characteristics as pump 6.

Table 3-3 Pump Characteristics at Peak Efficiency

Pump	Flow Rate (cfs)	TDH (ft)	Peak Efficiency	Brake Horsepower
2	20	100	88%	260
3	50	98	90%	610
4 & 5	40	84	88%	410
6	80	86	87%	560
7 & 8*	87	125	90%	1,370
Total	**404**	-	-	**4,990**

* Variable speed pumps; characteristics shown are for maximum pump speed of 511 rpm.

3.3.2 Pump Operating Ranges

Operating ranges for the constant speed pumps were verified by Flowserve Corporation, owners of Worthington Pumps, the manufacturer of the Folsom Pumping Plant's constant speed pumps. Application Engineer Stephen Thorwart of the Flowserve facility in Rancho Dominguez, California, indicated that their pumps can generally be expected to operate satisfactorily when run at no less than 80 percent of their peak efficiency (i.e., 0.8 x Peak Efficiency). Below that level of efficiency, the pumps do not necessarily follow the pump curve and are subject to cavitation, recirculation, and uneven loading of moving parts that will significantly shorten pump life.

Acceptable operating ranges for constant speed pumps, based on manufacturer's pump curves, are presented in Table 3-4. The actual pump curves are included in Appendix D.

Table 3-4 Acceptable Operating Ranges of Constant Speed Pumps at Folsom Pumping Plant

Pump	Flow Rate Range (cfs)	TDH Range (ft)	Min. Acceptable Efficiency
2	10 to 28	60 to 124	70%
3	22 to 69	64 to 114	72%
4 & 5	18 to 51	50 to 116	70%
6 & Emerg.	41 to 106	50 to 106	70%

Variable speed pumps generally operate over a wider range of flow rates and head than constant speed pumps. The nominal capacity of the variable speed pumps at different heads and speeds is illustrated in Figure 3-1.

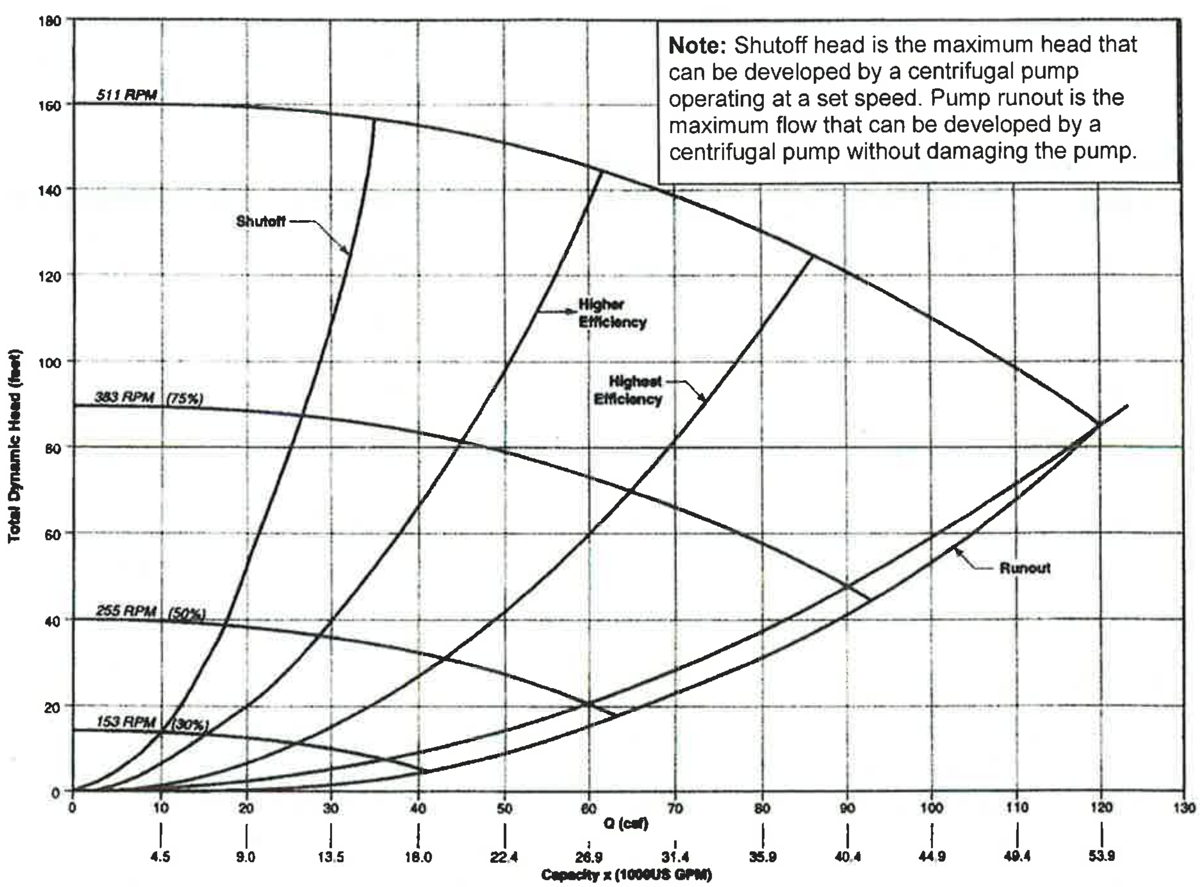

Figure 3-1 Variable Speed Pump Performance Curves

Source: Folsom Pumping Plant Training for Pumps 7 & 8 Operation, Preliminary Session, March 13, 2000 Agenda, prepared by Will B. Betchart in March 2000.

3.3.3 Operational Constraints

Reported by Operators

Operators indicated that the following constraints, developed through operational experience, are applied to pumping plant operation:

- Pumps 7 and 8 are not operated together.
- Pumps 7 and 8 are operated individually only at 50 to 75 percent of maximum speed.
- Discharge valves on constant and variable speed pumps are kept fully open during pump operation.

Pumps 7 and 8 are normally operated individually in automatic mode. Pump controls are locked to prevent pump speed from increasing above 75 percent of full speed.

Confirmed by Field Tests

Pump operation tests were conducted on September 25, 2009 (see Appendix B for memorandum summarizing pump tests). Pumps were operated without valve throttling. Test observations showed that:

- Pumps 7 and 8 do not perform well when operated together at 75 percent speed; pumps operated acceptably well, however, at 65 percent speed.
- Pumps 7 or 8 do not perform well above 75 percent of full speed.
- Reported poor performance of the variable speed pumps at speeds lower than 50 percent speed was not observed during tests with speeds as low as 40 percent.

Pump power consumption tests were conducted on October 26, 2009 (see Appendix B for test summary). These tests showed that pumps 2 through 5 were operated outside manufacturer-recommended efficiency ranges. The lowest measured efficiency was 26 percent (Pump 5).

Performance Problems Observed

A phenomenon known as "discharge recirculation" occurred when the VFD pumps were operated individually at greater than 75 percent speed or together at greater than 65 percent speed. This phenomenon is characterized by random crackling noises and intermittent knocking sounds in the suction and discharge piping. Discharge recirculation causes cavitation pitting of the impeller resulting in poor pump performance (off the pump curve) and eventual mechanical failure.[8]

Discharge recirculation was also evident at the constant speed pumps operated during the field tests. The recirculation could be the result of operating the pumps well outside their prescribed efficiency range, unfavorable approach flow conditions, or a combination of both. When the discharge valve on Pump 3 was throttled to increase pump TDH (going outside of normal operation protocol for a limited time), the noises that characterize discharge recirculation dissipated at about the half-closed position.

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[8] A more detailed description of discharge recirculation can be found at http://www.lawrencepumps.com/Newsletter/news_v04_i4_Apr07.html

Likely Cause of Discharge Recirculation

Approach flow conditions are a possible cause of discharge recirculation. Impellers are designed with the assumption that incoming flow is evenly distributed throughout the approach section. A number of approach flow conditions have been identified in laboratory tests to be detrimental to impeller performance:

- *Uneven flow distribution*: flow tends to favor one side over the other.
- *Pre-rotation*: flow approaches the impeller in a circulatory pattern that may or may not be in the same direction that the impeller rotates.
- *Vorticity*: a tight flow spiral forms immediately upstream of the impeller.

These conditions are generally a function of the geometry of the approach section. The approach geometry for all pumps at the Folsom Pumping Plant is likely to cause approach flow problems, even when pumps operate within acceptable efficiency ranges.

3.3.4 System's Pumping Capacity

Ideal Conditions

If pumps within the pumping plant (i.e., not including the emergency pump) were to operate at the peak efficiencies shown in Table 3-3, the total system capacity would be 404 cfs, sufficient to meet maximum (2009) demands, which are estimated at 361 cfs (Table 2-2).

Constraints

It is impossible to operate all pumps at maximum efficiency under current conditions for the following reasons:

- *Pumps have different TDHs and there is no valve throttling:* Since valves on the suction and discharge sides of the pumps are operated fully open, pumps in operation are subject to the same head (i.e., the pressure differential between the suction side and the discharge side would be about the same, other than for minor losses which could be slightly different for each pump). Without individual throttling of discharge valves, there is no way to set the head for each pump at its optimum level.
- *Variable speed pumps are operated at 50 to 75 percent of full speed:* The range at which variable speed pumps are operated is illustrated in Figure 3-2.

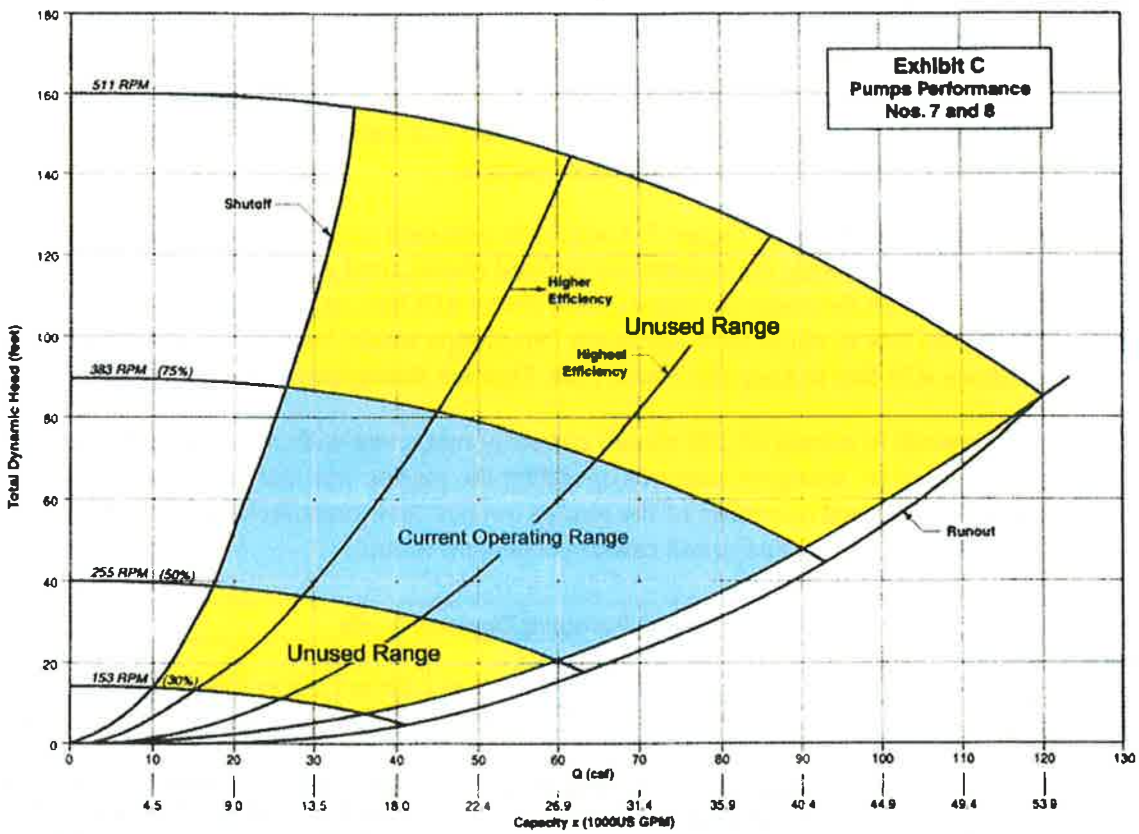


Figure 3-2 Range at which Variable Speed Pumps Are Operated

- *Unfavorable intake geometry:* The geometry of the pump intakes, which consist of different diameter pipes branching off an 84-inch diameter manifold, is likely to cause uneven flow distributions and vorticity that lower pump efficiency. Even if operated within the ranges prescribed by the manufacturer, the pumps are likely to operate at reduced efficiency.

Theoretical Capacity

A system capacity was calculated for a theoretical scenario in which:

- Constant speed pumps were operated to stay within the ranges prescribed by pump curves (included in Appendix D) and summarized in Table 3-4.
- Butterfly valves on the discharge of pumps 6, 7, and 8 were throttled when needed to keep the pumps within acceptable operating ranges.
- Surge tank water levels on the North Fork Pipeline were kept below the overflow elevation of 479.3 feet, with the target generally set at elevation 455 feet.
- Pumps 7 and 8 were operated between 50 and 75 percent speed.
- When needed to meet demands, pumps 7 and 8 were operated together at speeds not exceeding 65 percent, one manually and the other one in automatic mode.

A pump selection schedule using the parameters described above is presented in Appendix F.

The theoretical scenario described above is achievable with the system that is now in place. Operating constraints would not be altered, with the exception of introducing throttling for valves on the discharge pipes for pumps 6, 7, and 8, for which throttling is feasible. The operating instructions for pumps 7 and 8 actually indicate that valve throttling should be part of standard operating practices for the variable speed pumps.

The system capacity illustrated in Figure 3-3 would be obtained under the theoretical operating scenario. When operating using the constraints defined above, total system demands in excess of 220 cfs could not be met for reservoir water levels above 410 feet, as the TDH for constant speed pumps would be too low to allow their operation. No pumps would be operated for reservoir water levels above 430 feet to keep the North Fork Pipeline standpipes from overflowing.

Total system demands in excess of 220 cfs are currently met, even with reservoir water levels above 410 feet. Doing so, however, requires operating the pumps well outside their normal operating ranges. Continued operation of the pumps outside their manufacturer-recommended ranges will damage the pumps and could cause mechanical failure.

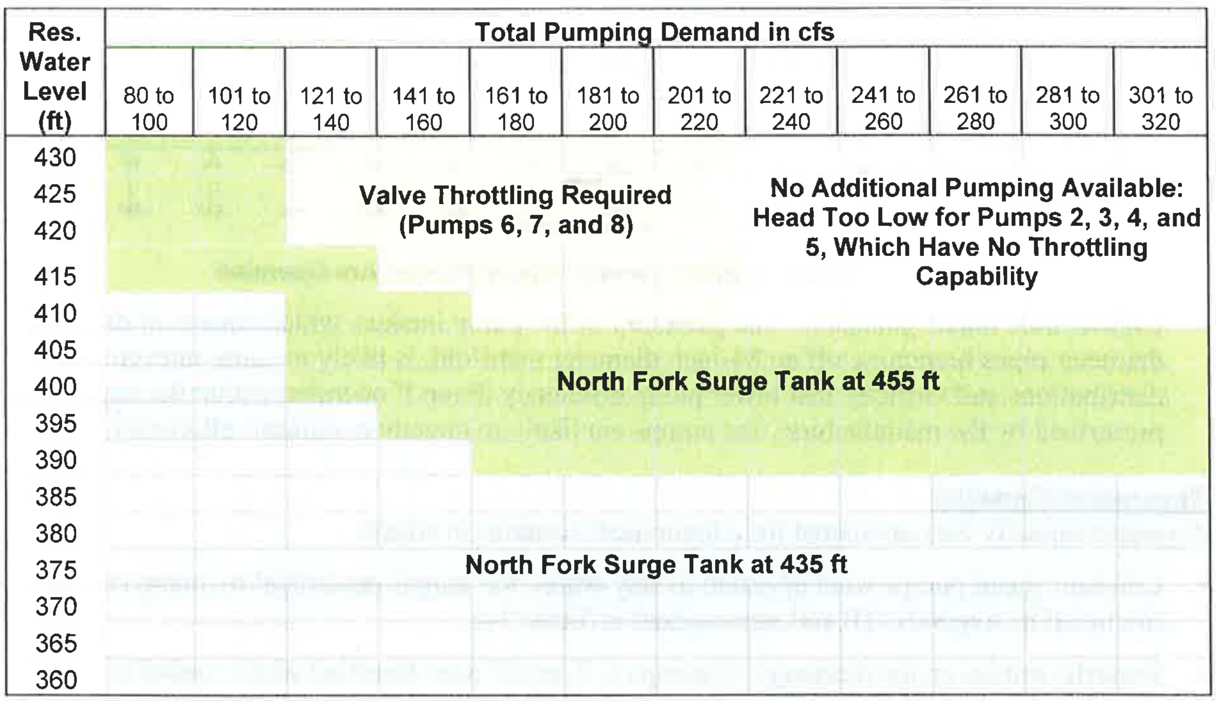


Figure 3-3 System Capacity When Operating All Pumps within Acceptable Ranges

3.4 Conclusions about Hydraulic Performance

The hydraulic performance evaluation identified:

- Physical deficiencies
- Operational deficiencies
- Operational limits
- Potential corrective measures

3.4.1 Physical Deficiencies

Pumping Plant Intake Geometry

The geometrical design of pump intakes at the Folsom Pumping Plant generates adverse approach flow conditions that affect pump performance. Suction pipes come off the 84-inch-diameter pipeline at a 30-degree angle. The intake to Pump 2 is first, followed by intakes to pumps 3 through 8 (Figure 3-4). The streamlines into each suction pipe change depending on the pumps that are operating; due to the geometrical arrangement, streamlines generate a skewed flow distribution at pump impellers. The skewed flow distribution causes uneven loading of moving parts and can result in vorticity and pre-rotation. These phenomena lower pump efficiency and can eventually damage the pump.

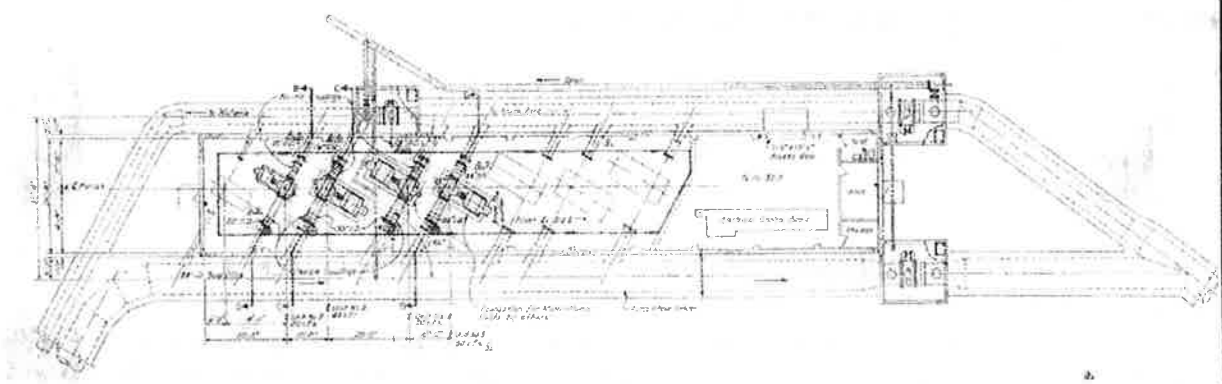


Figure 3-4 Plan View of Pump Arrangement
Source: "Pumping Plant Equipment, Mechanical – Pump Installation" drawing, Folsom Reservoir Project, US Army Corps of Engineers, September 1951

Valving Arrangement on New North Fork Pipeline

The current valving arrangement conveys water from Folsom Reservoir to both the SJWD and the City of Roseville through the North Fork Pipeline. Both gravity and pumped flows have the same path.

The thresholds for gravity flow are quite different for SJWD and the City of Roseville. The end point water surface elevation at SJWD is 423.4 feet, while at the City of Roseville it is 400 feet. Demands from the City of Roseville could be met by gravity with reservoir levels of 405 feet and higher. Demands from SJWD, however, can only be met with reservoir levels above 426 feet.

If the pipes and valves were re-arranged to separate deliveries to the City of Roseville from deliveries to SJWD, the number of days in which gravity flows to Roseville were possible would increase (and therefore the number of days in which pumping was required would decrease). Between 2001 and 2007, Folsom Reservoir had water levels between 405 and 426 feet about 13 percent of the time. Those 326 days (an average of 47 days per year), the City of Roseville could have been served through gravity flows had the piping arrangement been suitable.

Separating Roseville and SJWD flows would require a connecting pipe from the pumping plant's suction manifold to the new 72-inch diameter North Fork Pipeline. Several additional valves would also be required to allow both gravity and pumped flows into the new pipeline.

3.4.2 Operational Deficiencies

Pump Selection
Current operating procedures do not take into account the characteristics of the constant speed pumps (i.e., "pump curves") and their operating ranges. The constant speed pumps are selected for operation based on their rated flow capacity only. The variable speed pumps are operated at 50 to 75 percent of the total speed, without regard to total dynamic head (TDH). To deliver its rated capacity, however, a pump requires a particular TDH. At the appropriate TDH, a pump operates at peak efficiency and delivers its rated flow.

Valve Throttling
To reach the appropriate TDH, discharge valves would have to be throttled. Only Pumps 6, 7, and 8 are equipped with discharge valves suited for throttling.

Throttling of the valves on pumps 7 and 8 is likely to increase their range of operation. The training manual on the operation of the variable speed pumps indicates the appropriate valve angle required to achieve the proper TDH. Adherence to these guidelines would improve pump performance.

3.4.3 Operational Limits

Gravity Flows
The raw water delivery system is capable of satisfying current and anticipated future demands by gravity, when the reservoir water level is high enough to allow it. From 2000 to 2007, the water level in the reservoir was high enough to allow deliveries by gravity to the North Fork Pipeline about 28 percent of the time. Pumping was required the remaining 72 percent of the time. If reservoir water levels were to remain in the same range in the future, demand increases will shorten the time periods in which raw water deliveries can be made with gravity flows.

Pumped Flows
The 7 pumps in the Folsom Pumping Plant have a combined rated capacity of 404 cubic feet per second (cfs) when operated at peak efficiency. Current typical summer demands are approximately 309 cfs (Table 2-1). Maximum current demands based on treatment capacities at the end points are about 361 cfs; maximum future demands based on anticipated treatment plant expansions would be 474 cfs (Table 2-2). If the pumps were operated at peak efficiency, they would be able to meet current typical and maximum demands; they would not be able to meet future maximum demands. Operation of the pumps at peak efficiency would generally require valve throttling: by partially closing the discharge valves, additional head would artificially be created that would bring the pumps to their most efficient operating level.

3.4.4 Potential Corrective Measures

New Pump Intake Structure
A more efficient pump intake would improve pump performance. Design of an appropriate structure would require physical modeling.

Revised Operating Procedures
Operating procedures must consider each pump's TDH and its acceptable operating range. Operation outside manufacturer-recommended ranges should be avoided.

New Discharge Valves/Controls

The only way to operate the pumps at peak efficiency given the prevailing lake levels is by throttling discharge valves. Three of the seven pumps in the pumping plant have butterfly valves suited for throttling. The other four pumps should have discharge valves that allow throttling. The discharge valves should be automated to open only to the point where the differential head across the pump matches the ideal TDH.

Pump Replacement

The implementation of valve throttling would allow existing pumps to operate efficiently at any reservoir water level. If future reservoir water levels are similar to the levels of the past 10 years, significant valve throttling would be required: constant speed pumps have rated heads of 84 to 100 feet, but the vast majority of the time they would be pumping against lower heads. Pumping to meet a TDH of 100 feet when the water level differential is much lower wastes power. Replacement of the constant speed pumps with variable speed pumps would reduce power consumption.

4 EVALUATION OF THE EFFECTS OF HIGHER DEMANDS

The conveyance of higher-than-current flow rates through the raw water delivery system was evaluated to assess the effects on system components and plant operations. This section of the report presents:

- Basis of evaluation
- Impact of higher flow velocities
- Raised gravity flow thresholds
- Increased frequency of pump use
- Increased power consumption
- Conclusions about impacts of higher demands

4.1 Basis of Evaluation

The effects of conveying higher-than-current flow rates through the Folsom Pumping Plant raw water delivery system were evaluated on the basis of:

- Hydraulic calculations of flow velocities through system components.
- Computer simulations of gravity flows assuming simultaneous deliveries to four purveyors, to calculate gravity flow thresholds for future demand conditions.
- Review of flow and reservoir level data provided by Reclamation for the period between January 1, 2001 and November 29, 2007, to assess increased frequency of pump use for the future demands outlined in Section 2 of this document.
- Power consumption calculations for increased demands, assuming pumps are operated within manufacturer-specified ranges.

4.2 Impact of Higher Flow Velocities

A comparison of flow velocities in the system's pipelines, for current and future maximum demands, is presented in Table 4-1. Estimated energy losses due to friction for every 1,000 feet of pipeline are included in the table.

The increase in flow velocities would have the most significant effect on the North Fork Pipeline. At a future maximum delivery rate of 387 cfs, velocity in the 84-inch-diameter pipe would reach 10.0 feet per second (ft/s). Although the pipe should be able to sustain a velocity of this magnitude without detrimental abrasion or scouring, energy losses due to friction would almost double from 1.5 to 2.7 ft/thousand feet of pipe. Energy losses could be reduced by splitting the total flow between the 84- and 72-inch-diameter pipelines.

Table 4-1 Comparison of Flow Velocities for Maximum Delivery Rates, Current and Future

Condition	Pipeline	Max Flow Rate (cfs)	Max Flow Velocity (ft/s)	Friction Loss per 1,000 ft of Pipe (ft)
Current	84-inch North Fork	279	7.2	1.5
Future	84-inch North Fork	387	10.0	2.7
Current	42-inch Natoma	82	8.5	4.5
Future	42-inch Natoma	87	9.0	5.0
Current	48-inch Natoma	82	6.5	2.3
Future	48-inch Natoma	87	6.9	2.6
Current	60-inch Natoma	82	4.2	0.8
Future	60-inch Natoma	87	4.4	0.9

4.3 Raised Gravity Flow Thresholds

4.3.1 North Fork Pipeline

A comparison of reservoir water levels required for gravity flow deliveries, for current and future demands, is presented in Table 4-2. The "current" capacity of the City of Roseville water treatment plant was assumed to be 93 cfs, its capacity before recent improvements. Summer demands were assumed to range from 90 to 100 percent of treatment capacity. Future winter demands were assumed at 25 percent above current levels.

Table 4-2 North Fork Pipeline: Comparison of Threshold Reservoir Levels for Gravity Flow*

Condition	Demand	Flow Rates (cfs) Roseville	Flow Rates (cfs) SJWD	Flow Rates (cfs) Total	Min. Water Level in Folsom Reservoir (ft)
Current	Winter Avg. Daily	26	54	80	426
Future	Winter Avg. Daily	34	68	102	427
Current	90% Capacity	84	167	251	438
Future	90% Capacity	140	209	349	449
Current	100% Capacity	93	186	279	441
Future	100% Capacity	155	232	387	455

* Based on computer simulations that assumed simultaneous deliveries to four end users.

As indicated in Table 4-2, to deliver maximum future demands by gravity, the water level in the reservoir would have to be at least 455 feet. The model simulations assumed that the Natoma

Pipeline surge tank (overflow elevation at 436 feet) would be isolated to prevent overflows. SJWD valves were assumed fully open, and the valves at the City of Roseville water treatment plant were throttled to divide flow evenly between the 48- and 60-inch-diameter Roseville pipelines. Conveying most of the flow to the City of Roseville through the 60-inch-diameter pipe would reduce head losses (i.e., lower the reservoir level); current practice, however, is to split flow about evenly between the two pipelines.[9]

4.3.2 Natoma Pipeline

A comparison of reservoir water levels required for gravity flow deliveries for current and future demands is presented in Table 4-3. The current maximum delivery to the Folsom Prison Water Treatment Plant was limited to 5 cfs, based on the constraint imposed by the overflow weir in the distribution box upstream of the delivery point. The future maximum of 10 cfs assumed that improvements would be made to the overflow weir. Summer demands were assumed to range from 90 to 100 percent of treatment/delivery capacity. Future winter demands were assumed at 25 percent above current levels.

Table 4-3 Natoma Pipeline: Comparison of Gravity Flow Threshold Reservoir Levels*

		Flow Rates (cfs)			Min. Water Level
Condition	**Demand**	**Folsom Prison**	**City of Folsom**	**Total**	**in Folsom Reservoir (ft)**
Current	Winter Avg. Daily	3	23	26	411
Future	Winter Avg. Daily	4	29	33	413
Current	90% Capacity	4.5	69	74	430
Future	90% Capacity	9	69	78	433
Current	100% Capacity	5	77	82	435
Future	100% Capacity	10	77	87	437

* Based on computer simulations that assumed simultaneous deliveries to four end users.

As indicated in Table 4-3, to deliver maximum future demands by gravity the water level in the reservoir would have to be at least 437 feet. The water level at the Natoma Pipeline surge tank did not reach its overflow level (elevation 436 feet) during the gravity flow simulations summarized in Table 4-3.

[9] Based on flow data for 2005 and 2006 provided by the City of Roseville.

4.4 Higher Frequency of Pump Use

The frequency of pump use was analyzed on the basis of flow and reservoir level data provided by Reclamation for the period between January 1, 2001 and November 29, 2007. Flow and reservoir level data for these 2,524 days were examined to determine the percentage of days in which pumps would have been required to deliver raw water to the various purveyors. This determination was made on a theoretical basis (i.e., based on threshold levels for gravity flow determined by the hydraulic model) and does not necessarily reflect actual pumping plant operation during that time period.[10]

The future frequency of pump use was calculated by assuming increased daily deliveries and the same reservoir levels recorded between 2001 and 2007. The daily delivery data for the 2,524 days of record were multiplied by a factor that accounts for the anticipated delivery increase.

The higher frequency of pump use calculated as described above assumes that the reservoir levels between 2001 and 2007 are representative of future conditions.

4.4.1 North Fork Pipeline

Increased deliveries to North Fork Pipeline purveyors are limited by the capacities of the treatment plants at the end points. The maximum increase for SJWD would be 25 percent, from the current capacity of 186 cfs to a future capacity of 232 cfs. The maximum increase for the City of Roseville would be 67 percent, from 93 to 155 cfs.

Increased frequency of pump use is illustrated in Table 4-4, which is based on flow and reservoir level data for the 2,524 days between January 1, 2001 and November 29, 2007. At current delivery conditions, pumps would have to be used, on average, 70 percent of the time. Increasing deliveries would raise the frequency of pump use to as much as 87 percent of the year.

[10] Theoretical rather than actual operation was used in the analysis to provide a valid before-and-after comparison; actual operation was not consistent in gravity flow threshold levels and pump selection.

Table 4-4 North Fork Pipeline: Frequency of Pump Use*

Condition	Reservoir Level and Flow	No. of Days	Percent
Current	Reservoir level above 441 ft (no pumping)	591	24
	Reservoir level between 426 and 441 ft, flows under 80 cfs (no pumping)	154	6
	Reservoir level between 426 and 441 ft, flows over 80 cfs (pumping required)	540	21
	Reservoir level under 426 ft (pumping required)	1,239	49
Future: SJWD and Roseville Demands Up 25%	Reservoir level above 455 ft (no pumping)	222	9
	Reservoir level between 427 and 455 ft, flows under 102 cfs (no pumping)	149	6
	Reservoir level between 427 and 455 ft, flows over 102 cfs (pumping required)	867	34
	Reservoir level under 427 ft (pumping required)	1,286	51
Future: SJWD Demand Up 25% and Roseville Demand Up 67%	Reservoir level above 455 ft (no pumping)	222	9
	Reservoir level between 427 and 455 ft, flows under 102 cfs (no pumping)	100	4
	Reservoir level between 427 and 455 ft, flows over 102 cfs (pumping required)	916	36
	Reservoir level under 427 ft (pumping required)	1,286	51

* Based on flow and reservoir level data for the 2,524 days between 01/01/2001 and 11/29/2007

4.4.2 Natoma Pipeline

Increased deliveries to Natoma Pipeline purveyors were limited to the doubling of the delivery capacity to Folsom Prison, from 5 to 10 cfs. Deliveries to the City of Folsom were assumed to remain at a maximum of 77 cfs based on information from treatment plant management.

Frequency of pump use is illustrated in Table 4-5. The number of days when pumping to the Natoma Pipeline is necessary would actually decrease slightly, from 46 percent to an estimated 42 percent, if deliveries to Folsom Prison increased to 10 cfs.

Table 4-5 Natoma Pipeline: Frequency of Pump Use*

Condition	Reservoir Level and Flow	No. of Days	Percent
Current	Reservoir level above 435 ft (no pumping)	863	34
	Reservoir level between 411 and 435 ft, flows under 26 cfs (no pumping)	504	20
	Reservoir level between 411 and 435 ft, flows over 26 cfs (pumping required)	627	25
	Reservoir level under 411 ft (pumping required)	530	21
Future: Folsom Prison Delivery Capacity Up to 10 cfs	Reservoir level above 437 ft (no pumping)	779	31
	Reservoir level between 413 and 437 ft, flows under 33 cfs (no pumping)	673	27
	Reservoir level between 413 and 437 ft, flows over 33 cfs (pumping required)	482	19
	Reservoir level under 413 ft (pumping required)	590	23

* Based on flow and reservoir level data for the 2,524 days between 01/01/2001 and 11/29/2007

4.5 Increased Power Consumption

Increases in power consumption were analyzed using the 2001-2007 flow and reservoir level data provided by Reclamation. Since the analysis in Section 4.4.2 above indicates that anticipated future demands are not likely to increase the number of days when pumping is required to satisfy Natoma Pipeline demands, the power consumption analysis was limited to North Fork Pipeline requirements.

The following assumptions were made when estimating current and future power use:

- For each "pumping required" day (1,779 days out of 2,524 days of record for "current conditions" in the North Fork pipeline), the power used by the pumps in operation was calculated assuming that the average daily flow was maintained for 24 hours:

  Power in kW-hours = 24 hours x (Flow Rate) x (Head) / (Efficiency x 11.81)

- For future conditions, demand up 25 percent, the average daily flow rates recorded between January 1, 2001 and November 29, 2007 were increased by 25 percent; reservoir levels remained the same. This increased the number of "pumping required" days from the 1,779 days in the "current" column to 2,153 out of 2,524 days, as shown in Table 4-4.
- For future conditions, SJWD demand up 25 percent, Roseville demand up 67 percent, the average daily flow rates recorded for the 2001-2007 period were increased by the appropriate percentages; reservoir levels remained the same. This increased the number of "pumping required" days from the 1,779 days in the "current" column to 2,202 days out of 2,524 days of record, as shown in Table 4-4.
- Pump efficiencies of 0.80 for variable speed pumps and 0.75 for constant speed pumps were assumed. These efficiencies are easily attainable when pumps are operated within their normal operating ranges.

- The selection of pumps to be operated for a given combination of demand and reservoir water level was based on the assumptions that: pumps would operate within acceptable ranges specified in Table 3-4; valves on pumps 6, 7, and 8 would be throttled as needed; and pumps 7 and 8 would be operated together. See Appendix F for full pump selection schedule.

Results of the power consumption analysis are summarized in Table 4-6.

Table 4-6 Calculated Current and Future Power Usage for Deliveries to the North Fork Pipeline*

	Current (2001-2007)	Deliveries up 25%	Deliveries up 25% for SJWD, 67% for Roseville
Number of Days Operating:			
2 Variable Speed Pumps	781	343	337
2 Variable Speed Pumps + Pump 6	469	333	353
1 Variable Speed Pump + Constant Speed Pumps	529	1,477	1,512
Total	1,779	2,153	2,202
Average Annual Power Consumption in MW-hours (based on daily power calculations for 6.9 years)	5,670	12,872	14,477
Future Power Use as Percent of Current	100%	227%	255%

* Based on flow and reservoir level data for the 2,524 days between 01/01/2001 and 11/29/2007

A 25 percent demand increase would more than double power consumption. This would occur because at higher demands, the less efficient constant speed pumps would be used more frequently and the more efficient variable speed pumps would be used less frequently (see Appendix F to better understand how moving to a higher demand affects pump selection).

4.6 Conclusions about Impacts of Higher Demands

Increased delivery volumes would:

- Increase flow velocities, resulting in higher energy losses due to friction.
- Raise the gravity flow threshold for both the North Fork and the Natoma pipelines.
- Increase the number of days on which pumping is required to meet raw water demands.
- Cause the less efficient constant speed pumps to be used more frequently, and the more efficient variable speed pumps to be used less frequently.
- Substantially increase power consumption.
- Result in increased pump maintenance by accelerating the degradation of equipment that is already operating at low efficiencies under adverse hydraulic conditions.

5 EVALUATION OF POWER AND CONTROL SYSTEMS

Power and control systems were evaluated to assess their condition and operational limits. This section of the report presents:

- Basis of electrical and control systems evaluation
- Configuration of electrical and control systems
- Reliability of electrical and control systems
- Conclusions about power and control system reliability

5.1 Basis of Electrical and Control Systems Evaluation

The assessment of electrical power supply and control system reliability is based on observations made during a site visit on October 16, 2007,[11] and review of the drawings listed in Table 5-1 (included in Appendix E).

Table 5-1 Electrical and Control Drawings Reviewed

Drawing No.	Title	Author	Date
485-218-1093	Folsom Power Plant & Switchyard UHA Panel 2 Breaker 52-3 (312) Pumping Plant Feeder No. 1 Wiring Diagram	USBR	2/5/2007
485-218-1094	Folsom Power Plant & Switchyard UHA-Feeder 52-6 (612) Wiring Diagram	USBR	2/5/2007
485-218-1461	Folsom Pumping Plant Stand Pipe High Level – Pump Trip Control Schematic Diagram	USBR	3/14/2002
485-218-1470	Folsom Dam Pumping Plant Expansion Single Line Diagram	USBR	9/22/2005
485-218-1784	Folsom Switchyard Electrical Installation Switching Diagram	USBR	6/8/2007
485-218-1859	Folsom Pumping Plant Electrical Installation 208/120C Power Distribution System Single Line Diagram	USBR	10/22/2005
Pumping Plant Expansion E-1	Single line diagram	SAI Engineers	3/7/1997
Pumping Plant Expansion E-3	Pump 7 & 8 and Mov-14 & 26 Control Schematics	SAI Engineers	3/13/1997

[11] The site visit and review of drawings were conducted by Lawrence Lam, P.E. of YEI Engineers, Inc., a member of the consulting team responsible for this study.

5.2 Configuration of Electrical and Control Systems

5.2.1 Electrical System

Power Source

As indicated in Section 2 of this document, a double-ended substation supplies electrical power to the pumping plant's 4.16kV switchgear (designated "UHA"). In the double ended substation, switchgear UHA receives power from transformers KZ4A and KV9A. Switchgear UHA has two main breakers, 52-A connecting to transformer KV9A, and 52-B connecting to transformer KZ4A and a tie breaker, UHA5. The main breakers and tie breaker are electrically interlocked (see drawing number 485-218-1784). Switchgear UHA is connected to the pumping plant's switchgears 1 and 2. Switchgear 1 through transformer KPA provides power to 208/120V panel CPC. Switchgear 2 through transformer KPB provides power to 208/120V panel CPB. Both transformers KPA and KPB provide power to 208/120V panels CPA and CPD through an automatic transfer switch.

Transformers

There are two main transformers serving the pumping plant:

- Transformer KZ4A is three phase, 10/12.5MVA, 13.8(Y)-4.16(Y) kV; its primary voltage is 13.8kV, derived from a 220kV substation.
- Transformer KV9A is three phase 10/12.5MVA, 115(delta)-4.16(Y) kV; its primary voltage is 115kV.

There are two secondary transformers serving the internal loads in the pumping plant:

- Transformer KPA is three-phase, 75kVA, 4.16kV (delta)-208/120V; its primary voltage is 4.16kV, derived from a switchgear 1.
- Transformer KPB is three-phase, 75kVA, 4.16kV (delta)-208/120V; its primary voltage is 4.16kV, derived from a switchgear 2.

Switchgear UHA and Breakers

The primary voltages of transformers KZ4A and KV9A are different, and their power is supplied from different substations. By the electrical interlock of the main breakers 52-A, 52-B and tie breaker UHA5, switchgear UHA can supply power to its loads from either or both of its power sources.

Connecting Cables

Switchgear UHA is connected to the pumping plant's switchgear 1 and 2 through individual sets of three 1/C 500 kcmil cables. For switchgear 1, the total connected potential maximum load of pumps 2, 3, 4 and 8 appears to be 2,750 horsepower (HP) or about 382 Amps. For switchgear 2, the total connected load of pumps 5, 6 and 7 is 2,800 HP or about 389 Amps. Determination of the actual loading on the switchgears would require load testing; no load testing was performed for this evaluation.

Main Switchgear 1 and Switchgear 2

Main switchgear 1 receives power from breaker 312 of switchgear UHA. Main switchgear 2 receives power from breaker 612 of switchgear UHA. Switchgear 1 connects to pumps 2, 3, 4 and 7. Switchgear 2 connects to pumps 5, 6, 8 and the emergency pump (see drawing number 485-218-1784).[12] While switchgear 2 is connected to pumps 5, 6, 8, and the emergency pump, it does not provide power to all simultaneously: the emergency pump is designed to be used only when the Folsom Reservoir water level is too low to allow operation of the other pumps; the switchgear, therefore, is set up to provide power to either pumps 5, 6, and 8 (individually or together) or to the emergency pump alone.

5.2.2 Control System

Information about control system configuration was derived from two drawings. Pumping Plant Expansion Project drawing E-3 shows that VFD pumps (7 and 8) and motor operated valves (V26 and V14) are hard wired for start/stop or open/close control, interlock and remote monitoring. The VFD pumps share an 115VAC-24VDC control power supply. The VFD control and the PLC in the remote panel 1101 are mentioned in the drawing but no details are provided.

Drawing 485-218-1461 is a partial representation of a Pumping Plant Central Start Stop Control Schematic. The drawing indicates that a loss of 24V control power device 27CPC was installed (the device, however, is not shown on the drawing). How the Central Start Stop Control is connected to each pump start-stop control circuit is likewise not shown. The physical protection of these control circuits from central control to local pump could not be determined from information available.

5.3 Reliability of Electrical and Control Systems

5.3.1 Electrical System Reliability

Power Sources

From a system configuration standpoint, a setup with double-ended substations and independent power sources, such as the Folsom Dam Pumping Plant setup, is considered to be highly reliable. Simultaneous failure of both independent power sources is very unlikely. According to the Institute of Electrical and Electronics Engineers' *Survey of Reliability of Electric Utility Power Supplies to Industrial Plants*, if the two power sources from adjacent substations are considered utility circuits, the probability of losing both circuits is 0.312 per year (probabilities are generally expressed as a number between 0 and 1, with 0 representing no possible occurrence and 1 representing certain occurrence). The estimated downtime is 0.52 hours per failure, which equates to 0.1622 probable hours of downtime in a year (8,760 hours) and power availability 99.998 percent of the time.

Transformers

The two main transformers (located in the switchyard area), KZ4A and KV9A, are rated 10MVA each. Each of the transformers is capable of serving the whole plant with ample capacity. The

[12] Drawing 485-218-1470 shows a different power supply arrangement, with switchgear 2 connected to pumps 5, 6, 7 (instead of 8), and emergency pump. For purposes of this memorandum the more recent drawing number 485-218-1784 was assumed correct.

physical condition of the transformers is not known; if they were installed at the same time as switchgear UHA, the transformers would have been in service for 25 years. Since transformers generally have longer life expectancies than switchgear, their reliability can still be considered high.

Switchgear UHA and Breakers

Switchgear UHA is connected to two redundant power sources. It receives power from two separate transformers, which in turn derive their primary power from separate substations. Switchgear UHA provides the flexibility of supplying power to its loads by either or both of its power sources. For a UHA bus serving one group of pumps (pumps 2, 3, 4, 7 or pumps 5, 6, and 8/or emergency pump) to lose power, one of the following would have to happen:

- Both power sources lost, a highly unlikely occurrence, as indicated above.
- Failure of one transformer (KZ4A or KV9A) and simultaneous failure of tie breaker UHA5, a highly improbable occurrence.
- Failure of one main breaker (52-A or 52-B) and simultaneous failure of tie breaker UHA5, which is also highly unlikely.
- Failure of the bus section, the probability of which is also low.

The reliability of switchgear UHA, therefore, is high based on its configuration. On visual inspection, however, the switchgear UHA looked aged and rusted, presumably due to its outdoor location and extended exposure to the elements. It appeared to be in its late stages of useful life. This was confirmed by a subsequent check of UHA nameplates, which show a manufacturing date of April 1983. The likelihood of failure of 25-year-old equipment under normal operations or under fault conditions depends on the frequency and quality of maintenance, but can generally be expected to be high.

UHA feeder breakers are protected by over-current relay only (see drawings 485-218-1093 and 485-218-1094). This type of protection was typical for switchgear in the 1980's, but current standards would include a micro-processor-based multi-function relay. The multi-function relay can provide various protections and more information under fault. More information about a fault assists operation and maintenance personnel in identifying problems so that the system can be put back on line faster, thereby improving its reliability.

Connecting Cables

Age and condition of the cables and terminations are not known. Since failure of a cable can affect the availability of an entire section of the pumping plant, cables that are old and/or in poor condition would adversely impact the reliability of the pumping plant's power supply.

The reliability of cables is different for the two switchgears, as the total connected load of switchgear 1 is about 381 Amps and total connected load of Switchgear 2 is 389 Amps. Each is served by a set of 500 kcmil cable, which is rated for 380 Amps. Both sets of cable are marginally able to serve the full load .Unless there is some load diversity, i.e. not all the loads running at the same time, no more loads can be added to switchgear 1 or switchgear 2 through existing cables.

Main Switchgear 1 and Switchgear 2
Since main switchgear 1 is connected to UHA via breaker 312, failure of this breaker or the interconnecting cable D-D will cause main switchgear 1 and its associated pumps to lose power. Similarly, failure of breaker 612 or interconnecting cable E-E will cause main Switchgear 2 and its associated pumps to lose power. Under the existing system configuration, therefore, pumps would lose power upon failure of breakers or interconnecting cables. This switchgear configuration offers a low level of power supply reliability.

Detailed shop drawings of switchgears 1 and 2 were not available for review. The bus ratings for both switchgears are unknown. From field observation, however, switchgear 1 and switchgear 2 appear to be in fair condition (see Figures 5-1 and 5-2).

In the 1999 Pumping Plant Expansion Project, field modifications were made to change the switchgear bus from one section to two sections. Modifications of this kind are likely to adversely impact equipment reliability. Drawing E-1 of the Pumping Plant Expansion project indicates that main switchgears 1 and 2 were originally a single switchgear. The main breaker was removed and bypassed by bus bar jumpers. A section of bus bars was removed to create separation of switchgear 1 and switchgear 2. There is no connection that can tie switchgear 1 and switchgear 2 together. To provide redundancy and therefore increase reliability, a new tie (breaker) would be required so that the busses and associated loads of switchgears 1 and 2 can have access to power via either set of cables (assuming the switchgear busses are adequately rated, say for 800 Amps or above). Each set of cables would need to be upgraded to carry switchgear 1 and switchgear 2 loads, as well as the total load of the entire pumping plant.

Drawings indicate that switchgear 1 has a spare position for another pump starter. The position can be used to support pump additions (with appropriate cable upgrades).

Depending upon the load addition required to support increases in pumping capacity, switchgear 1 and switchgear 2 should be re-evaluated for further modification or replacement.

The reliability of the 208/120V power distribution system is somewhat better than the 4160V distribution system for the pumping loads. By using automatic transfer switch 2802, the 208/120V power panels CPA and CPD have access to power from both switchgear 1 and switchgear 2 (refer to Drawing 485-218-1859). The power service to the 208/120V loads, mainly HVAC blowers, large pump heaters, pump discharge valves, and control panels, is reliable.

5.3.2 Control System Reliability

The hard wire schematics for start/stop or open/close control, interlock and remote monitoring appear to have adequate reliability.

Since Pump 7 and Pump 8 share a control power supply, a malfunction or even a blown fuse can cause control power loss to both pumps. To improve reliability, a separate power supply should be provided to each pump control circuit.

Figure 5-1 Switchgear 1 and Switchgear 2 Front View

Figure 5-2 Switchgear 1 and Switchgear 2 Back View

5.4 Conclusions about Power and Control System Reliability

Switchgear UHA should be replaced with new, modern switchgear to significantly improve the reliability of power supply to the pumping plant. To maintain 100 percent redundancy and thereby achieve high reliability, the new breakers replacing the existing breakers 312 and 612 should have adequate rating to handle the total existing pumping plant loads (Pumps 2 to 8) plus any new pump motor loads. Microprocessor-based multi-functional relays are recommended for the new switchgear.

New cable feeders from Switchgear UHA to pumping plant main switchgear should be further evaluated. Replacement of cable and termination would improve overall system reliability at relatively moderate cost and little disruption to plant operations.

The configuration of the pumping plant's main switchgear 1 and switchgear 2 should be changed to provide redundancy and improve power supply reliability. Under the existing system configuration, pumps would lose power upon failure of breakers or interconnecting cables.

6 Recommended Actions and Their Approximate Costs

Improvements suggested in the hydraulic performance and power/control system evaluation sections of this document are summarized in this section. Approximate implementation costs are presented. Improvements are described in order of increasing cost, followed by an assessment of their impacts on system reliability, ability to meet increased demands, and energy usage. The following improvements and their impacts are discussed:

- Development and adoption of new SOP
- Replacement of discharge valves
- Upgrades of power supply
- Pump replacement
- Reconfiguration of pump intake
- Combinations of recommended actions
- Impact of recommended actions

6.1 Development and Adoption of New SOP

Standard operating procedures for the Folsom Pumping Plant could be revised to utilize pumps more efficiently. More efficient utilization (i.e., operation of pumps within manufacturer-recommended ranges) would reduce maintenance requirements and prevent further pump damage.

Adoption of a new SOP without changing valves or pumps is "recommended" only to the extent that it would help prevent further pump damage caused by operating them outside prescribed ranges. Adoption of such an SOP clearly does not resolve capacity issues, use power efficiently, or remedy the hydraulic deficiencies inherent in the configuration of the suction piping.

6.1.1 Basics of New SOP

A pump-selection schedule was developed (see Appendix F) that can form the basis for a new SOP. The schedule tries to retain current pump operating practices to the extent possible:

- Select pumps based on reservoir level and total pumping demand.
- Operate variable speed pumps only within 50 and 75 percent of total speed.
- When two variable speed pumps are operated together, operate one manually and place the other one in automatic mode; do not operate either pump above 65 percent of total speed when operated together.
- Start pumps against closed valves and allow discharge valves to open fully (except as noted below).

The proposed procedures differ from current practices in two important ways:

- Pumps would be operated only within the range of total dynamic heads (TDH) suggested by the manufacturer. At the Folsom Pumping Plant the TDH is approximately equivalent to the elevation difference between the reservoir and the North Fork Pipeline surge tank.
- Limited valve-throttling is proposed, to keep pumps within proper TDH ranges when the required lift is low (i.e., when the reservoir water level is above 405 feet). Throttling is proposed only for pumps 6, 7, and 8, which are equipped with butterfly valves suitable for operation in a partially open position.

The proposed pump selection schedule (Appendix F) would maintain the pressure in the North Fork Pipeline under the maximum operating level of its main surge tank (which has an overflow elevation of 479.3 feet). The target water level in the surge tank would be set at:

- Elevation 435 feet for reservoir water levels of 405 feet and lower.
- Elevation 455 feet for reservoir levels between 405 and 430 feet.

The pump selection schedule does not consider pumping at reservoir levels above 430 feet. In order to keep pumps within their operating ranges at such high reservoir water surface elevations, surge tank levels would have to be set close to the overflow point, creating a high potential for spills.

The new Natoma Pipeline surge tank has a maximum operating level of 434 feet and overflows at 436 feet. If the North Fork surge tank is operated with a water level at 455 feet while pumps are used to deliver water to the City of Folsom and the Folsom Prison, the Natoma Pipeline surge tank would have to be isolated to prevent overflows. Alternatively, various valves could be throttled to keep levels in the Natoma surge tank below the maximum operating level.

The proposed pump selection schedule consists of a simple set of spreadsheets (included in Appendix F). An operator would locate the total demand and the reservoir level, and the best pump combination for that set of conditions would be listed.

Deviation from the pump selection schedule would be necessary for demands higher than 220 cfs with reservoir levels at 410 feet or higher (see Figure 3-3 or spreadsheets in Appendix F). The only way to meet demands under those conditions would be to use the constant speed pumps well outside their normal operating ranges.

6.1.2 Costs Associated with New SOP

If the new SOP is implemented in-house (based on the pump selection schedule in Appendix F) and the training of operators is done by the SOP developers, there would be no external costs. No equipment purchases are proposed under this action.

6.2 Replacement of Discharge Valves

Five of the existing eight pumps have gate valves on their discharge lines. Butterfly valves on the discharge lines would allow throttling.

As established in Section 3 of this document, current practice at the pumping plant is to operate with discharge valves fully open, regardless of the most efficient operating range of each pump. The only way to keep pumps within their efficient, manufacturer-recommended operating ranges is through valve throttling.

6.2.1 Description of Valve Improvements

Changing over to butterfly valves on discharge lines would require:

1. Removal and disposal of existing valves.
2. Modification of discharge piping; the extent of the modifications would depend on the size difference between old and new valves.
3. Installation of new butterfly valves with electric actuators.
4. Installation of pressure sensors on suction and discharge piping.
5. Purchase and installation of a new programmable logic controller (PLC), set to operate valves to open as far as necessary to keep the differential pressure at the pump's optimum head.
6. Incorporation of new controls into overall pumping plant controls.

Valve replacement work can be performed during periods when gravity flows are possible. There would be no disruptions of water deliveries.

6.2.2 Costs Associated with New Valves

Approximate costs of purchasing and installing new valves are presented in Table 6-1. Soft costs (engineering, financing, pre- and post-construction costs) and internal Reclamation costs are not included.

Table 6-1 Valve Replacement Costs

Description	Unit	Quantity	Unit Cost	Total Cost
Removal and disposal of existing valves	LS	1	$3,500	$3,500
Discharge pipe modifications	EA	4	$3,000	$12,000
18-inch butterfly valve w/ elec. motor actuator	EA	1	$17,000	$17,000
24-inch butterfly valve w/ elec. motor actuator	EA	3	$26,000	$78,000
Pressure sensors	EA	8	$2,000	$16,000
New valve controls	LS	1	$50,000	$50,000
Reconfiguring plant controls	LS	1	$5,000	$5,000
Subtotal Direct Construction Cost				**$181,500**
Mobilization/demobilization 5% of Subtotal				$9,100
Subtotal				$190,600
General Contractor's General Conditions, OH&P @ 10%				$19,100
Subtotal				$209,700
Design development & estimating contingencies (20%)				$41,900
Estimated construction cost				$251,600
Construction contingency (25%)				62,900
Estimated Field Cost (FC), in 2011 Dollars				**$314,500**

6.3 Upgrades of Power Supply

The improvements described in this section are independent of changes to controls associated with new valves and/or new pumps. The power supply improvements can be implemented individually or together. Implementation of all improvements at the same time would minimize plant disruption and reduce costs.

6.3.1 Description of Power Supply Upgrades

Three upgrades are suggested:

- Replacement of switchgear UHA with new, modern switchgear with microprocessor-based, multi-functional relays; new switchgear would be designed to accommodate new pumps as required.
- Replacement of feeder cable and termination at switchgear UHA.
- Improve reliability of plant's main switchgear 1 and switchgear 2: add a new tie (breaker) so that the busses and associated loads of switchgears 1 and 2 can have access to power via either set of cables (assuming the switchgear busses are adequately rated, say for 800 Amps or above). Upgrade each set of cables to carry switchgear 1 and switchgear 2 loads, as well as the total load of the entire pumping plant.

6.3.2 Costs Associated with Power Supply Upgrades

Approximate costs are presented in Table 6-2. Upgrades can be implemented individually or together. Soft costs (engineering, financing, pre- and post-construction costs) and internal Reclamation costs are not included.

Table 6-2 Cost of Power Supply Upgrades

Item	Estimated Cost	Assumptions/Comments
Replacement of switchgear UHA	$600,000	Implementation of all changes together would reduce total costs an estimated 20%
Cable and termination replacement	$200,000	
Improvement of plant's main switchgear	$2M	

6.4 Pump Replacement

Replacement of constant speed pumps with variable speed pumps would reduce power consumption and allow efficient pump operation at all times. Different combinations of pump replacement are possible. Only combinations that would be capable of satisfying the maximum future demands were considered.

6.4.1 Description of Pump Replacement Alternatives

Many combinations of pump replacement are possible. The following are suggested for further consideration:

- Replace pumps 2, 3, 4, and 5 (four pumps) with three variable speed pumps rated for 75 cfs each. Pumps 6, 7, and 8 to remain in place. Although Pump 6 is a constant speed pump, its butterfly discharge valve allows operation at the appropriate TDH.
- Replace pumps 2, 3, 4, 5, and 6 (five pumps) with four variable speed pumps rated for 75 cfs each. Pumps 7, and 8 to remain in place.
- Replace pumps 2, 3, 4, 5, and 6 (five pumps) with five variable speed pumps rated for 60 cfs each. Pumps 7, and 8 to remain in place.

Total pumping plant capacities for existing pumps and suggested replacement alternatives are presented in Table 6-3.

Pump replacement work can be performed during periods when gravity flows are possible. There would be no disruptions in water deliveries.

Table 6-3 Pumping Plant Capacity for Various Pump Replacement Scenarios

	Rated* Pump Capacities (cfs)			
Pump	**Existing**	**Three New Pumps**	**Four New pumps**	**Five New Pumps**
2	20	-	-	60
3	50	75	75	60
4	40	75	75	60
5	40	75	75	60
6	80	80	75	60
7	87	87	87	87
8	87	87	87	87
Total	**404**	**479**	**474**	**474**

* "Rated" pump capacities represent flow rates at maximum operating efficiency.

6.4.2 Costs Associated with Pump Replacement

Approximate pump replacement costs are presented in Table 6-4 for the "Four New Pumps" alternative. Unit costs are provided to facilitate estimating the costs of the three-pump and five-pump alternatives. Soft costs (engineering, financing, pre- and post-construction costs) and internal Reclamation costs are not included.

Table 6-4 Pump Replacement Costs (Four New Variable Speed Pumps)

Description	Unit	Quantity	Unit Cost	Total Cost
Removal and disposal of (4) existing pumps	LS	1	$7,000	$7,000
New pumps/drivers	EA	4	$600,000	$2,400,000
New VFDs	EA	4	$300,000	$1,200,000
New suction valves	EA	4	$24,500	$98,000
New butterfly discharge valves	EA	4	$24,500	$98,000
Sensors and piping modifications	EA	4	$5,000	$20,000
New pump/valve controls	LS	1	$50,000	$50,000
Reconfiguring plant controls	LS	1	$5,000	$5,000
Subtotal Direct Construction Cost				**$3,878,000**
Mobilization/Demobilization 5% of Subtotal				$193,900
Subtotal				$4,071,900
General Contractor's General Conditions, OH&P @ 10%				$407,200
Subtotal				$4,479,100
Design Development & Estimating Contingencies (20%)				$895,800
Estimated Construction Cost				$5,374,900
Construction Contingency (25%)				1,343,700
Estimated Field Cost (FC), in 2011 Dollars				**$6,718,600**

The approximate costs for the 3- and 5-pump alternatives would be:

- Three new 75 cfs variable speed pumps: $5.1M
- Five new 60 cfs variable speed pumps: $8.4M

6.5 Reconfiguration of Pump Intake

The cost of reconfiguring the pump intakes is impossible to determine until a new configuration is selected. The three-dimensional flow patterns from suction manifold to pumps are very complex and make analytical design methods unsuitable. A physical model of the pump intakes would be required to properly analyze approach flow patterns and arrive at a satisfactory design.

The cost of physical model tests would vary depending on the extent of the model, its scale, and the complexity of the testing program. Modeling costs are likely to be in the range of $150,000 to $200,000.

The reconfigured intake could take many shapes. One would be a modified manifold with strategically placed metal guide vanes on the approaches to each suction pipe and inside of each suction pipe. Another possibility would be a pressurized sump that provides evenly distributed flow to each pump intake. In either case, a temporary pumping plant bypass would have to be provided and the work scheduled for one gravity-flow period; opening valves V1, V2, V5, and V10 and closing V3 would allow use of the discharge piping as a bypass with limited capacity.

The physical model study would identify hydraulic deficiencies and might also identify a "quick fix." In that case construction costs might be in the hundreds of thousands of dollars. If the model study could only identify complex redesigns, construction costs could run above $5M.

6.6 Combinations of Recommended Actions

The actions described above must be evaluated individually, although implementation of several or all of them together is also feasible:

- *Development and adoption of a new SOP:* The current SOP could be revised while keeping the same equipment. If new valves and/or pumps are to be installed, development of the new SOP should be delayed to incorporate details of the operation of the new equipment.
- *Installation of new butterfly valves:* The new valves would include automated controls to keep existing pumps within acceptable operating ranges. The existing SOP would have to be revised upon valve installation.
- *Power supply upgrades:* The upgrades could be implemented while keeping the existing equipment. Their implementation would make more sense, however, as part of a pumping plant refurbishing that included new pumps, valves, and controls.
- *Installation of new variable speed pumps:* New valves are assumed with the new pumps. Power supply upgrades would be necessary as well, as the new pumps would increase the total power demand at the plant. A new SOP would be needed.
- *Pump intake reconfiguration:* A new intake configuration would improve the efficiency of the existing pumps only if they are operated within acceptable ranges; new discharge valves

and a new SOP would be required along with the intake reconfiguration; pumps could remain as they are, and minor power supply upgrades would suffice.

6.7 Impacts of Recommended Actions

The effects of the recommended actions are summarized in Table 6-5.

Table 6-5 Impacts of Recommended Actions

Action	Impacts
Develop new SOP	For existing equipment: • Would reduce maintenance needs, prevent undue pump/valve wear • Would NOT reduce power consumption • Would NOT increase capacity or system reliability For new valves and pumps with power system upgrades: • Would reduce maintenance needs, prevent undue pump/valve wear • Would reduce power consumption • Would improve system reliability
New discharge valves	• Would stop cavitation damage of constant speed pumps • Would reduce maintenance needs, prevent undue pump/valve wear • Would NOT reduce power consumption but rather increase it • Would make 404 cfs capacity attainable without pump damage • Would increase system reliability by operating at best pump efficiency
Power supply upgrades	• Would improve reliability of power supply to the pumping plant • Would maintain one hundred percent redundancy • Would modernize plant
New pumps	• Would reduce power consumption • Would increase pumping capacity • Would increase system reliability • Would reduce long term maintenance needs
Reconfigured intake	• Would improve pump efficiency • Would reduce maintenance needs, prevent undue pump/valve wear

APPENDIX A
References

DRAWINGS

Pumping Plant Expansion Drawings

Drawing No.	Title	Author	Date
T-1	Title Sheet, Vicinity and Location Map and Drawing List	SAI Engineers	3/7/1997
A-1	Building floor and roof	SAI Engineers	3/11/1997
A-2	Building elevations	SAI Engineers	3/7/1997
S-1	Civil/structural demolition plan	SAI Engineers	2/26/1997
S-2	Foundation plan	SAI Engineers	2/26/1997
S-3	Building addition, roof framing plan	SAI Engineers	2/26/1997
S-4	Building sections	SAI Engineers	2/26/1997
S-5	Pump foundation plan	SAI Engineers	2/26/1997
S-6	Miscellaneous details	SAI Engineers	2/25/1997
E-0	Legend, abbreviations and general notes	SAI Engineers	3/7/1997
E-1	Single line diagram	SAI Engineers	3/7/1997
E-2	Three line diagram	SAI Engineers	3/7/1997
E-3	Pump 7 & 8 and Mov-14 & 26 Control Schematics	SAI Engineers	3/13/1997
E-5	Electrical demolition plan	SAI Engineers	3/7/1997
E-6	New electrical equipment and grounding plans	SAI Engineers	3/7/1997
E-7	Switchgear 1 and 2 Sections and Details	SAI Engineers	3/13/1997
E-8	Raceway plan and Switchgear 1 and 2 elevation	SAI Engineers	3/7/1997
E-9	Lighting and power plans and panel schedule	SAI Engineers	3/7/1997
M-1	Mechanical demolition plan and sections	SAI Engineers	2/21/1997
M-4	VFD room AC plan	SAI Engineers	2/21/1997
M-5	Pump area sections	SAI Engineers	2/21/1997

Miscellaneous Drawings

Drawing No.	Title	Author	Date
	Natoma Raw-Water Pipeline Phase B	Camp Dresser & McKee Inc.	April 2000
	S.N.T.W.P.T. San Juan Suburban Water Treatment District - Raw Water Pipeline	Clendenen Engineers	9/30/1986
	Pipelines for Sidney N. Peterson Water Treatment Plant	Clendenen & Associates	1976
	Construction of Natoma Pipeline - Phase A - Vol 2 - Drawings	Robert Miles	8/7/1998
	Construction of Natoma Standpipe Relocation – Vol 2 - Drawings (Conformed to Addendum No. 1)	Robert Miles	March 2007
C-1	Construction of Natoma Standpipe Relocation – Sheet C-1	Robert Miles	8/21/2007
C-4	Construction of Natoma Standpipe Relocation – Sheet C-4	Robert Miles	8/21/2007
C-4	Construction of Natoma Standpipe Relocation – Sheet C-8	Robert Miles	8/21/2007
485-208-603	North fork pipe line by-pass and regulating valve	USBR	Unknown
485-208-846	Emergency pumping system general plan and installation	USBR	July 1992
485-208-852	Emergency pumping plan electrical installation	USBR	July 1992
485-208-854	Emergency pumping system 36 inch pipe installation in valve unit	USBR	6/7/1992
485-208-855	Emergency pumping system standpipe	USBR	6/4/1992
485-208-942	San Juan and Roseville pipeline plan and profile	USBR	9/1/1987
485-208-950	Natoma distribution box and 42" butterfly valve profile, details and sections	USBR	11/1/1988
485-208-951	Natoma regulating system and 42" butterfly valve schematic	USBR	11/1/1988
485-208-953	Natoma waterline remote control electrical installation	USBR	11/1/1988
485-208-980	Folsom Dam Pumping Plant Pumping Unit No. 6	USBR	10/16/1989
485-208-1147	General plan and tap installation	USBR	2/13/1992
485-208-1149	Emergency pumping system 84" pipe tap installation	USBR	3/4/1992
485-218-688	Folsom Pumping Plant Water Distribution Flow Diagram	USBR	8/8/1991

Drawing No.	Title	Author	Date
485-218-1093	Folsom Power Plant & Switchyard UHA Panel 2 Breaker 52-3 (312) Pumping Plant Feeder No. 1 Wiring Diagram	USBR	2/5/2007
485-218-1094	Folsom Power Plant & Switchyard UHA-Feeder 52-6 (612) Wiring Diagram	USBR	2/5/2007
485-218-1461	Folsom Pumping Plant Stand Pipe High Level - Pump Trip Control Schematic Diagram	USBR	3/14/2002
485-218-1470	Folsom Dam Pumping Plant Expansion Single Line Diagram	USBR	9/22/2005
485-218-1479	Bypass Pipe and Valve at Sta 10+90 Details	USBR	6/17/2000
485-218-1480	Bypass Pipe and Valve at Sta 10+90 Details	USBR	6/17/2000
485-218-1719	Folsom Dan Natoma Pipeline Phase A Plan & Profile Station 0+82 to 13+00	USBR	August 1998
485-218-1720	Natoma Pipeline Phase A Plan & Profile Station 13+00 to 25+00	USBR	August 1998
485-218-1721	Natoma Pipeline Phase A Plan & Profile, Station 25+0 to 37+00	USBR	August 1998
485-218-1722	Natoma Pipeline Phase A Plan & Profile Station 37+00 to 49+56	USBR	August 1998
485-218-1753	Roseville 60" raw water pipeline project cover sheet	USBR	6/30/2001
485-218-1754	Roseville 60" raw water pipeline project layout and notes	USBR	6/30/2001
485-218-1755	Roseville 60" raw water pipeline project abbreviations, symbols and general notes	USBR	6/30/2001
485-218-1756	Roseville 60" raw water pipeline project horizontal/vertical control and hydraulic profile	USBR	6/30/2001
485-218-1757	Roseville 60" raw water pipeline project Barton Road plan and profile	USBR	6/30/2001
485-218-1758	Roseville 60" raw water pipeline project Barton Road plan and profile	USBR	6/30/2001
485-218-1759	Roseville 60" raw water pipeline project Barton Road plan and profile	USBR	6/30/2001
485-218-1760	Roseville 60" raw water pipeline project Barton Road plan and profile	USBR	6/30/2001
485-218-1761	Roseville 60" raw water pipeline project Baldwin Reservoir plan and profile	USBR	6/30/2001
485-218-1762	Roseville 60" raw water pipeline project Baldwin Reservoir plan and profile	USBR	6/30/2001

Drawing No.	Title	Author	Date
485-218-1763	Roseville 60" raw water pipeline project Baldwin Reservoir plan and profile	USBR	6/30/2001
485-218-1764	Roseville 60" raw water pipeline project Auburn-Folsom road plan and profile	USBR	6/30/2001
485-218-1765	Roseville 60" raw water pipeline project Auburn-Folsom road plan and profile	USBR	6/30/2001
485-218-1766	Roseville 60" raw water pipeline project Facility tie-in details	USBR	6/30/2001
485-218-1767	Roseville 60" raw water pipeline project appurtenance details	USBR	6/30/2001
485-218-1768	Roseville 60" raw water pipeline project trench details	USBR	6/30/2001
485-218-1769	Roseville 60" raw water pipeline project pipeline details	USBR	6/30/2001
485-218-1770	Roseville 60" raw water pipeline project miscellaneous details	USBR	6/30/2001
485-218-1771	Roseville 60" raw water pipeline project Barton Road Tree Removal Plan	USBR	6/30/2001
485-218-1772	Roseville 60" raw water pipeline project Baldwin Reservoir tree removal plan	USBR	6/30/2001
485-218-1773	Roseville 60" raw water pipeline project Auburn-Folsom road tree removal plan	USBR	6/30/2001
485-218-1774	Roseville 60" raw water pipeline project tree information sheet	USBR	6/30/2001
485-218-1775	Roseville 60" raw water pipeline project test station installation	USBR	6/30/2001
485-218-1776	Roseville 60" raw water pipeline project test station & cable connection	USBR	6/30/2001
485-218-1777	Roseville 60" raw water pipeline project test station & cable connection	USBR	6/30/2001
485-218-1778	Roseville 60" raw water pipeline project traffic control plan	USBR	6/30/2001
485-218-1784	Folsom Switchyard Electrical Installation Switching Diagram	USBR	6/8/2007
485-218-1859	Folsom Pumping Plant Electrical Installation 208/120C Power Distribution System Single Line Diagram	USBR	10/22/2005
485-D-65	Steel penstocks plan and profiles	USBR	4/16/1951
485-D-1293	Main concrete dam typical sections	USCOE	6/1/1951
485-D-1294	Main concrete dam	USCOE	Unknown

Drawing No.	Title	Author	Date
485-D-1322	Folsom Dam North Fork Pipe Line Plan, Profile & Sections	USCOE	3/18/1954
485-D-1324	Folsom Dam Natoma Pressure Pipe Line Plan, Profile and Details	USCOE	6/25/1951
485-D-1354	Folsom Dam Pumping Plant Inlet Emergency Valve Installation	USCOE	8/15/1956
485-D-1551	Pumping Plant Equipment Mechanical Pump Installation	USCOE	9/11/1951
485-D-1570	Mechanical flow control & measuring equipment	USCOE	11/28/1952
485-D-1680	North fork Natoma water supply system flow diagrams	USCOE	4/25/1973
485-D-1826	Electrical installation surge tank	USBR	4/2/1969
485-D-1827	Roseville Water Service General Arrangement	USBR	9/25/1969
485-D-1828	Roseville Water Service Surge Tank and Standpipe Modifications	USBR	4/16/1969
485-D-1829	Roseville Water Service Meter Installation Plan & Section	USBR	7/24/2000
485-D-1831	Roseville Water Service Pressure Relief Station Plan and Sections	USBR	3/12/1973
485-D-1844	Meter Installation 42" Hinkle Pipe Line General Arrangement Location	USBR	2/5/1973
485-D-1847	Roseville/ San Juan flow control equipment schematic and wiring diagrams	USBR	6/17/1991
G-7	General Process Flow Schematic, Roseville Water Treatment Plant Phase III Expansion	City of Roseville Env. Utilities Department	Feb 2006
G-9	General Hydraulic Profile, Roseville Water Treatment Plant Phase III Expansion	City of Roseville Env. Utilities Department	Feb 2006
Sheets 40 to 51 of 57	Raw Water Pipeline, Contract II, Schedule C, City of Roseville Water Supply Facilities	Brown & Caldwell	Feb 1969
G-5	Schematic Flow Diagram and Hydraulic Profile, Sidney N. Peterson Water Treatment Plant	Clendenen & Associates	June 1977

DOCUMENTS

Title	Author	Date
Folsom Pumping Plant Training for Pumps 7 & 8 Operation, Preliminary Session, March 13, 2000 Agenda	Will B. Betchart	3/11/2000
Folsom Pumping Plant Capacity Evaluation, Attachment B-1: Maximum Bypass Capacity Through the Discharge Header Assuming Maximum Flood Control Water Surface	Will B. Betchart	12/31/2004
Folsom Pumping Plant Emergency Pump Test	Will B. Betchart	12/31/2004
Flow Data 2004	City of Roseville Water Treatment Plant	2004
Flow Data 2005	City of Roseville Water Treatment Plant	2005
Flow Data 2006	City of Roseville Water Treatment Plant	2006
Designer's Operating Criteria and Standard Operating Procedure, Folsom Dam Emergency Pumping Plant	Folsom Dam, American River Division, Central Valley Project, California	Unknown
Standpipe & Isolation Valve Structure, Natoma Pipeline	Folsom Water Treatment Plant	11/23/1999
Pump Test Data	Ingersoll-Dresser Pump Company	5/18/1998
San Juan Water District Water Treatment Plant Flows	San Juan Water District	January 2004 - October 2007
Folsom Dam Flow Data	USBR	January 2001 - November 2007
Folsom Pumping Plant Flows 2006-2007	USBR	2006 - 2007
Folsom Pumping Plant Delivery and Efficiency Data	Unknown, Provided by USBR	7/1/1994

APPENDIX B

Field Tests

B1. Head Loss Test Memorandum

B2. Pump Operation Memorandum

B3. Pump Power Consumption Tests

B1. Head Loss Tests Memorandum

WRE Water Resources Engineering, Inc.

MEMORANDUM

DATE: December 21, 2009

TO: Brian Zewe, US Bureau of Reclamation

FROM: Gustavo Arboleda, WRE

RE: Folsom Pumping Plant Capacity Evaluation
Field Testing on Raw Water Distribution System – Test Results

Background

Water Resources Engineering, Inc. (WRE) was retained by Bureau of Reclamation (Reclamation) to conduct an evaluation of the capacity of the Folsom Pumping Plant and associated water transmission pipelines. As part of the hydraulic evaluation of the system, WRE developed a computer model that replicates the hydraulic performance of pumps, valves, and pipes. The computer model uses one of the more advanced software packages available (Info Water by MWH Soft); its accuracy, however, depends on assumptions regarding energy losses.

Field test were conducted to measure actual energy losses during system operation. Tests were performed on April 29, 2009, following the previously devised and approved test plan attached as Appendix B1-1. This memorandum presents the results of the field tests.

Summary of Test Results

Head loss data are summarized in Figures 1 to 3. In addition to the test data points, Figures 1 to 3 include best-fit curves representing the head loss versus flow rate relationship for various segments of the piping system. These relationships were used to calibrate the computer model.

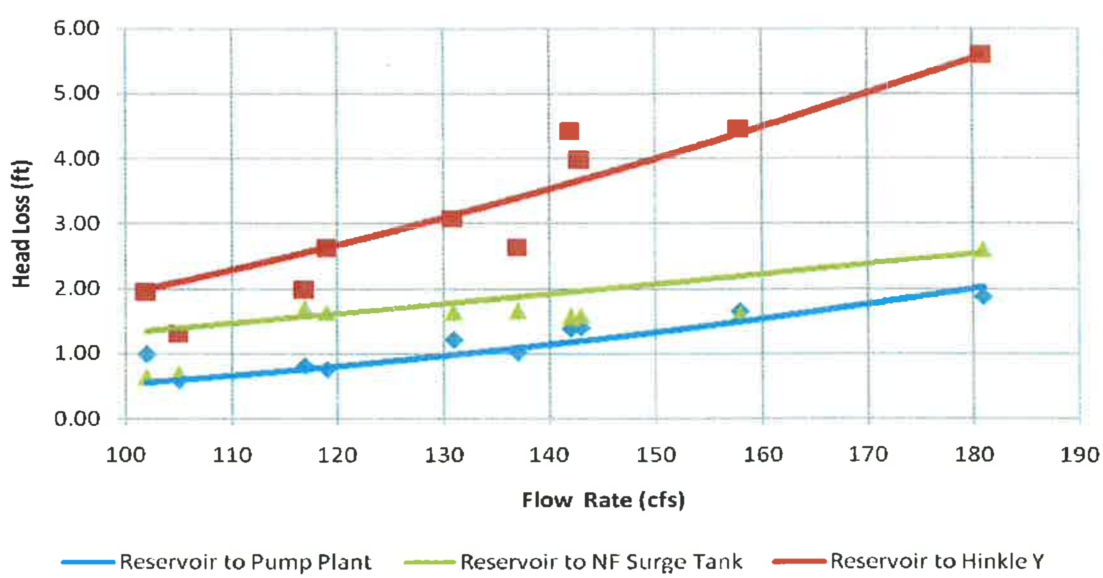


Figure 1. Head Losses in North Fork Pipeline

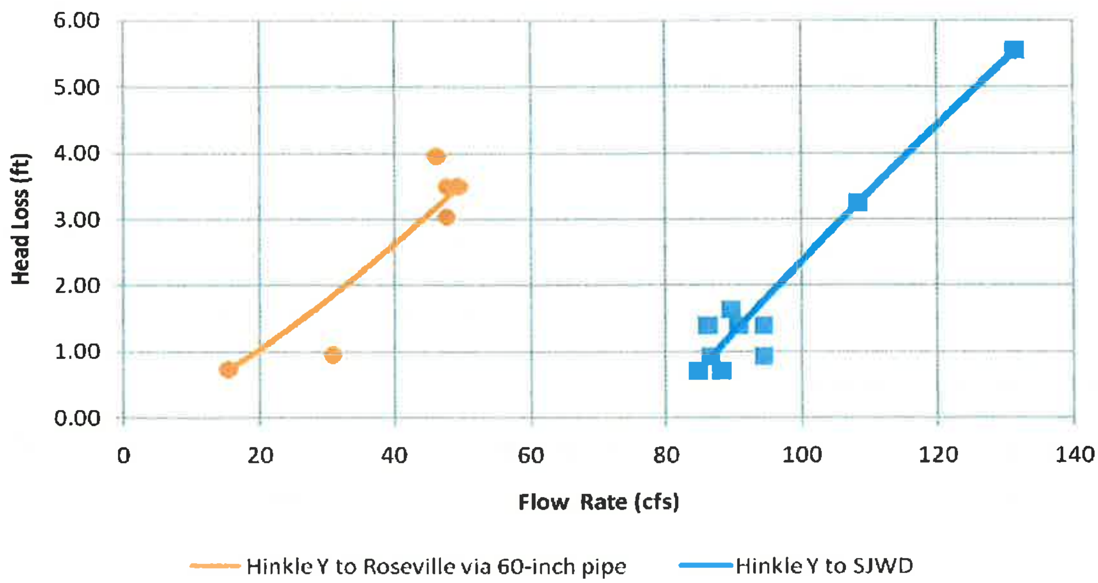


Figure 2. Head Losses in SWJD and City of Roseville Pipes

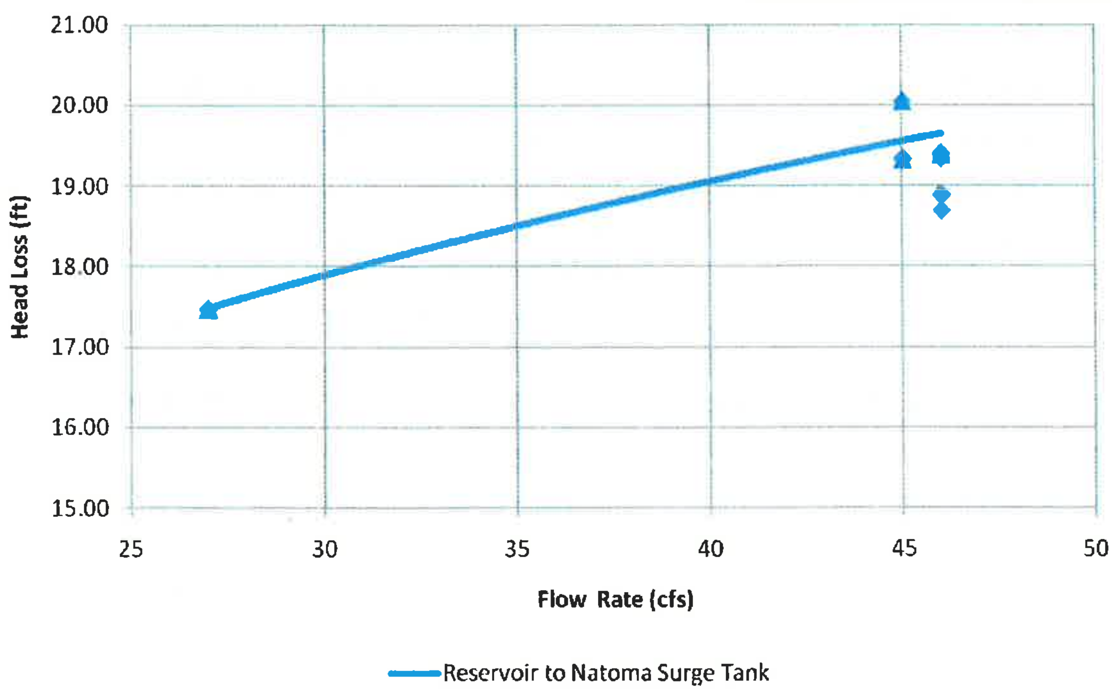


Figure 3. Head Losses in Natoma Pipeline

Field Test Procedures

Field tests were conducted by Reclamation's pumping plant operators with support from Reclamation and WRE engineers and with the collaboration of treatment plant operators at the San Juan Water District (SJWD), City of Roseville, City of Folsom, and Folsom Prison.

Tests consisted of setting flow rates at pre-determined levels, waiting for the system to stabilize, and then collecting 14 measurements: 6 flow meter readings and 8 pressure readings. Of the 8 pressure readings 5 were collected from the gages installed for the tests, and the other 3 from digital readouts at Reclamation's central controls. Table 1 summarizes measurement locations.

Table 1. Measuring Stations

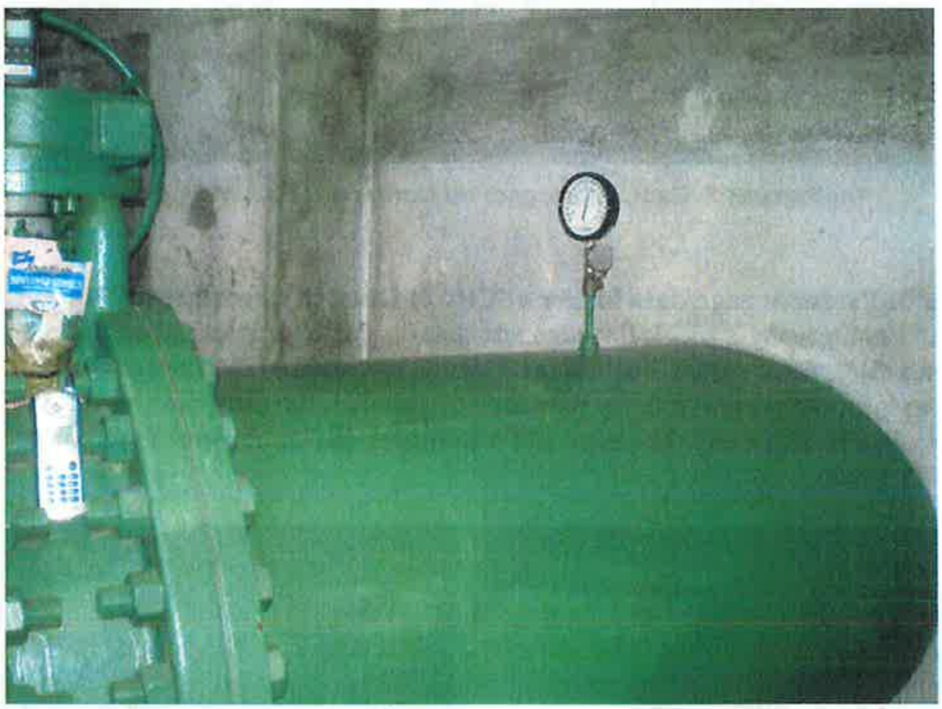
Station No.	Location	Parameter	Units*
1	Control room	Reservoir water surface elevation	Feet
2	Pumping plant, Pump 6 suction line	Pressure	psi
3	Control room	Water level on North Fork Pipeline surge tank	Feet
4	North Fork Pipeline at Hinkle Y, Pipeline Sta. 49+60	Pressure on North Fork Pipeline	psi
5	City of Roseville Treatment Plant	Pressure upstream of flow control valve	psi
6	SJWD Treatment Plant	Pressure upstream of flow control valve	psi
7	Control room	Water level on Natoma Pipeline surge tank	Feet
8	City of Folsom Treatment Plant	Pressure upstream of flow control valve	psi
9	Control room	Flow rate on Reclamation's North Fork Pipeline flow meter	cfs
10	Control room	Flow rate on Reclamation's Natoma Pipeline flow meter	cfs
11	City of Roseville Treatment Plant	Flow rate on City of Roseville's flow meter	MGD
12	SJWD Treatment Plant	Flow rate on SJWD's flow meter	MGD
13	Control room	Flow rate on Reclamation's flow meter on pipe to Folsom Prison	cfs
14	City of Folsom Treatment Plant	Flow rate on City of Folsom's flow meter	MGD

* psi: Pounds per square inch; 1 psi = 2.307 feet of head
cfs: cubic feet per second; 1 cfs = 448.8 gallons per minute = 7.48 gallons per second
MGD: million gallons per day; 1 MGD = 1.547 cfs

Pressure gages provided readings in terms of "psi" at the point of measurement. In order to calculate head losses, the psi were converted to feet of water above the gage. Adding the feet of water to the gage elevation provided a water surface elevation that could be compared to the reservoir water level and to readings from other gages. Gage elevations were determined as indicated below.

Station No. 2: Pressure gage/data logger on suction line to Pump 6
As shown in Photograph 1, the gage was installed approximately 10 inches above the Pump 6 suction pipe. Folsom Pumping Plant Drawing 485-208-980 (see Appendix B1-2) shows a pipeline centerline elevation of 314.75 feet and a 30-inch pipe diameter. The gage, therefore, was approximately at elevation $314.75 + ½(30/12) + (10/12) = 316.8$ feet.

A data logger was also installed at this location and 4 others. This electronic device continuously recorded pressures and stored readings every 10 seconds. The data logger at the Pump 6 suction line was 3.5 inches below the gage centerline, at an elevation of approximately 316.5 feet.



Photograph 1. Gage/data logger on Pump 6 suction pipe

Station No. 4: Pressure gage/data logger at Hinkle Y
As shown in Photograph 2, the gage was installed on the center of the North Fork Pipeline. The gage was roughly 40 feet upstream of the "Y" connection to SJWD pipelines. According to Reclamation Drawing 485D-1322 (see Appendix B1-2), the pipe centerline at the "Y" is at elevation 388.5 feet, and the pipe slopes up to the Y at 0.0052 feet/foot. The gage, therefore, was approximately at elevation $388.5 - (40 \times 0.0052) = 388.3$ feet. The data logger was approximately at 388.0 feet.

Photograph 2. Gage/data logger on North Fork Pipeline at Hinkle Y

Station No. 5: Pressure gage/data logger at City of Roseville Treatment Plant
As shown in Photographs 3 and 4, the gage and data logger were installed on the center of the City of Roseville's water supply line a short distance upstream of the flow control valve. According to information provided by the City of Roseville, the centerline elevation for the water supply line is 385.5 feet. The gage and data logger, therefore, were approximately at elevation 385.5 feet.

Photographs 3 & 4. Gage/data logger on City of Roseville's water supply line

Station No. 6: Pressure gage/data logger at SJWD Treatment Plant
The gage and data logger were installed over the 54-inch diameter influent pipe at the chemical feed vault (Photograph 5). SJWD measured the distance from the floor of the vault (elevation

402 feet) to the gage at 34.5 inches. The gage, therefore, was approximately at elevation 402.0 + (34.5/12) = 404.9 feet. The data logger was approximately at 404.6 feet.

Photograph 5. Gage/data logger in SJWD chemical feed vault

Station No. 8: Pressure gage at City of Folsom Treatment Plant
According to information provided by the City of Folsom, the gage was approximately at elevation 388.0 feet.

Test Conditions

Tests were initiated at 7 a.m. on April 29, 2009. The water level in the Folsom Reservoir was 448.2 feet at the beginning of the tests and at 448.1 feet at the end. No pumps were used; all deliveries were made by gravity. Due to unanticipated delays in a SJWD valve installation project, only one of the two lines from the Hinkle Y to SJWD was used (72-inch pipe, which reduces to 66-inch and then to 54-inch).

A total of 10 tests were performed, as follows:

1. City of Roseville operating at about 10 MGD with all flow through 48-inch pipeline (valve on 60-inch pipeline closed); other purveyors operating normally.
2. City of Roseville operating at about 20 MGD with all flow through 48-inch pipeline (valve on 60-inch pipeline closed); other purveyors operating normally.
3. City of Roseville operating at 30 MGD with all flow through 48-inch pipeline (valve on 60-inch pipeline closed); other purveyors operating normally.

4. City of Roseville operating at about 10 MGD with all flow through 60-inch pipeline (valve on 48-inch pipeline closed); other purveyors operating normally.
5. City of Roseville operating at about 20 MGD with all flow through 60-inch pipeline (valve on 48-inch pipeline closed); other purveyors operating normally.
6. City of Roseville operating at 30 MGD with all flow through 60-inch pipeline (valve on 48-inch pipeline closed); other purveyors operating normally.
7. SJWD operating at 70 MGD; other purveyors operating at normal capacities; the City of Roseville using only its 60-inch pipeline.
8. SJWD operating at 85 MGD; other purveyors operating at normal capacities; the City of Roseville using only its 60-inch pipeline.
9. All purveyors operating at normal capacities; the City of Roseville using only its 60-inch pipeline.
10. City of Folsom operating at a reduced capacity, other purveyors at normal capacities; the City of Roseville using only its 60-inch pipeline.

Test Data

The first set of readings was collected at approximately 7:15 a.m. and subsequent readings were collected at roughly half-hour intervals. A full set of readings was collected within a 10-minute span. The readings collected through visual inspection of the gages and digital readouts are presented in Appendix B1-3 and summarized in Figures 1 to 3.

The data loggers recorded readings every 10 seconds and captured pressure spikes produced during valve adjustments as well as small fluctuations. The data logger readings were analyzed for each test. Averaging data logger readings after the system stabilized resulted in the measurements presented in Appendix B1-3.

Test data were analyzed for accuracy and consistency. Adjustments were made where visual readings did not coincide with data logger output. The data logger readings were given preference over visual readings because of their higher accuracy. Since the data logger readings changed over time, the visual readings in some instances helped determine the time interval to be selected from the data logger readings.

Flow Rate Measurements

Flow rates were measured two different ways: on the North Fork Pipeline, separate readings were obtained from Reclamation's meter and from the meters at SJWD and City of Roseville treatment plants. On the Natoma Pipeline readings were obtained from Reclamation's meter and from the meters at the City of Folsom treatment plant and the pipe to Folsom Prison.

As would be expected given the timing of the readings and the accuracy of flow metering devices, there were some differences in the readings from separate sources. The differences, presented in Tables 2 and 3, ranged from less than one percent to close to seven percent.

Table 2. Flow Rate Differences in North Fork Pipeline

	Flow Rates in "cfs"				North Fork Pipeline Flow Meter Readings Greater than Sum of Purveyors' Readings by
Test	SJWD	Roseville	SJWD + Roseville	North Fork Pipeline	
1	83.5	15.3	98.8	105	6.30%
2	83.5	31.1	114.6	117	2.10%
3	83.5	46.4	129.9	137	5.50%
4	83.5	16.2	99.7	102	2.30%
5	83.5	30.9	114.4	119	4.00%
6	83.5	46.4	129.9	131	0.80%
7	108.3	46.1	154.4	158	2.30%
8	131.5	45.8	177.3	181	2.10%
9	92.8	46.7	139.5	143	2.50%
10	92.8	46.6	139.4	142	1.90%

Table 3. Flow Rate Differences in Natoma Pipeline

	Flow Rates in "cfs"				Natoma Pipeline Flow Meter Readings Less than Sum of Individual Readings by
Test	Folsom Prison	City of Folsom	Prison + City of Folsom	Natoma Pipeline	
1	2.9	44.9	47.8	46	3.70%
2	3.2	43.9	47.1	46	2.40%
3	3	43.6	48.2	45	6.60%
4	2.9	44.9	47.8	46	3.70%
5	3.2	45	48.2	45	6.70%
6	2.9	44.9	47.8	45	5.80%
7	3.2	44.9	48.1	46	4.30%
8	3.2	44.9	48.1	46	4.30%
9	3.1	43.5	46.6	45	3.40%
10	3.3	25.2	28.5	27	5.30%

Flow rates measured by Reclamation's meter on the North Fork Pipeline were consistently higher than the sum of the flow rates measured by the meters at SJWD and City of Roseville treatment plants, as shown in Figure 3.

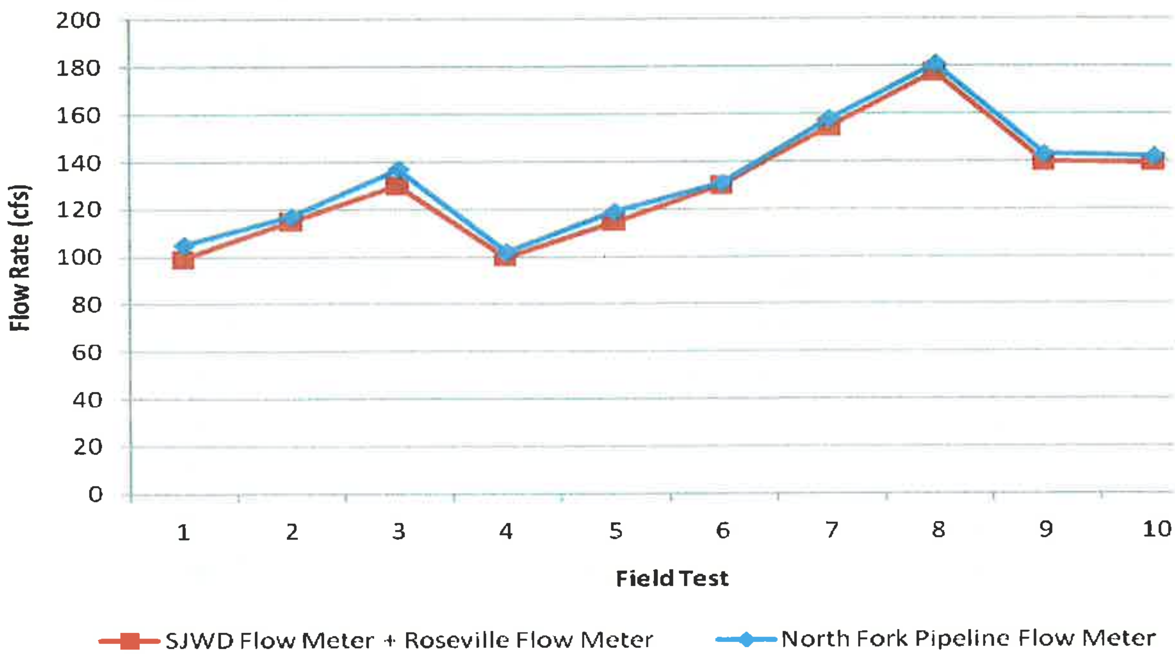


Figure 3. Flow Rate Measurements in North Fork Pipeline

Flow rates measured by Reclamation's meter on the Natoma Pipeline were consistently lower than the sum of the flow rates measured by the meters at the City of Folsom treatment plant and the pipe to Folsom Prison, as shown in Figure 4.

The test data presented in this memorandum used the flow rates measured by the water purveyors, for consistency. As indicated above, the differences between these readings and Reclamation's meters were relatively small. Use of either set of readings would not alter test findings.

WRE Water Resources Engineering, Inc.

DRAFT TEST PLAN - REVISED

DATE: April 1, 2009

TO: Brian Zewe, US Bureau of Reclamation

FROM: Gustavo Arboleda, WRE

RE: Folsom Pumping Plant Capacity Evaluation
Field Testing on Raw Water Distribution System

Background

Water Resources Engineering, Inc. (WRE) was retained by Reclamation to conduct an evaluation of the capacity of the Folsom Pumping Plant and associated water transmission pipelines. As part of the hydraulic evaluation of the system, WRE developed a computer model that replicates the hydraulic performance of pumps, valves, and pipes. The computer model uses one of the more advanced software packages available (InfoWater by MWH Soft); its accuracy, however, depends on assumptions regarding energy losses.

Field testing is the only reliable way of determining energy losses through the pumps, valves, and pipes of the distribution system. WRE, under its contract with Reclamation, was tasked to prepare a plan for a series of field activities that would allow the direct measurement of energy losses. The plan was initially submitted on January 15, for tests to be performed in February, when the Folsom Reservoir water level was low enough to require use of the pumping plant for raw water deliveries. The plan was revised on March 11 to delete tests on the Natoma Pipeline due to a pipeline collapse in late February.

Tests are now anticipated to be performed on April 29, 2009. A revised plan is required, as the Folsom Reservoir water level is currently at 442 feet and rising, precluding the use of pumps for raw water delivery. This document presents the newly revised plan.

Objective

The objective of the field testing is to collect data on energy losses between the reservoir and four end users: San Juan Water District (SJWD), the City of Roseville, Folsom Prison, and the City of Folsom. Specifically, the field testing will consist of recording pressures along the North Fork and Natoma pipelines for various rates of flow. Field measurements of energy losses will be used to refine the hydraulic model and verify its predictive abilities.

Preparatory Activities

Field tests will require a collaborative effort between Reclamation and raw water purveyors. The water treatment plants that receive Folsom Reservoir water will have to deviate from their normal operating procedures for the duration of the tests. A list of preparatory activities is presented below.

- **Set Test Date.** Reclamation has set a tentative test date of April 29, 2009.

- **Instrument Check.** The following pressure gages (or water level indicators) and flow meters will be used for the tests:
  - Folsom Reservoir water level indicator, reading water surface elevation in feet.
  - Pressure gage on suction side of pumping plant piping; piping is at a centerline elevation of 314.75 feet (see Figure 1, *Folsom Dam Raw Water Delivery System Schematic,* attached to this document); for a reservoir water level of 444 feet, the gage would read close to 129.25 feet (444 – 314.75 = 129.25) or 56 psi. Installing a digital gage such as the one illustrated in Figure 2 would greatly facilitate data collection.
  - Water level indicator on North Fork Pipeline surge tank, reading water surface elevation in feet.
  - Pressure gage on North Fork Pipeline at the Hinkle Y (at the location shown in Figure 3, attached, which is a short distance from the end of the North Fork Pipeline. The pipeline centerline elevation at the gage will need to be determined; the *Folsom Dam Raw Water Delivery System Schematic* shows a centerline elevation of 388.5 feet at the Y with the 42-inch pipe to SJWD, so the gage will be at an elevation slightly lower. For a reservoir water level of 444 feet, the static (no-flow) reading on the gage would be close to 55.5 feet (444 – 388.5 = 55.5) or 24 psi, depending on location. Installing a digital gage such as the one illustrated in Figure 2 would greatly facilitate data collection.
  - Pressure gages immediately upstream of the City of Roseville's end valves on their 48- and 60-inch pipelines. Shawn Barnes of the City of Roseville indicated on March 30 that readings from these gages were readily available from their electronic data acquisition system. Their datum (centerline pipe elevation at gage) will be provided by the City of Roseville.
  - Pressure gage immediately upstream of the San Juan Water District flow control valve. Bill Sadler of SJWD indicated on March 31 that readings from this gage are readily available from their electronic data acquisition system. Its datum (centerline pipe elevation at gage) will be provided by SJWD.
  - Water level indicator on new Natoma Pipeline surge tank, reading water surface elevation in feet.
  - Pressure gage immediately upstream of the City of Folsom flow control valve. Jim Bridges of the City of Folsom indicated on March 31 that the gage has not been calibrated recently but believes it can be checked by the April 29 tentative test date. Its datum (centerline pipe elevation at gage) will be provided by the City of Folsom.
  - Reclamation's flow meter on North Fork Pipeline, reading flow rate in cfs.
  - Reclamation's flow meter on Natoma Pipeline, reading flow rate in cfs.
  - City of Roseville's flow meters, reading flow rate in MGD.
  - SJWD's flow meter, reading flow rate in MGD.
  - Folsom Prison flow meter, reading flow rate in cfs.
  - City of Folsom flow meter, reading flow rate in MGD.
- **Prepare Data Sheets.** WRE will prepare the data sheets that will be used to record test data, and submit them to Reclamation for review and approval.

- **Test Procedure Review.** Reclamation pumping plant operators and SJWD, City of Roseville, City of Folsom, and Folsom Prison chief water treatment plant operators will review test procedures and confirm they are prepared to operate the system in accordance with these procedures on the scheduled test date.
- **Test Notification.** WRE will remind Reclamation pumping plant operators and SJWD, City of Roseville, City of Folsom, and Folsom Prison chief water treatment plant operators of impending tests 48 and 24 hours prior to testing.

Test Procedures

Tests will be directed by the test coordinator, either a Reclamation engineer or chief pumping plant operator, with support from WRE engineers. Reclamation valves referenced in the test procedures are shown in the system schematic attached at the end of this document. Purveyor valves and instrumentation are located in each water treatment plant and the respective plant operators will be responsible for their operation. Communication between the test coordinator and the treatment plant operators will be via cell phone.

The procedures outlined below assume that the reservoir water level will be at or above 444 feet and therefore the normal mode of delivering raw water will be by gravity flow. Based on data from previous years, anticipated normal rates of delivery at the end of April are in the order of 100 MGD to SJWD, 30 MGD to the City of Roseville, 35MGD to the City of Folsom and about 3 MGD to Folsom Prison. Test flow rates will stay within (i.e., will not exceed) the normal delivery rates.

Test procedures assume that:

- Reclamation valves will remain at their normal settings throughout the tests. Valve throttling to regulate flow rates will be done at the four end points by personnel from the respective water treatment plants.
- The raw water delivery system will be operating normally at the start of the testing, delivering water to the 4 purveyors via gravity flows.
- Pressure readings along the system will be made at indicated locations at the same time, or as close to it as practical (i.e., readings taken within 15 minutes of each other will be acceptable).

Tests will consist of the sequential steps listed below. WRE engineers will check each set of readings collected during a test step for "reasonableness" (falling within expected values) before proceeding to next test step.

Step	Approximate Time	Procedure
1	7:00 to 7:30 AM	Ask City of Roseville operators to set their flow rate to 10 MGD, all through the 48-inch pipeline (i.e., they should make sure the valve on the 60-inch pipeline is totally closed). Other water purveyors can maintain their normal settings. When flow stabilizes on Roseville's flow meter, record data from pressure gages and flow meters, listed below to facilitate referencing. • Folsom Reservoir water surface elevation • Pressure on gage on pumping plant suction header • Water level on North Fork Pipeline surge tank • Pressure on gage at Hinkle Y • Pressure immediately upstream of Roseville's flow control valve • Pressure immediately upstream of SJWD's flow control valve • Water level on Natoma Pipeline surge tank • Pressure immediately upstream of the City of Folsom flow control valve. • Flow rate on Reclamation's North Fork Pipeline flow meter. • Flow rate on Reclamation's Natoma Pipeline flow meter. • Flow rate on City of Roseville's flow meter. • Flow rate on SJWD's flow meter. • Flow rate on Folsom Prison's flow meter. • Flow rate on City of Folsom's flow meter.
2	7:30 to 8:00 AM	Ask City of Roseville operators to set their flow rate to 20 MGD, all through the 48-inch pipeline. Other water purveyors can maintain their normal settings. When flow stabilizes on Roseville's flow meter, record data from pressure gages and flow meters.
3	8:00 to 8:30 AM	Ask City of Roseville operators to set their flow rate to 30 MGD, all through the 48-inch pipeline. Other water purveyors can maintain their normal settings. When flow stabilizes on Roseville's flow meter, record data from pressure gages and flow meters.
4	8:30 to 9:00 AM	Ask City of Roseville operators to set their flow rate to 10 MGD, all through the 60-inch pipeline (i.e., they should make sure the valve on the 48-inch pipeline is totally closed). Other water purveyors can maintain their normal settings. When flow stabilizes on Roseville's flow meter, record data from pressure gages and flow meters

5	9:00 to 9:30 AM	Ask City of Roseville operators to set their flow rate to 20 MGD, all through the 60-inch pipeline. Other water purveyors can maintain their normal settings. When flow stabilizes on Roseville's flow meter, record data from pressure gages and flow meters.
6	9:30 to 10:00 AM	Ask City of Roseville operators to set their flow rate to 30 MGD, all through the 60-inch pipeline. Other water purveyors can maintain their normal settings. When flow stabilizes on Roseville's flow meter, record data from pressure gages and flow meters.
7	10:00 to 10:30 AM	Ask City of Roseville operators to resume normal operations. Ask SJWD operators to set their flow rate at 70 MGD. Other water purveyors can maintain their normal settings. When flow stabilizes on SJWD's flow meter, record data from pressure gages and flow meters.
8	10:30 to 11:00 AM	Ask SJWD operators to set their flow rate at 85 MGD. Other water purveyors can maintain their normal settings. When flow stabilizes on SJWD's flow meter, record data from pressure gages and flow meters.
9	11:00 to 11:30 AM	Ask SJWD operators to resume normal operations. Other water purveyors can maintain their normal settings. When flow stabilizes on SJWD's flow meter, record data from pressure gages and flow meters.
10	11:30 AM to 12:30 PM	Lunch Break
11	12:30 to 1:00 PM	Ask City of Folsom operators to set flow rate to about half of the "normal operations" flow rate. When flow stabilizes on City of Folsom's flow meter, record data from pressure gages and flow meters.
12	1:00 to 1:30 PM	Resume normal operations system-wide.

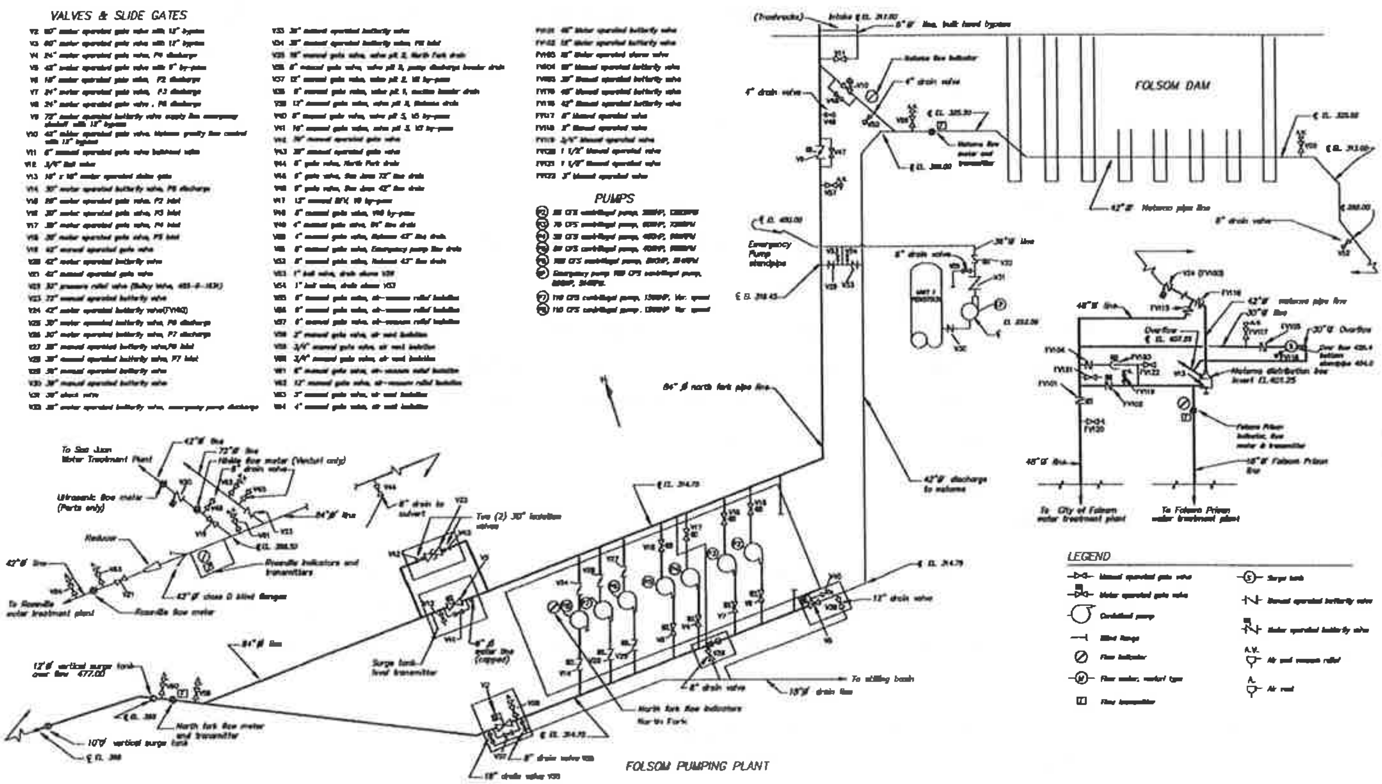


Figure 1. Folsom Dam Raw Water Delivery System Schematic
Source: Drawing 485-218-688, Folsom Pumping Plant Water Distribution Flow Diagram, USBR, August 8, 1991

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Ideal replacement for mechanical gauges—digital display makes reading measurements easy!

- Press the reset (Z-button) to zero readings
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What's included. one 9 V battery that provides power for up to two years; and an ABS case with either a brass or stainless steel bottom connection.

Gauge with brass fitting 68110-20

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Specifications & Ordering Information

Accuracy: ±1% full-scale
Dial size: 2½"
Operating temperature: 30 to 120°F (–1 to 49°C)
Media compatibility
Gauges with brass fitting: clean, noncorrosive gases compatible with brass and silicone
Gauges with stainless steel fitting: gases or liquids compatible with stainless steel
Process connection. ¼" NPT(M)
Power: one 9 V battery (included)
Display: four-digit LCD, 0.6"H
Dimensions
Gauges with brass fitting: 2½" diameter, 3½"H x 1¼"D
Gauges with SS fitting. 2½" diameter, 4¼" H x 1¼"D

Range	Resolution	Gauges with brass fitting		Gauges with stainless steel fitting	
		Catalog number	Price	Catalog number	Price
0 to 30" Hg	0.1" Hg	K-68110-00	[illegible]	—	—
0 to 30 psig	0.02 psi	K-68110-10	[illegible]	—	—
0 to 100 psig	0.1 psi	K-68110-20	[illegible]	K-68111-20	$160.00
0 to 200 psig	0.2 psi	—	—	K-68111-25	160.00
0 to 300 psig	0.2 psi	K-68110-30	[illegible]	—	—
0 to 500 psig	0.5 psi	—	—	K-68111-35	160.00
0 to 1000 psig	1 psi	—	—	K-68111-40	160.00
0 to 3000 psig	1 psi	—	—	K-68111-45	160.00
0 to 5000 psig	1 psi	—	—	K-68111-50	160.00

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Injection-molded case and 316 stainless steel wetted parts resist rust and corrosion

Gauges offer high accuracy plus the convenience of a digital display. The semiconductor sensing element measures pressure with ±0.25% accuracy and lasts for millions of cycles. Adjust zero and span potentiometers on front of case.

Models 68920-00 through -25 have a 3½-digit LCD and approximately 500 hours of battery life. Models 68920-30 through -74 have a four-digit LCD, peak hold function, TARE function to rezero the gauge, 16-minute auto-off function, and approximately 1000 hours of battery life.

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68920-38

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EG-17040-10 NIST-traceable pressure/vacuum certificate with five test points across range (5 to 5000 psi, 0 to 28" Hg) $104.00

Specifications & Ordering Information

CE

Accuracy: ±0.25% full scale
Dial size: 2½"
Operating temperature: 0 to 160°F (–17 to 71°C)
Media compatibility: gases or liquids compatible with 300-series SS
Process connection: ¼" NPT(M)
Power: one 9 V battery (included)
Display: 0.6"H
Dimensions: 2⅝" dia, 3⅝"H x 1⅜"D

Catalog number	Range	Resolution	Price
Gauges with 3½-digit LCD			
K-68920-25	0 to 30.0" Hg	0.1" Hg	$260.00
K-68920-00	0 to 19.99 psi	0.01 psig	390.00
K-68920-10	0 to 199.9 psi	0.1 psig	260.00
K-68920-20	0 to 1999 psi	1 psig	260.00
Gauges with 4-digit LCD and peak hold			
K-68920-30	0 to 30" Hg	0.1" Hg	416.00
K-68920-32	0 to 15 psi	0.01 psig	546.00
K-68920-34	0 to 30 psi	0.1 psig	416.00
K-68920-36	0 to 60 psi	0.1 psig	416.00
K-68920-38	0 to 100 psi	0.1 psig	416.00
K-68920-40	0 to 500 psi	0.1 psig	416.00
K-68920-42	0 to 1000 psi	1 psig	416.00
K-68920-44	0 to 4000 psi	1 psig	416.00
K-68920-48	0 to 10,000 psi	1 psig	416.00
K-68920-50	0 to 1000 mbar	1 mbar	481.00
K-68920-52	0 to 100 kPa	0.1 kPa	546.00
K-68920-54	0 to 200 kPa	0.1 kPa	546.00
K-68920-56	0 to 400 kPa	0.1 kPa	546.00
K-68920-58	0 to 700 kPa	1 kPa	416.00
K-68920-70	0 to 40 kg/cm²	0.01 kg/cm²	416.00
K-68920-72	0 to 100 kg/cm²	0.1 kg/cm²	416.00
K-68920-74	0 to 400 kg/cm²	0.1 kg/cm²	416.00

K-09376-04 Replacement batteries, 9 V.
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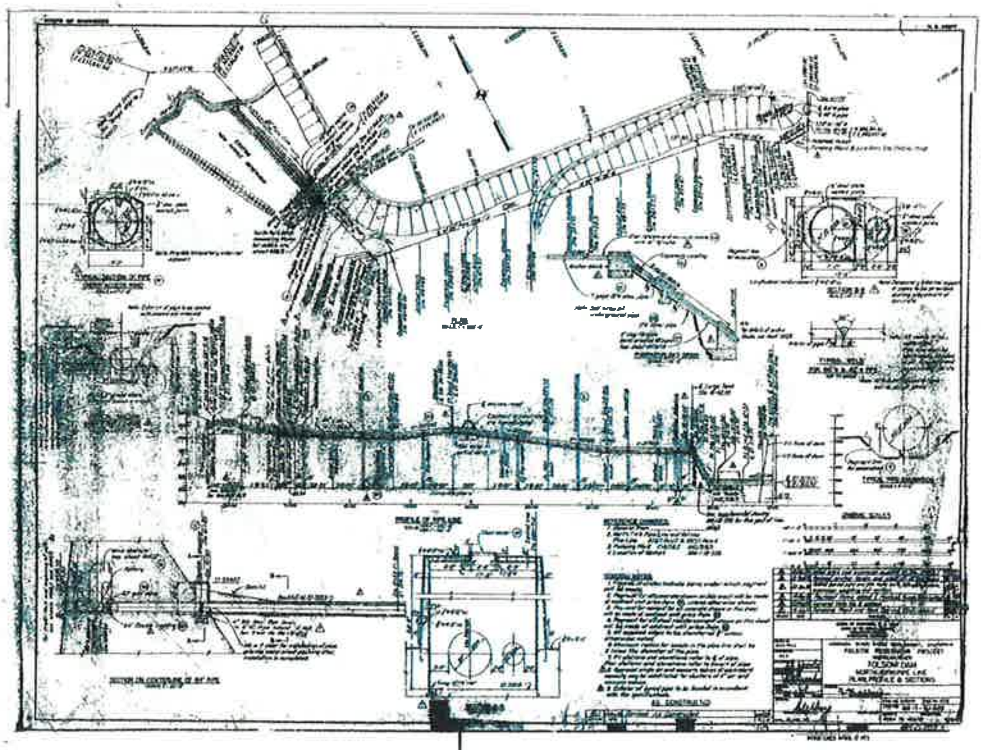
Figure 2. Sample Digital Pressure Gages

Figure 3. Pressure Gage Location at Hinkle Y

APPENDIX B1-2
Reference Drawings


FOLSOM DAM
PLAN, PROFILE & SECTIONS
AS CONSTRUCTED

DATA SHEET

Folsom Pumping Plant Capacity Evaluation
Field Testing on Raw Water Distribution System
April 29, 2009

Station	Unit	7:00-7:30 AM	7:30-8:00 AM	8:00-8:30 AM	8:30-9:00 AM	9:00-9:30 AM
1	ft	448.19	448.18	448.15	448.14	448.14
2	psi	57	57	57	57	57
3	ft	448	447	447	448	447
4	psi	26	25.5	25	26	25.5
5	psi	26.25	26	25.28	26	25
6	psi	16.5	15.5	16	16.1	16.1
7	ft	429.3	429.1	428.1	428.8	428.8
8	psi	8.5	8.5	8.5	8.5	8.5
9	cfs	105	117	137	102	119
10	cfs	46	46	45	46	45
11	MGD	9.9	20.1	30	10.51	20
12	MGD	54	54	54	54	54
13	cfs	2.9	3.2	3.0	2.9	3.2
14	MGD	29	28.4	28.2	29	29.1

KEY

1	Folsom Reservoir water surface elevation
2	Pressure on gage on pumping plant suction header
3	Water level on North Fork Pipeline surge tank
4	Pressure on gage at Hinkle Y
5	Pressure immediately upstream of Roseville's flow control valve
6	Pressure immediately upstream of SJWD's flow control valve
7	Water level on Natoma Pipeline surge tank
8	Pressure immediately upstream of the City of Folsom flow control valve
9	Flow rate on Reclamation's North Fork Pipeline flow meter.
10	Flow rate on Reclamation's Natoma Pipeline flow meter.
11	Flow rate on City of Roseville's flow meter.
12	Flow rate on SJWD's flow meter.
13	Flow rate on Folsom Prison's flow meter.
14	Flow rate on City of Folsom's flow meter.

DATA SHEET

Folsom Pumping Plant Capacity Evaluation
Field Testing on Raw Water Distribution System
April 29, 2009

Station	Unit	9:30-10:00 AM	10:00-10:30 AM	10:30-11:00 AM	11:00-11:30 AM	12:30-1:00 PM
1	ft	448.13	448.1	448.1	448.09	448.07
2	psi	57	57	57	57	57
3	ft	447	447	446	447	447
4	psi	25	25	24	25	25
5	psi	24	23.5	22.75	23.75	23.75
6	psi	16	14.1	12.5	16.1	15.5
7	ft	428.8	429.4	428.7	428.7	430.6
8	psi	8.5	8.5	8.5	8.5	7.9
9	cfs	131	158	181	143	142
10	cfs	45	46	46	45	27
11	MGD	30	29.8	29.6	30.2	30.1
12	MGD	54	70	85	60	60
13	cfs	2.9	3.2	3.2	3.1	3.3
14	MGD	29	29	29	28.1	16.3

KEY

1	Folsom Reservoir water surface elevation
2	Pressure on gage on pumping plant suction header
3	Water level on North Fork Pipeline surge tank
4	Pressure on gage at Hinkle Y
5	Pressure immediately upstream of Roseville's flow control valve
6	Pressure immediately upstream of SJWD's flow control valve
7	Water level on Natoma Pipeline surge tank
8	Pressure immediately upstream of the City of Folsom flow control valve
9	Flow rate on Reclamation's North Fork Pipeline flow meter.
10	Flow rate on Reclamation's Natoma Pipeline flow meter.
11	Flow rate on City of Roseville's flow meter.
12	Flow rate on SJWD's flow meter.
13	Flow rate on Folsom Prison's flow meter.
14	Flow rate on City of Folsom's flow meter.

APPENDIX B1-3
Field Test Data

Data Logger Summary Sheet
Folsom Pumping Plant Capacity Evaluation
Field Test on Raw Water Distribution System
April 29, 2009

		Average Pressure Readings (psi)			
		Pumping Plant Suction Header	**North Fork Pipeline at Hinkle Y**	**Uptream Side of Roseville Valve**	**Upstream Side of SJWD Valve**
Test	Time Range	**Sta. No 2**	**Sta. No. 4**	**Sta No. 5**	**Sta. No. 6**
1	7:15 - 7:20 A.M.	56.7	27.9	28.3	17.5
2	7:35 - 7:45 A.M.	56.6	27.6	28.0	17.3
3	8:10 - 8:20 A.M.	56.5	27.3	27.6	17.0
4	8:50 - 9:00 A.M.	56.5	27.6	27.5	17.5
5	9:15 - 9:25 A.M.	56.5	27.3	27.1	17.3
6	9:35 - 9:45 A.M.	56.4	27.1	25.6	17.1
7	10:00 - 10:10 A.M.	56.2	26.5	25.2	15.4
8	10:30 - 10:40 A.M.	56.1	26.0	24.7	13.9
9	11:10 - 11:20 A.M.	56.3	26.7	25.4	16.5
10	12:40 - 12:50 A.M.	56.2	26.5	26.5	16.4

Key

Test	Descriptions
1	Roseville operating at 10 MGD (15.5 cfs) with flows thru 48-inch pipeline
2	Roseville operating at 20 MGD (30.9 cfs) with flows thru 48-inch pipeline
3	Roseville operating at 30 MGD (46.4 cfs) with flows thru 48-inch pipeline
4	Roseville operating at 10 MGD (15.5 cfs) with flows thru 60-inch pipeline
5	Roseville operating at 20 MGD (30.9 cfs) with flows thru 60-inch pipeline
6	Roseville operating at 30 MGD (46.4 cfs) with flows thru 60-inch pipeline
7	SJWD operating at 70 MGD (108.3 cfs); Roseville using 60-inch pipeline
8	SJWD operating at 85 MGD (131.5 cfs); Roseville using 60-inch pipeline
9	All purveyors operating at normal; Roseville using 60-inch pipeline
10	Folsom operating at reduced capacity; Roseville using 60-inch pipeline

B2. Pump Operation Memorandum

WRE Water Resources Engineering, Inc.

MEMORANDUM

DATE: September 25, 2009

TO: Brian Zewe, US Bureau of Reclamation

FROM: Gustavo Arboleda, WRE

RE: Folsom Pumping Plant Capacity Evaluation
Field Tests – Variable and Constant Speed Pumps

Background

Task 3C of the Folsom Pumping Plant Capacity Evaluation, Field Monitoring and Investigation, calls for:

- Interviewing pumping plant operators regarding current operating practices;
- Investigating operational constraints on VFD pumps;
- Inspecting and observing pumps in operation, including constant speed pumps (#2, 3, 4, 5, 6) and pumps equipped with variable frequency drives, or VFDs (#7 and 8);
- Developing a pumping plant Standard Operating Procedure (SOP) for existing equipment.

This memorandum addresses the first three bullet points. The Standard Operating Procedure will be presented as a separate document.

Current operating practices, operational constraints, and field observations are summarized below. Conclusions are presented at the end of the document.

Current Operating Practices

Current operating practices were provided by Reclamation Senior Relief Operator Art Pakao and Control Operator Kenneth Zellner on Monday, September 21, 2009. Conversations with Kenneth Zellner regarding operating practices continued at the pumping plant through Wednesday morning, September 23. Current pumping plant operating practices are summarized below.

Assessing Demand: Operators get water demand information from four purveyors.

- Total demand = North Fork Pipeline demand + Natoma Pipeline demand*
- North Fork Pipeline demand = San Juan Water District (SJWD) demand + City of Roseville demand
- Natoma Pipeline demand = City of Folsom demand + Folsom Prison demand

* If Natoma Pipeline is supplied by gravity, pumping demand for Natoma Pipeline is 0.

Selecting Pumps: One of the VFD pumps (#7 or 8) is generally operated, along with one or more constant speed pumps, to meet total demand. One VFD pump is expected to deliver up to 85 cubic feet per second (cfs). Constant speed pumps are selected based on flow rate. Operators have labels on pump startup buttons, which read:
Pump 2 = 25 cfs
Pump 3 = 75 cfs
Pump 4 = 50 cfs
Pump 5 = 50 cfs
Pump 6 = 100 cfs

Activating Pumps: Pumps can be activated from the power plant control room. Current SOP, however, is to activate pumps from the controls in the pumping plant. A target North Fork Pipeline surge tank level is determined by looking it up on a table that relates surge tank levels to SJWD demand. A setting for the VFD is determined by looking it up on a table that relates surge tank level to VFD setpoint. All pumps start against a closed discharge valve and the valves are programmed to open slowly until fully open.

Operational Constraints

Operators indicated that the following constraints were applied to pumping plant operation:

- Pumps 7 and 8 not to be operated together.
- Pumps 7 and 8 to be operated only from 50 to 75 percent of maximum speed.
- Discharge valves on constant and variable speed pumps to be kept fully open during pump operation

Pumps 7 and 8 are normally operated individually in automatic mode. Pump controls are locked to prevent pump speed from increasing above 75 percent of full speed.

Field Observations

Monday September 21

Engineers Brian Zewe and John Robinson of Reclamation witnessed Monday's tests. Total demand on Monday morning, September 21, was about 210 cfs:

SJWD	94 cfs
City of Roseville	70 cfs
City of Folsom	43 cfs
Folsom Prison	3 cfs
Total	210 cfs

Operators used pumps 3, 4, and 7 to meet total demand.

Instruments at the pumping plant read as follows:

Reservoir level:	406.4 feet
Surge tank level:	443 feet
North Fork flow rate:	157 cfs

Pump 7 was at 67 percent of full speed. Discharge valves on pumps 3, 4, and 7 were fully open. No unusual noise or vibration were observed from pump 7 or its motor. Random crackling noises and intermittent knocking sounds were clearly audible on the discharge side of pumps 3 and 4.

Pump 3 was shut off and Pump 6 started. With pumps 4, 6, and 7 in operation, random crackling noises and intermittent knocking sounds were clearly audible on the discharge side of pumps 4 and 6.

Tests of Pump 7 at Low Speed: Pump 7 speed was manually lowered to 45 percent of maximum speed. Pumps 4 and 6 remained in operation. Discharge valves remained fully open. The water level in the surge tank came down to 436.1 ft and the total North Fork flow rate changed to 138 cfs. No unusual noise or vibration were observed from pump 7 or its motor. Random crackling noises and intermittent knocking sounds were clearly audible on the discharge side of pumps 4 and 6.

The speed of pump 7 was lowered to 40 percent of maximum speed. Pumps 4 and 6 remained in operation. Discharge valves remained fully open. The water level in the surge tank came down to 431.5 ft and the total North Fork flow rate changed to 130 cfs. No unusual noise or vibration were observed from pump 7 or its motor. Random crackling noises and intermittent knocking sounds were clearly audible on the discharge side of pumps 4 and 6.

Tuesday, September 22

Test on Pump 3 with Partially Closed Discharge Valve: The discharge valve on Pump 3 was closed slowly after the pump had been in operation for several hours. When the valve was about 50 percent closed, the discharge pressure went up about 10 psi (approximately 23 ft) and the random crackling noises and intermittent knocking sounds started to dissipate. When the valve was about 55 percent closed, the discharge pressure went up about 15 psi (35 ft) above the open-valve pressure and the noises were no longer discernible.

Wednesday, September 23

Tests scheduled for Tuesday were cancelled and later re-scheduled for Wednesday morning. Engineers Jay Emami and Brian Zewe of Reclamation witnessed the tests. Control Operator Kenneth Zellner operated the pumps. The water levels in the reservoir and surge tank were initially 406.17 ft and 442.9 ft, respectively. Pumps 2, 3, 4 and 7 were in operation with pump 7 set to automatic mode.

Tests of Pump 7 at High Speed: Technicians modified the lock on VFD controls to allow pumps to operate up to 80 percent of full speed. As the speed of Pump 7 was manually raised to 75 percent of maximum speed, the surge tank level went over 450 ft; the operator shut off Pump 4 and the surge tank level dropped to 443 ft.

With pumps 2 and 3 in operation and Pump 7 at 75 percent of maximum speed, random crackling noises and intermittent knocking sounds became apparent on the discharge side of the pump. The noises grew louder in intensity as the speed on Pump 7 was raised to 80 percent for a few seconds. The speed was then lowered to 65 percent and the noises disappeared.

Tests of Pump 8 at High Speed: Pump 7 was shut off and Pump 8 started. Pumps 2 and 3 remained in operation. Manually raising the speed of Pump 8 above 75 percent of maximum speed had the same results observed for Pump 7: random crackling noises and intermittent knocking sounds became apparent on the discharge side of the pump. The noises grew louder in intensity as the speed on Pump 8 was raised to 80 percent for a few seconds. The speed was then lowered to 65 percent and the noises disappeared.

Tests of Pumps 7 and 8 Operating Together: With pumps 2, 3, and 8 in operation, Pump 7 was started on manual operation. The speed on pumps 7 and 8 were raised and lowered to observe the effect of these changes:

- With one VFD pump at 65 percent speed or lower, raising the other VFD pump to 75 percent speed brought about the crackling noises on the discharge side of the pump operating at the higher speed.
- With both VFD pumps operating at 65 percent speed or slower, there were no unusual noises or vibration.
- The conditions above remained the same for the VFD pumps when pumps 2 and 3 were shut off, one at a time.

Conclusions

Current Operating Practices

Current operating procedures do not take into account the characteristics of the constant speed pumps (i.e., "pump curves") and their operating ranges. The constant speed pumps are selected for operation based on their rated flow capacity. To deliver its rated capacity, however, a pump requires a particular total dynamic head (TDH). At the appropriate TDH the pump would operate at peak efficiency and deliver the rated flow.

The TDH required for the constant speed pumps to operate at peak efficiency, based on pump performance curves provided by Reclamation, are presented in Table 1, below.

Table 1. Pump Flow Rates and Heads at Peak Efficiency

Pump	Peak Efficiency Flow Rate (cfs)	TDH (ft)	Peak Efficiency
2	20	100	88%
3	50	98	90%
4	40	84	88%
5	40	84	88%
6	80	86	87%

The pumps at the Folsom Pumping Plant were manufactured by Worthington Pumps. Worthington became part of Flowserve Corporation several years ago. Flowserve engineers responsible for Worthington Pumps were contacted to verify acceptable operating ranges for the Folsom Pumping Plant pumps. Application Engineer Stephen Phorwart (Flowserve facility in Rancho Dominguez, CA) indicated that their pumps can generally be expected to operate satisfactorily when run within 80 percent of their peak efficiency. Below that level of efficiency the pumps do not necessarily follow the pump curve and are subject to cavitation, recirculation, and uneven loading on moving parts that will significantly shorten pump life.

The acceptable operating ranges for the constant speed pumps, based on pump performance curves provided by Reclamation, are presented in Table 2, below.

Table 2. Operating Range for Constant Speed Pumps

Pump	Flow Rate (cfs) Range	TDH (ft) Range	Lowest Acceptable Operating Efficiency
2	10-28	60-124	70%
3	22-69	64-114	72%
4	18-51	50-116	70%
5	18-51	50-116	70%
6	41-106	50-106	70%

The TDH for Folsom Pumping Plant pumps is roughly represented by the difference in water level between the surge tank on the North Fork Pipeline and the reservoir. The tables used to set a surge tank level do not take into account the head requirements of the constant speed pumps. On Monday September 21, for example, the target surge tank level resulted in a TDH under 40 feet. This TDH allowed the VFD pumps to operate satisfactorily, but was well below the acceptable operating range for any of the constant speed pumps.

Had the reservoir level been around 370 feet, setting the surge tank at 443 feet, as it was on Monday September 21, would have resulted in a TDH of 73 feet. Pumps 2, 3, 4, 5 and 6 would have been able to operate within their acceptable range. VFD pumps would operate near their limiting 75 percent of full speed for that TDH.

Operators should consider TDH (rather than simply a target surge tank level) and the pumps' operating ranges. Prolonged operation outside of acceptable operating ranges damages the pumps and shortens their useful life.

Operational Constraints and Field Observations

Two of three operational constraints were proven by field tests to be justified:

- Pumps 7 or 8 should not be operated at speeds greater than 75 percent of full speed.
- Pumps 7 and 8 do not perform well when operated together.

A phenomenon known as "discharge recirculation" occurs when the VFD pumps are operated individually at greater than 75 percent speed or together at greater than 65 percent speed. Discharge recirculation causes cavitation pitting of the impeller resulting in poor pump performance (off the pump curve) and eventual mechanical failure. A more detailed description of discharge recirculation can be found at:

http://www.lawrencepumps.com/Newsletter/news_v04_i4_Apr07.html

The third constraint postulated initially, that VFD pumps should not be operated below 50 percent speed, was not confirmed by the field tests performed. Pump 7 was operated as low as 40 percent speed with no sign of discharge recirculation. It is very probable, however, that discharge recirculation will occur at lower-than-50 percent speeds if the VFD pumps are operated out of their efficient operating range.

Discharge recirculation was evident at the constant speed pumps operated during the field tests. The recirculation could be the result of operating the pumps well outside their prescribed efficiency range. It is possible, however, that the discharge recirculation would occur even when the pumps are operated within their prescribed operating range, as one of the contributing factors are the approach flow conditions, as explained below.

Likely Cause of Discharge Recirculation

Approach flow conditions are a major determinant of pump performance. Impellers are designed with the assumption that incoming flow will be evenly distributed throughout the approach section. A number of approach flow conditions have been determined through laboratory tests to be detrimental to impeller performance:

- Uneven flow distribution, where flow tends to favor one side over the other.
- Pre-rotation, where flow approaches the impeller with a circulatory pattern which may or may not be in the same direction that the impeller rotates.
- Vorticity, where a tight flow spiral forms immediately upstream of the impeller.

These conditions are generally a function of the geometry of the approach section. The approach geometry for all pumps at the Folsom Pumping Plant is likely to cause approach flow problems, even when pumps operate within acceptable efficiency ranges.

Possible Remedies

The only way to improve approach flow conditions is by changing the suction header configuration. And the only fail-safe way to develop an approach geometry that will provide acceptable flow conditions is through physical model tests.

"Base" tests on a physical model that replicates the current pumping plant configuration would confirm the causes of poor pump performance. Structural modifications can then be tested in the model until a configuration is arrived at that provides satisfactory flow conditions for all combinations of pumps in operation. Major changes to the configuration of the pumping plant intake are likely to be needed.

Even with a favorable approach flow, pumps will not perform well if operated at low efficiencies. A different set of operating procedures needs to be adopted to reduce the potential for pump damage.

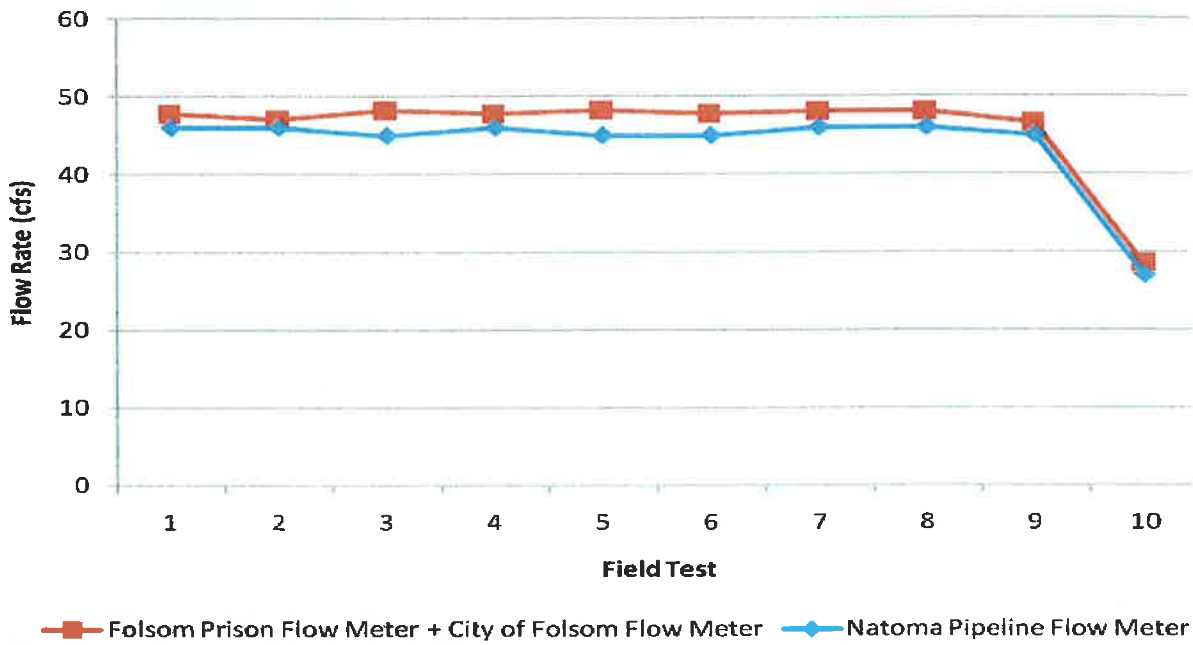


Figure 4. Flow Rate Measurements in Natoma Pipeline

APPENDIX B1-1
Test Plan

B3. Pump Power Consumption Tests

Pump Power Analysis

Field measurements were conducted on October 26, 2009 by Reclamation to estimate pump power consumption at the Folsom Pumping Plant. Power Quality Analyzers measured power applied to the constant speed pumps. For the variable frequency drive pumps (VFDs), operators recorded and averaged the power readings from the control panel.

Measurements

Power input was measured for each pump in the pumping plant. The Power Quality Analyzer provided continuous readings of voltage and current for several minutes, as illustrated in Figure A-1. A peaking factor was applied to the median voltage and current to derive power usage from:

Power (kW) = Median Voltage (volts) * Median Current (amps) * Peaking Factor / 1,000

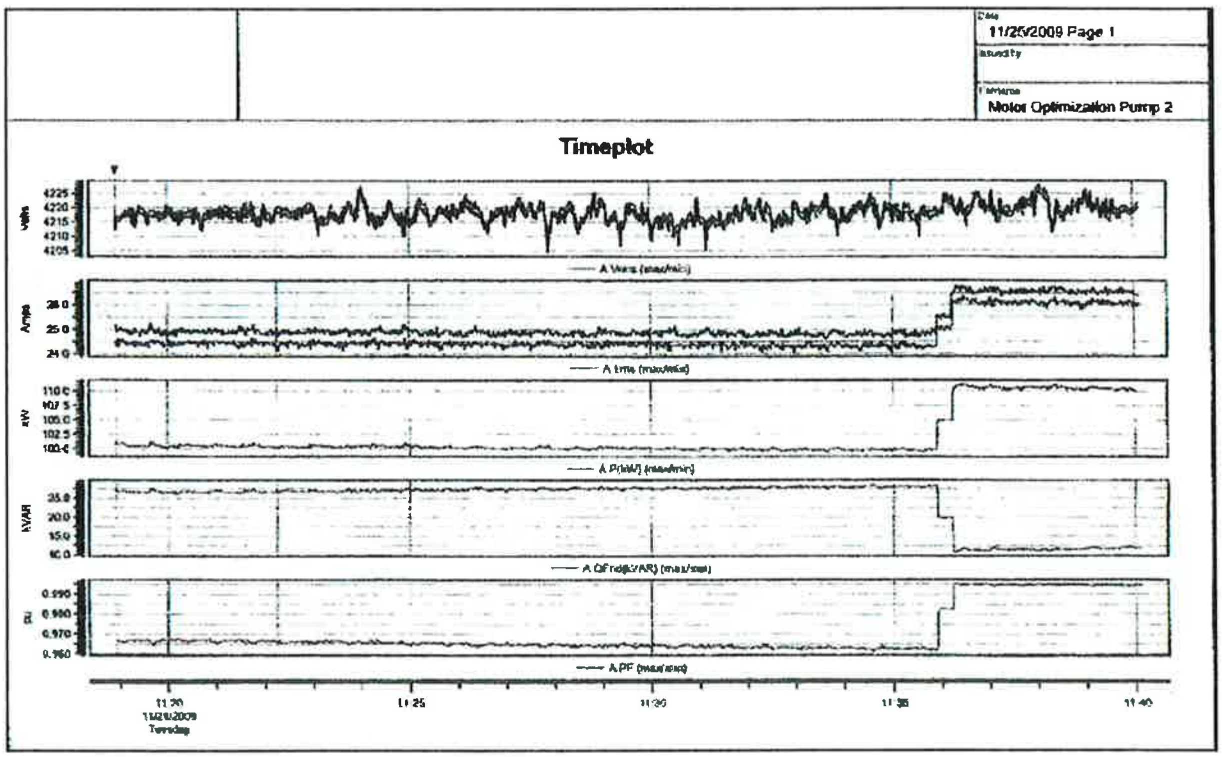


Figure A-1. Power Quality Analyzer Readings for Pump 2

Computed power usage derived from the pump tests are listed in Table A-1. The peaking factors were supplied by Reclamation.

Table A-1. Calculated Power Input from Pump Tests

Pump	Voltage (v)	Current (Amps)	Peaking Factor	Power (kW)	Power (HP)
2	4216.0	25.4	0.978	105	141
3	4195.0	69.4	0.963	281	377
4	4200.0	47.6	0.983	196	263
5	4219.0	47.2	0.949	190	255
6	4196.0	90.0	0.80	301	404
7				257	345
8				228	306

Other data related to pump operation were collected as well, as illustrated in Figure A-2 and summarized in Table A-2. The data for other pumps are shown on pages A-7 and A-8.

STarT **Test on Pump** 2 **Other pumps on** x, y, z
1458 **Time Start** = 0838 # 8, 4 aNd 2
Time End = 0848
S|D **Reservoir Level** = 397.84' ft
0945 **Surge tank level** = 441.9' ft 8 **VFD Speed** = 64%
North Fork flow = 105 cfs **Pump Disharge Pressure** = 50
Natoma flow = 31 cFs cfs

Figure A-2. Sample Record of Pump Test

Table A-2. Pump Operation Data during Power Tests

	Pump 2	Pump 3	Pump 4	Pump 5	Pump 6	Pump 7	Pump 8
Reservoir level (ft)	397.84	397.74	397.84	397.75	397.68	397.63	397.63
Surge tank level (ft)	441.9	441.8	442.1	441.8	441.8	442.7	441.8
North Fork flow rate (cfs)	105	116	103	110	111	90	92
Natoma flow rate (cfs)	31	31	33	30	37	37	34
Total flow rate (cfs)	136	147	136	140	148	127	126
Pumps in operation	8, 4, 2	8, 6, 3	8, 4, 2	8, 5, 3	8, 6	7, 3	8
VFD speed (% of max)	64	46	64	54	60	62	60
Pump outlet pressure (to nearest psi)	50	55	50	42	50	55	54
Pump Inlet pressure (to nearest psi)	35	35	35	35	35	35	35

Analysis

Each of the measurements made during the power tests is analyzed below for validity and accuracy. Pump efficiency is approximated based on power input measurements and flow and pump lift data.

Measured Inlet Pressures

Pressures measured by the gage upstream of the pump should reflect the water level in Folsom Reservoir less the energy losses in the suction piping, which should be small.

Measured inlet pressure (all tests) = 35 psi

Equivalent Head = 35 psi x 2.302 ft/psi = 80.6 ft

Hydraulic Grade = Gage elevation (approximately 317.8 ft) + 80.6 ft = 398.4 ft

This head is generally within a foot of reported reservoir water levels. Considering that gage readings were accurate only to about 0.5 psi or 1.15 feet, the pressure readings appear correct. They are not accurate enough, however, to assess pressure losses between the reservoir and the pump. For purposes of this analysis, those pressure losses can be neglected and the reservoir level assumed as the head upstream of the pump.

Measured Outlet Pressures

Pressures measured by the gage downstream of the pump should reflect the head gain or lift provided by the pump. These pressures should be slightly higher than the water levels in the surge tank, to account for head losses between pump and surge tank.

Table A-3. Comparison of Measured Outlet Pressures and Surge Tank Levels

	Pump 2	Pump 3	Pump 4	Pump 5	Pump 6	Pump 7	Pump 8
Surge tank level (ft)	441.9	441.8	442.1	441.8	441.8	442.7	441.8
Pump outlet pressure (to nearest psi)	50	55	50	42	50	55	54
Equivalent head in ft (psi x 2.302)	115.1	126.6	115.1	96.7	115.1	126.6	124.3
Hydraulic grade in ft (317.8 + Head)	432.9	444.4	432.9	414.5	432.9	444.4	442.1
Head loss (ft) between pump and surge tank	-9.0	2.6	-9.2	-27.3	-8.9	1.7	0.3

Table A-3 shows that measured outlet pressures in 4 out of 7 tests were lower than surge tank levels, which is physically impossible. This indicates that either the gage readings or the surge tank levels were recorded incorrectly. Pump efficiency calculations (discussed in "Pump Efficiency" section below) indicate that gage readings are more likely to be correct. For purposes of this analysis, the gage readings will be considered representative of the head downstream of the pump.

Measured Flow Rates

The flow rates recorded during the tests correspond to readings from flow meters on the North Fork and Natoma pipelines. There is no way to measure individual pump discharges at the pumping plant when there is more than one pump in operation.

Pump discharges can be approximated from pump performance curves provided by the pump manufacturer. These curves provide a relationship between total head or lift and pump discharge. For constant speed pumps there is a single performance curve. For pumps with VFDs, there is a separate performance curve for each pump speed.

The pump lifts measured during the power tests (Table A-4) are outside the normal range of operation of the constant speed pumps (Pumps 2, 3, 4, 5, and 6). These pumps are rated for lifts (i.e., total dynamic heads) from 84 to 100 feet; lifts during tests were all less than 50 feet.

Table A-4. Measured Pump Lifts and Pump Ratings

	Pump 2	Pump 3	Pump 4	Pump 5	Pump 6	Pump 7	Pump 8
Pump inlet pressure (to nearest psi)	35	35	35	35	35	35	35
Pump outlet pressure (to nearest psi)	50	55	50	42	50	55	54
Measured pump lift (psi)	15	20	15	7	15	20	19
Measured pump lift in ft (psi x 2.302)	35	46	35	16	35	46	44
Pump rated head (ft)	100	98	84	84	86	20-85	20-85
Pump rated flow (cfs)	20	50	40	40	80	18-90	18-90

Since pumps with VFDs (Pumps 7 and 8) were operated within acceptable ranges, pump curves (Figure A-3) were used to approximate the flow rate through these pumps. The remainder of the flow rate indicated by flow meters was assumed to be provided by the other pump(s) in operation. Where there was more than one constant speed pump in operation, they were assumed to contribute to the total flow rate in the same ratio as their rated capacities. For example, where pumps 2 and 4 were operating together, the flow rate for pump 4 was assumed to be twice that of pump 2. Estimated test flow rates are presented in Table A-5.

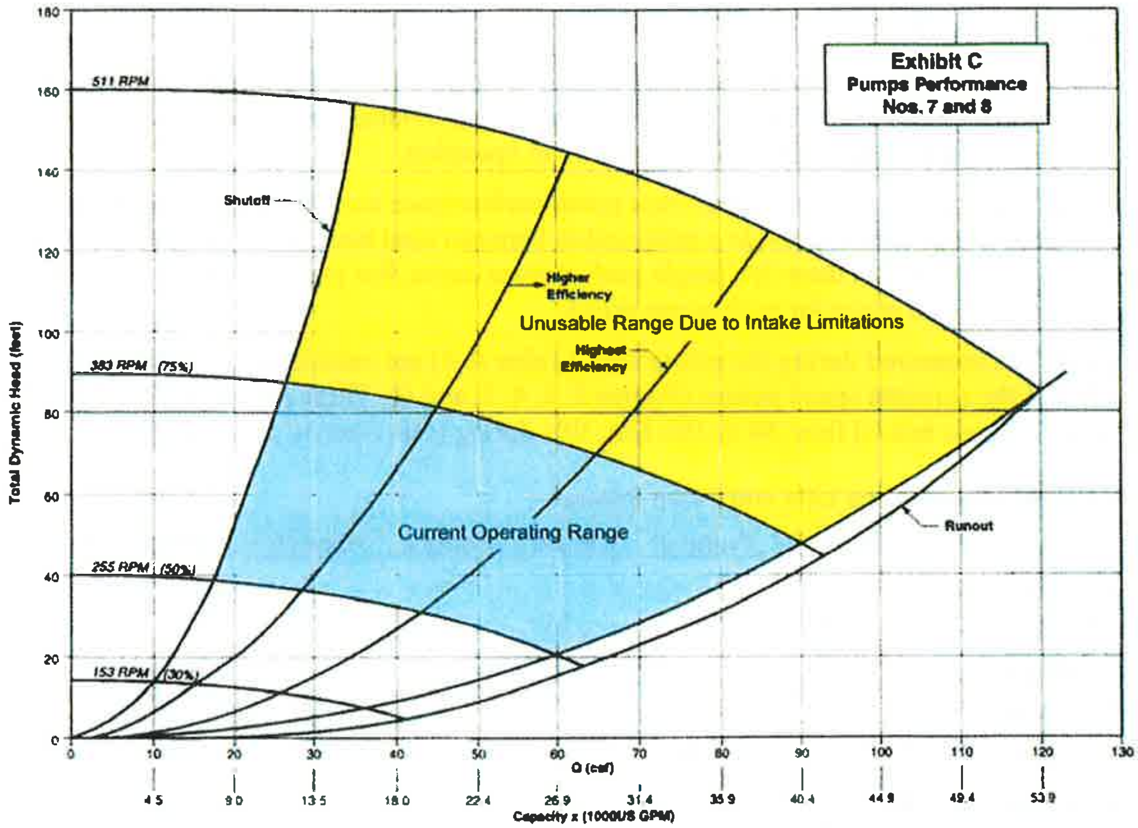


Figure A-3. Pump Performance Curves for Pumps 7 and 8

Table A-5. Estimated Test Flow Rates

Pumps Operating	Total Q (cfs)	Measured Lift (ft)	VFD Speed (% of Max)	VFD Pump Flow Rate (cfs)	Constant Speed Pump Flow Rates (cfs)	
2, 4, 8	136	35	64	P8= 75	P2= 20	P4= 41
3, 6, 8	147	46	64*	P8= 60	P3= 33	P6= 54
3, 5, 8	140	16	54	P8= 60	P3= 44	P5= 36
6, 8	148	35	60	P8= 70	P6= 78	
3, 7	127	46	62	P7= 55	P3= 72	
3, 8**	126	44	60	P8= 50	P3= 76	

* Test data sheet showed a speed of 46%. Pump curves indicate, however, that the measured lift is not possible at that speed. The most likely speed based on pump curves is 64%.

**Test data sheet showed pump 8 operating alone. Pump 8, however, does not have the capacity to deliver 126 cfs. The pump that was operating during the previous test (pump 3) was assumed to be still in operation.

Pump Efficiency

Pump efficiency is defined as the ratio of the water power to the power provided from a power source. The "water power" is the power added to the flowing water through the pump's rotating element; this power does not account for mechanical energy losses. The power provided from a power source is the measured power input; this power includes losses in the pump as well as mechanical losses from the bearings and seals and leakage.

Efficiency = Water Power / Measured Power Input

Water Power in hp = (pump lift in feet) * (flow rate in cfs) / 8.82

Note that 1 kW = 1.341 hp

Computed pump efficiencies based on power input measurements are listed in Table A-6.

Table A-6. Pump Efficiencies Based on Power Tests

Pump	Flow Rate (cfs)	Head (ft)	Water Power (HP)	Measured Power Input (HP)	Pump Efficiency
2	20	35	79	141	56%
3	33	46	172	377	46%
4	41	35	163	263	62%
5	36	16	65	255	26%
6	78	35	310	404	77%
7	55	46	287	345	83%
8	50	44	249	306	82%

The computed pump efficiencies indicate:

- Pumps 2, 3, and 4 show very low efficiencies; this is consistent with the fact that these pumps were operating outside their normal range (pumps are rated for lifts ranging from 84 to 100 feet; they were operated at lifts ranging from 35 to 46 feet).
- Pump 5 shows a very low efficiency; this is consistent with irregularities observed during the tests (the power measurements would not stabilize) and could be related to the very low lift (16 ft), well below the pump's normal operating range (pump 5 is rated for 84 ft).
- Pump 6 was operated close to its rated flow rate of 80 cfs but did not show its peak efficiency due to the low lift (operated at 36 feet, rated for 86 feet).
- Pumps 7 and 8 were operating at a reasonable efficiency, as the pump lifts were within their normal operating range.

Pump Power Analysis Field Record

10/26/09

Measuring Power Input at Folsom Pumping Plant

Scan to Ken

START 1458
Test on Pump 2
Time Start = 0838
Time End = 0848
S/D 0945
Reservoir Level = 397.84' ft
Surge tank level = 441.9' ft
North Fork flow = 105 cfs
Natoma flow = 31 cFs cfs

Other pumps on x, y, z
#8, 4 and 2
8 VFD Speed = 64%
Pump Disharge Pressure = 50

START 0944
Test on Pump 3
Time Start = 1347
Time End = 1357
S/D 1358
Reservoir Level = 397.74 ft
Surge tank level = 441.8 ft
North Fork flow = 116 cfs
Natoma flow = 31 cfs

Other pumps on x, y, z
#8, 6 and 3
8 VFD Speed = 46%
Pump Disharge Pressure = 55 PSi

START 1500
Test on Pump 4
Time Start 0902
Time End 0940
S/D 0942
Reservoir Level = 397.84 ft
Surge tank level = 442.1 ft
North Fork flow = 103 cfs
Natoma flow = 33 cfs

Other pumps on x, y, z
#8, 4 and 2
8 VFD Speed = 64%
Pump Disharge Pressure = 50

START 0941
Test on Pump 5
Time Start = 1322
Time End = 1333
S/D 1335
Reservoir Level = 397.75 ft
Surge tank level = 441.8 ft
North Fork flow = 110 cfs
Natoma flow = 30 cfs

Other pumps on x, y, z
#8, 5 and 3
8 VFD Speed = 54%
Pump Disharge Pressure = 42

START 1334
Test on Pump 6
Time Start = 1448
Time End
S/D 1459
Reservoir Level = ~~441.8~~ 397.68 ft
Surge tank level = 441.8 ft
North Fork flow = 111 cfs
Natoma flow = ~~37~~ 42 (37) cFs cfs

Other pumps on x, y, z
#8, 6 ~~and 3~~
#8 and 6
8 VFD Speed = 60%
Pump Disharge Pressure = 50 psi

Test on Pump 7
Time Start
Time End

Other pumps on x, y, z

other pmp 3

Reservoir Level 397.63 ft
Surge tank level 442.7 ft
North Fork flow 90 cfs
Natoma flow 37 cfs

VFD Speed =
Pump Disharge Pressure = 55

Test on Pump 8
Other pumps on x, y, z 0
Time Start
Time End
Reservoir Level 397.63 ft
Surge tank level 441.8 ft
North Fork flow 92 cfs
Natoma flow 34 cfs

VFD Speed =
Pump Disharge Pressure = 54

PUMP # 8

Time		KW	I	%N
1505	1	232	127	60
	2	233	126	60
	3	219	122	59
	4	241	127	61
	5	235	127	60
	6	220	122	59
	7	215	123	59
	8	214	122	59
	9	237	127	60
	10	234	126	60
1515	11	230	125	60

pump # 7

Time		KW	I	%N
1558	1	253	128	62
	2	253	129	62
	3	246	129	62
	4	271	134	63
	5	260	130	63
	6	244	128	62
	7	258	129	62
	8	250	129	62
	9	258	127	62
	10	268	131	63
1608	11	269	133	63

APPENDIX C

Hydraulic Model

Hydraulic Model Development

Pertinent reports, drawings, and operational data provided by Reclamation were compiled and reviewed. A listing of the documentation collected and analyzed is included in Appendix A. Information from these sources was used to establish the configuration of the physical system to be modeled.

A number of individuals provided or confirmed data related to the physical configuration and operation of the raw water delivery system at their respective ends. Information was provided by:

- Reclamation engineers Brian Zewe, Jesse Castro, and John Robinson.
- Reclamation Folsom Dam Operators Robert Skordas, Butch Branec, Art Pakao and Kenneth Zellner.
- Shawn Barnes, City of Roseville Water Treatment Plant
- Bill Sadler and Greg Turner, SJWD Water Treatment Plant
- Jim Bridges and Phil Carter, City of Folsom
- Mike Sundby and Pedro Reyes, Folsom Prison Water Treatment Plant

The raw water delivery system was modeled using InfoWater, a geospatial water distribution system modeling tool. The attributes of system components (elevations of junctions and valves, lengths and diameters of pipes, pump characteristics, reservoir water levels) were determined and coded into the InfoWater model.

Model Calibration

The computer model was initially calibrated using hand calculations and later re-calibrated using data from field test conducted in April and September, 2009. In its calibrated version, the model closely reproduces head losses measured during field tests, as illustrated in Figures C-1 to C-3.

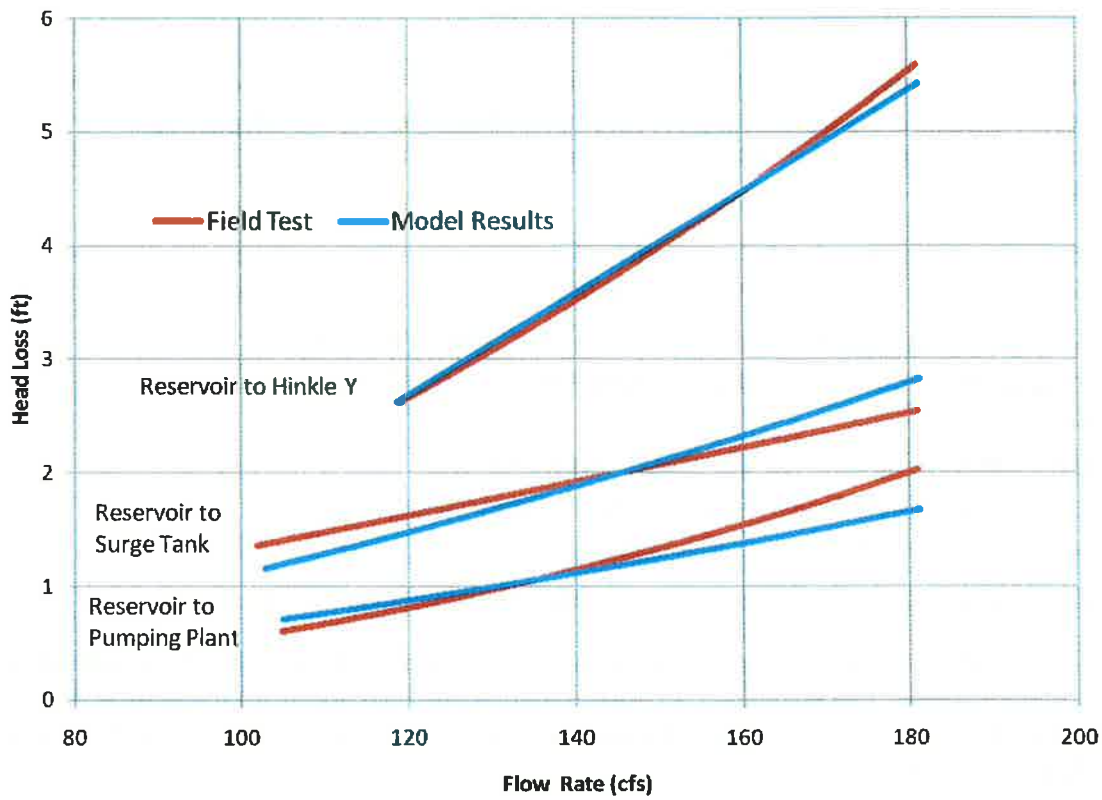


Figure C-1. Comparison of Head Losses – North Fork Pipeline

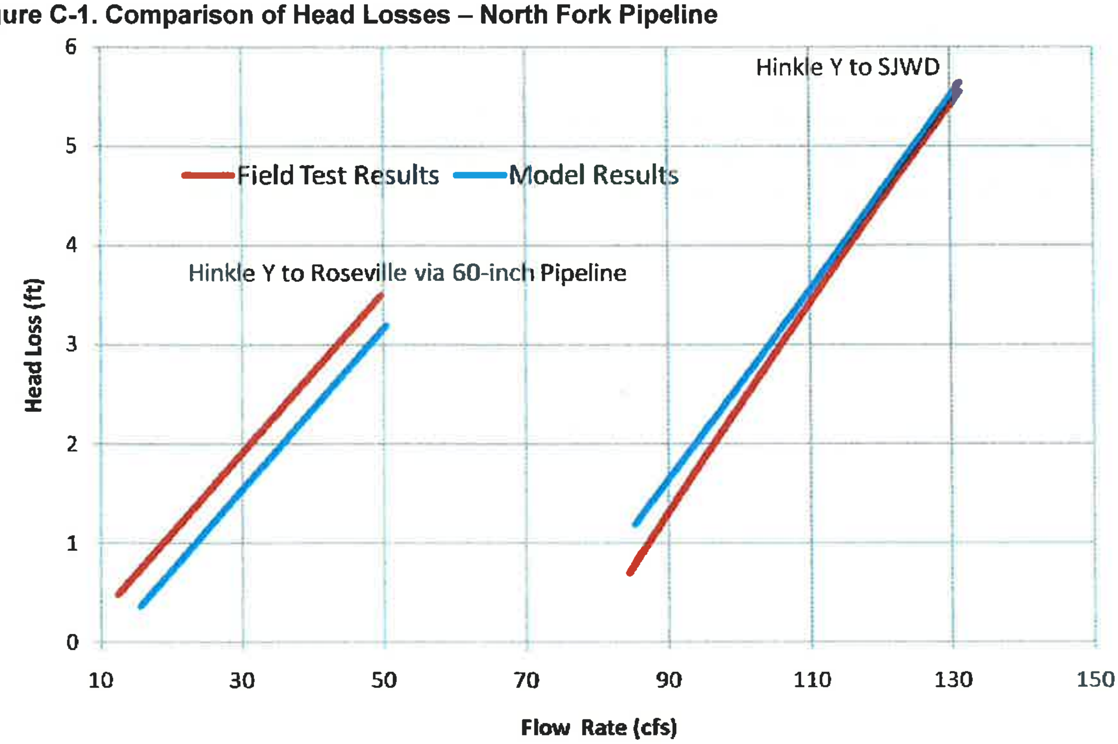


Figure C-2. Comparison of Head Losses – City of Roseville and SJWD Pipes

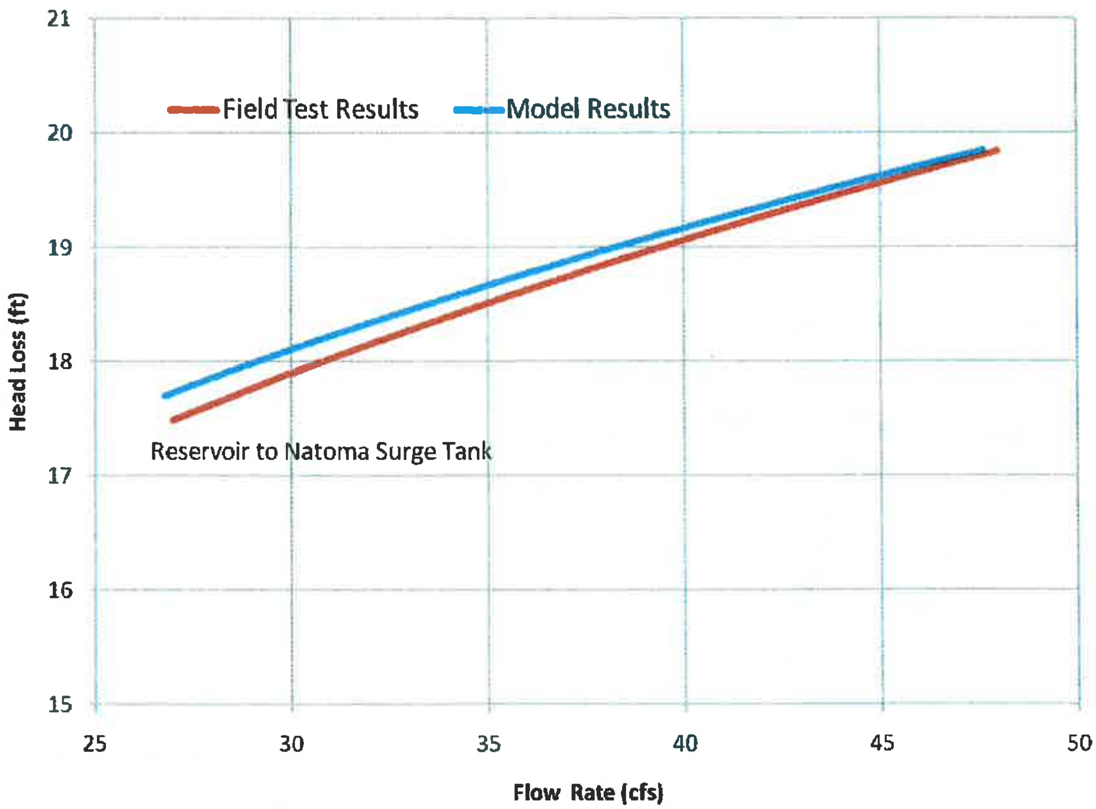


Figure C-3. Comparison of Head Losses – Natoma Pipeline

Attributes of Hydraulic Model Components

Figure C-4 illustrates the extent of the hydraulic model. Relevant elevations, diameters, lengths, and pump characteristics are listed below, Tables C-1 to C-12.

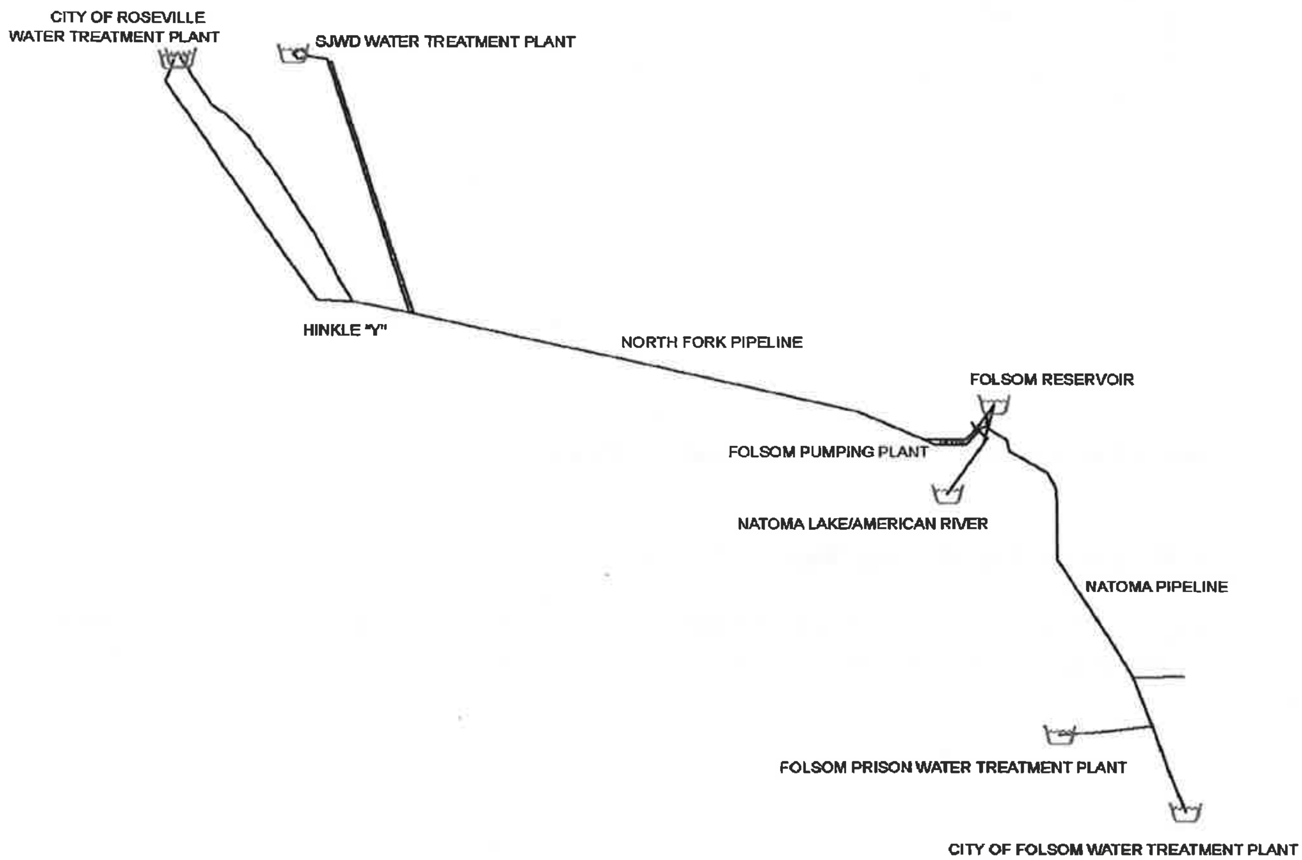


Figure C-4. Extent of Hydraulic Model - Folsom Dam Raw Water Delivery System

Table C-1. Junctions

ID	ELEVATION	ID	ELEVATION	ID	ELEVATION
J10	314.75	J72	314.75	J148	381.20
J12	314.75	J74	388.50	J150	396.34
J14	314.75	J76	367.70	J152	388.33
J16	314.75	J78	394.31	J154	403.37
J18	314.75	J80	386.50	J156	350.33
J20	314.75	J82	383.80	J158	407.84
J22	314.75	J90	388.50	J160	398.37
J24	314.75	J92	389.50	J162	344.95
J26	314.75	J94	389.50	J164	381.81
J28	314.75	J96	394.31	J166	372.74
J30	314.75	J98	394.31	J168	390.88
J32	314.75	J100	313.00	J170	388.62
J34	314.75	J102	328.30	J182	382.69
J36	314.75	J106	282.00	J184	372.00
J40	314.75	J108	447.00	J186	390.49
J44	314.75	J110	394.31	J188	373.71
J46	316.45	J112	368.50	J190	400.32
J48	317.00	J122	365.00	J192	357.12
J50	317.00	J124	396.00	J194	359.33
J52	319.00	J128	368.50	J196	401.23
J54	342.30	J130	368.50	J198	344.65
J56	318.65	J132	404.00	J200	317.00
J58	342.00	J134	387.75	J202	376.88
J60	258.00	J136	394.00	J204	373.55
J62	316.17	J138	392.51	J206	391.11
J64	316.17	J140	367.69	J208	369.95
J66	325.50	J142	383.10	J210	384.00
J68	325.50	J144	390.90		
J70	315.75	J146	378.73		

Table C-2. Reservoirs

ID	Water Level (ft)
FOLSOM	307 - 466
SAN_JUAN	423.4
AMRIVER	150.0
ROSEVILLE (60-inch)	400.0
ROSEVILLE (48-inch)	403.26
CITY_OF_FOLSOM	407.5
PRISON	408.8

Table C-3. Valves (K=0.2)

ID	Elevation (ft)	Diameter (In)
V2	314.75	60
V3	314.75	60
V4	314.75	24
V5	314.75	42
V6	314.75	18
V7	314.75	24
V8	314.75	24
V9	316.08	60
V10	316.20	42
V14	314.75	30
V15	314.75	20
V16	314.75	30
V17	314.75	30
V18	314.75	30
V24	360.00	42
V25	314.75	30
V26	314.75	30
V27	314.75	36
V28	314.75	36
V29	323.00	36
V30	256.00	36
V32	256.00	36
V33	319.00	36
V34	314.75	36
V101	368.50	48
V102	368.50	12
V8030	394.31	42
V8046	388.60	42

Table C-3. Valves (K=0.2)

ID	Elevation (ft)	Diameter (in)
V8048	394.00	42
V8064	388.60	72
V8068	386.50	60
V8070	394.31	42
V8072	389.50	48
V8074	389.50	48
V8076	383.80	36
V9000	394.00	48
V9002	385.00	24
VENTURI_METER	340.00	60

Table C-4. Pipes

ID	LENGTH	MATERIAL	DIAMETER	FROM NODE	TO NODE
P10	7	Steel	30	J10	V34
P12	10	Steel	84	J10	J14
P14	10	Steel	84	J14	J16
P16	20	Steel	84	J16	J18
P18	10	Steel	84	J18	J20
P20	20	Steel	84	J20	J22
P22	10	Steel	84	J22	J24
P24	10	Steel	60	J12	J26
P26	7	Steel	30	J14	V28
P28	7	Steel	30	J16	V27
P30	10	Steel	60	J26	J28
P32	20	Steel	60	J28	J30
P34	7	Steel	30	J18	V18
P36	10	Steel	60	J30	J32
P38	7	Steel	30	J20	V17
P40	20	Steel	42	J32	J34
P42	7	Steel	30	J22	V16
P44	10	Steel	42	J34	J36
P46	7	Steel	20	J24	V15
P48	7	Steel	30	V34	PUMP_8_VFD
P50	8	Steel	24	PUMP_8_VFD	V14

Table C-4. Pipes

ID	LENGTH	MATERIAL	DIAMETER	FROM NODE	TO NODE
P52	8	Steel	24	V14	J12
P54	7	Steel	30	V28	PUMP_7_VFD
P56	8	Steel	24	PUMP_7_VFD	V26
P58	8	Steel	24	V26	J26
P60	7	Steel	30	V27	PUMP_6
P62	8	Steel	24	PUMP_6	V25
P64	8	Steel	24	V25	J28
P66	7	Steel	30	V18	PUMP_5
P68	8	Steel	24	PUMP_5	V8
P70	8	Steel	24	V8	J30
P72	7	Steel	30	V17	PUMP_4
P74	8	Steel	24	PUMP_4	V4
P76	8	Steel	24	V4	J32
P78	7	Steel	30	V16	PUMP_3
P80	8	Steel	24	PUMP_3	V7
P82	8	Steel	24	V7	J34
P84	7	Steel	20	V15	PUMP_2
P86	8	Steel	18	PUMP_2	V6
P88	8	Steel	18	V6	J36
P90	10	Steel	42	J36	V5
P96	10	Steel	42	V5	J40
P98	20	Steel	84	J24	J44
P100	50	Steel	84	J44	J46
P102	50	Steel	84	J48	J46
P104	50	Steel	84	V9	J48
P106	5	Steel	36	J46	V29
P108	5	Steel	36	J52	V33
P110	10	Steel	36	J54	J52
P112	207	Steel	42	J40	J56
P114	50	Steel	36	J54	J58
P116	10	Steel	36	V32	J58
P118	50	Steel	84	J50	V9
P120	5	Steel	36	V29	J52
P122	5	Steel	36	V33	J56
P124	72	Steel	42	J56	J62
P126	22	Steel	42	J62	J64
P128	24	Steel	42	J64	J66

Table C-4. Pipes

ID	LENGTH	MATERIAL	DIAMETER	FROM NODE	TO NODE
P130	610	Steel	42	J66	J68
P132	50	Steel	42	J50	V10
P134	50	Steel	42	V10	J66
P136	30	Steel	60	J12	V2
P138	59	Steel	60	J72	J70
P140	29	Steel	84	J10	V3
P142	75	Steel	84	J70	VENTURI_METER
P144	1,279	Steel	84	J76	J124
P146	10	Steel	42	J74	V8046
P148	5	Steel	60	V2	J72
P150	30	Steel	84	V3	J70
P152	400	Steel	60	J74	J80
P154	25	Steel	60	J80	V8068
P156	100	Steel	60	J82	V8076
P158	20	Steel	60	V8076	ROSEVILLE
P162	26	Steel	84	J90	J74
P164	855	Steel	54	J78	J110
P166	100	Steel	60	J92	J94
P168	10	Steel	72	J90	V8064
P170	855	Steel	66	J98	J96
P172	78	Steel	42	J68	J100
P174	415	Steel	42	J100	J106
P176	1,087	Steel	42	J102	V24
P178	53	Steel	84	J200	J50
P180	81	Steel	42	J106	J102
P182	5	Steel	36	EMERGENCY_PUMP	V32
P184	5	Steel	36	V30	EMERGENCY_PUMP
P186	200	Steel	48	J54	J108
P188	50	Steel	54	J110	J92
P190	5	Steel	42	J96	V8070
P192	750	Steel	42	V8046	V8048
P194	15	Steel	42	J98	V8030
P196	10	Steel	42	V8048	J78
P198	5	Steel	42	V8030	J78
P200	38	Steel	48	J112	J128
P202	3,000	Steel	48	J134	J136
P204	5	Steel	48	J136	V9000

Table C-4. Pipes

ID	LENGTH	MATERIAL	DIAMETER	FROM NODE	TO NODE
P206	103	Steel	48	J128	J130
P208	1	Steel	48	V9000	CITY_OF_FOLSOM
P210	378	Steel	60	J138	J82
P212	500	Steel	60	J140	J138
P214	307	Steel	60	J142	J144
P216	251	Steel	60	J144	J146
P218	768	Steel	60	J146	J148
P220	1,227	Steel	60	J148	J150
P222	951	Steel	60	J150	J152
P224	103	Steel	60	J152	J154
P226	707	Steel	84	J122	J76
P228	1,354	Steel	84	J124	J90
P230	503	Steel	60	J154	J156
P232	0.1	Steel	84	FOLSOM	J200
P234	761	Steel	72	V8064	J98
P236	100	Steel	48	J94	V8072
P238	1,057	Steel	60	J156	J158
P240	5	Steel	36	J60	V30
P242	9,400	Steel	60	V8068	J82
P244	20	Steel	42	V8070	J110
P246	200	Steel	186	FOLSOM	J60
P248	400	Steel	186	AMRIVER	J60
P250	82	Steel	42	V24	J112
P252	4	Steel	48	J130	V101
P254	4,480	Steel	60	V101	J134
P256	4	Steel	18	J130	V102
P258	1,000	Steel	18	V102	PRISON
P260	100	Steel	48	J94	V8074
P262	10	Steel	48	V8072	SAN_JUAN
P264	10	Steel	48	V8074	SAN_JUAN
P266	200	Steel	30	J128	J132
P268	67	Steel	84	VENTURI_METER	J122
P270	96	Steel	60	J158	J160
P272	551	Steel	60	J160	J162
P274	1,077	Steel	60	J162	J164
P276	437	Steel	60	J164	J166

Table C-4. Pipes

ID	LENGTH	MATERIAL	DIAMETER	FROM NODE	TO NODE
P280	565	Steel	60	J168	J140
P282	456	Steel	60	J170	J80
P292	40	Steel	60	J80	J182
P294	1,181	Steel	48	J182	J184
P296	970	Steel	48	J184	J186
P298	460	Steel	48	J186	J188
P300	750	Steel	48	J188	J190
P302	300	Steel	48	J190	J192
P304	270	Steel	48	J192	J194
P306	665	Steel	48	J194	J196
P308	785	Steel	48	J196	J198
P310	1,480	Steel	48	J198	J202
P312	200	Steel	48	J202	J204
P314	380	Steel	48	J204	J206
P316	700	Steel	48	J206	J208
P318	569	Steel	48	J208	J210
P324	50	Steel	48	J210	V9002
P330	50	Steel	48	V9002	RES9010

Table C-5. Pump IDs

ID	ELEVATION
Pump_8_VFD	314.75
Pump_7_VFD	314.75
Pump_6	314.75
Pump_5	314.75
Pump_4	314.75
Pump_3	314.75
Pump_2	314.75
Emergency_Pump	252.56

Pump Characteristics

Table C-6. Pump 2

Q (cfs)	Head (ft)	Efficiency (%)	BHP
0	137	0	120
10	123	67	220
15	118	83	240
20	100	88	260
25	78	84	260
28	60	72	250

Table C-7. Pump 3

Q (cfs)	Head (ft)	Efficiency (%)	BHP
0	122	0	300
10	118	43	360
20	117	65	440
30	110	80	480
40	106	88	540
50	98	90	610
60	93	88	620
70	63	78	620

Table C-8. Pumps 4 and 5

Q (cfs)	Head (ft)	Efficiency (%)	BHP
0	130	0	290
10	124	50	340
20	115	73	370
30	102	85	400
40	84	88	410
50	50	70	390
53	29	45	370

Table C-9. Pumps 6 and Emergency

Q (cfs)	Head (ft)	Efficiency (%)	BHP
0	120	0	450
20	114	43	475
40	106	67	500
50	104	76	530
60	98	82	550
70	94	85	555
80	86	87	560
90	80	86	550
100	65	83	530
108	54	80	500

Table C-10. Pumps 7 and 8 at 511 RPM (Full Speed)

Q (cfs)	Head (ft)	Efficiency (%)	BHP
0	162	0	885
23	155	37	1069
45	154	71	1112
68	140	86	1263
87	126	91	1371
91	122	90	1403
111	78	67	1468

Table C-11. Pumps 7 and 8 at 358 RPM

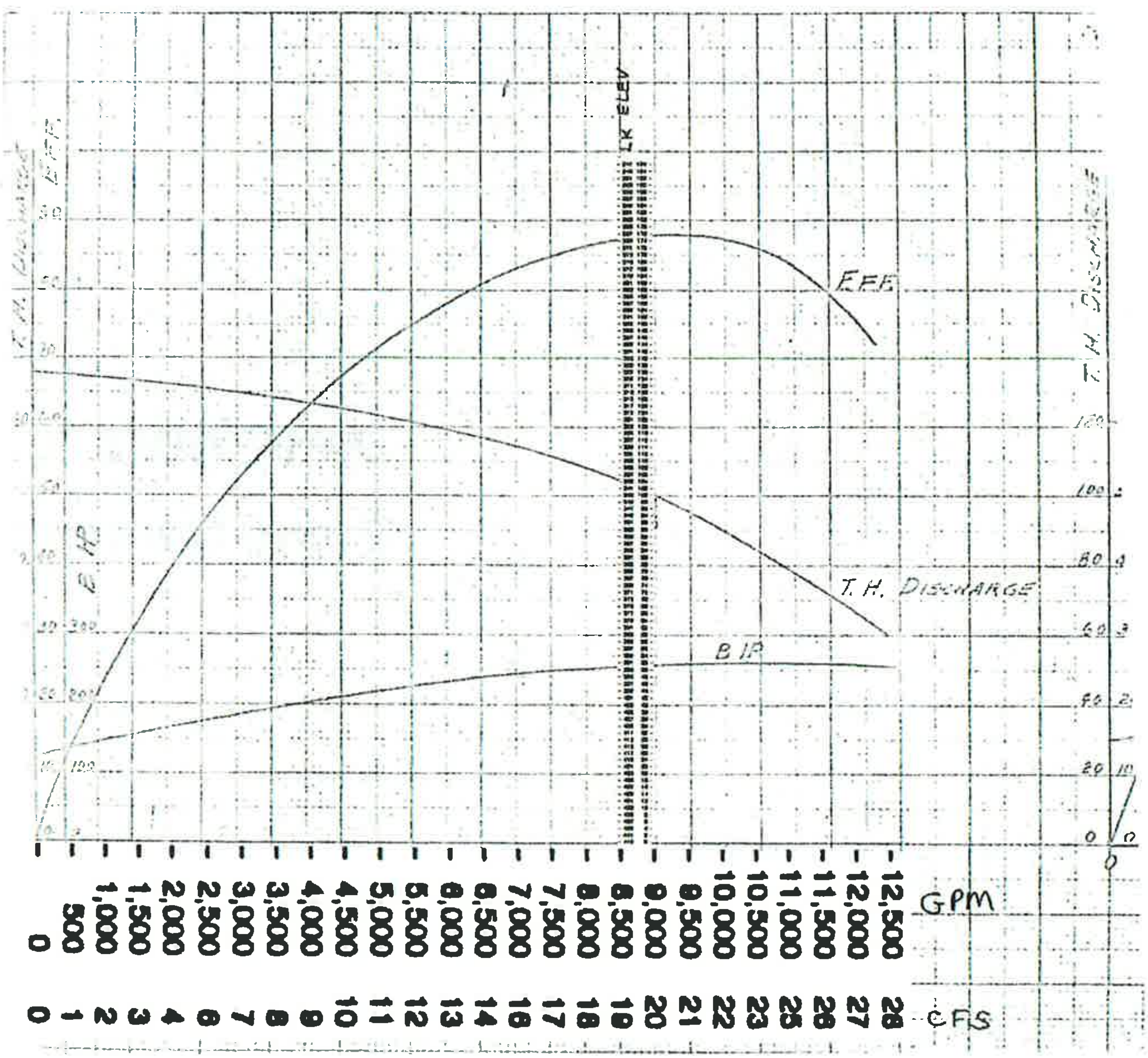
Q (cfs)	Head (ft)	Efficiency (%)	BHP
0	81	0	307
15	77	37	339
32	74	68	387
47	69	86	430
61	63	91	472
63	61	91	483
79	49	85	515
88	41	78	520

Table C-12. Pumps 7 and 8 at 255 RPM

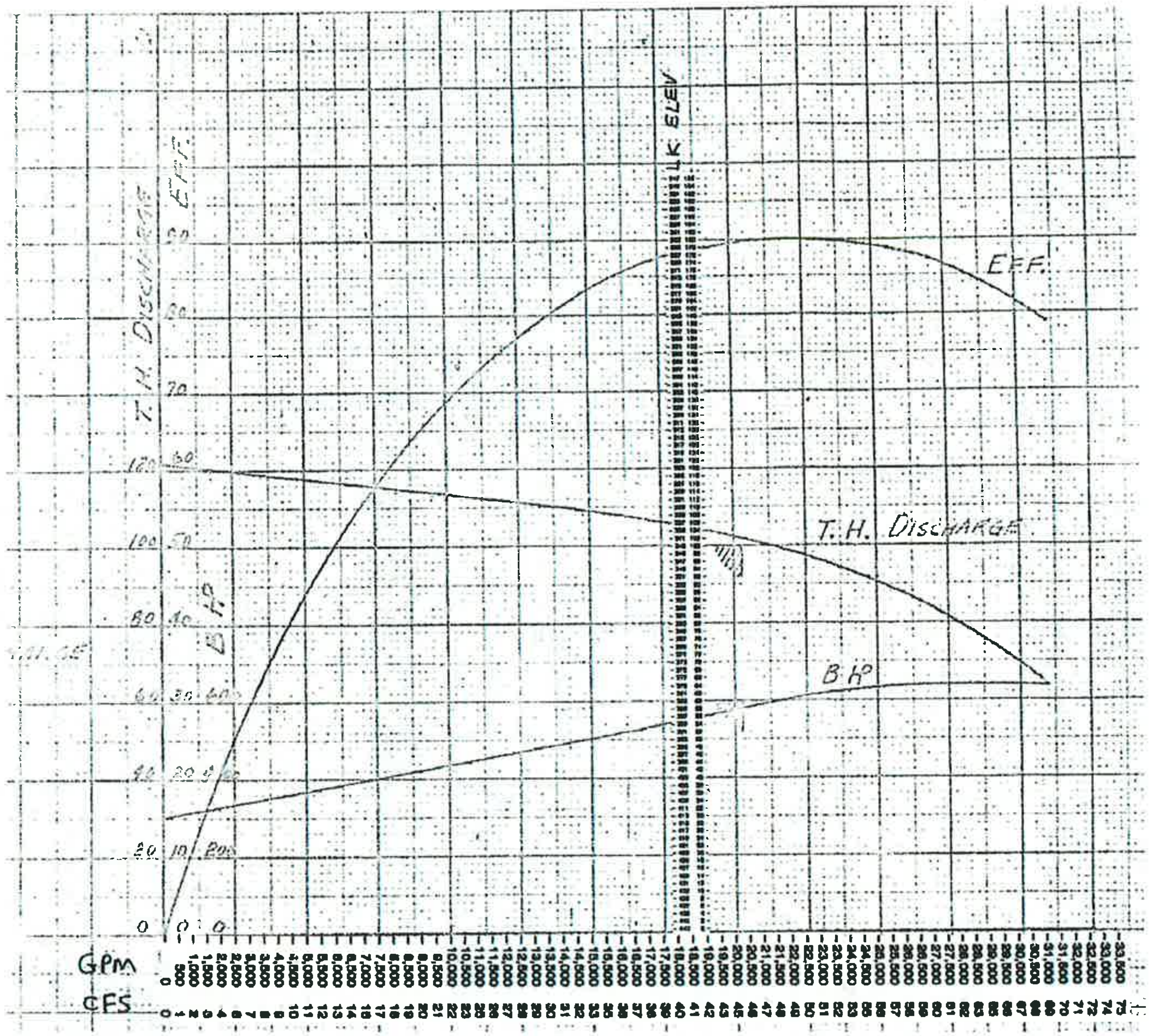
Q (cfs)	Head (ft)	Efficiency (%)	BHP
22	38	70	
28	36	86	
43	31	91	
50	28	86	

APPENDIX D

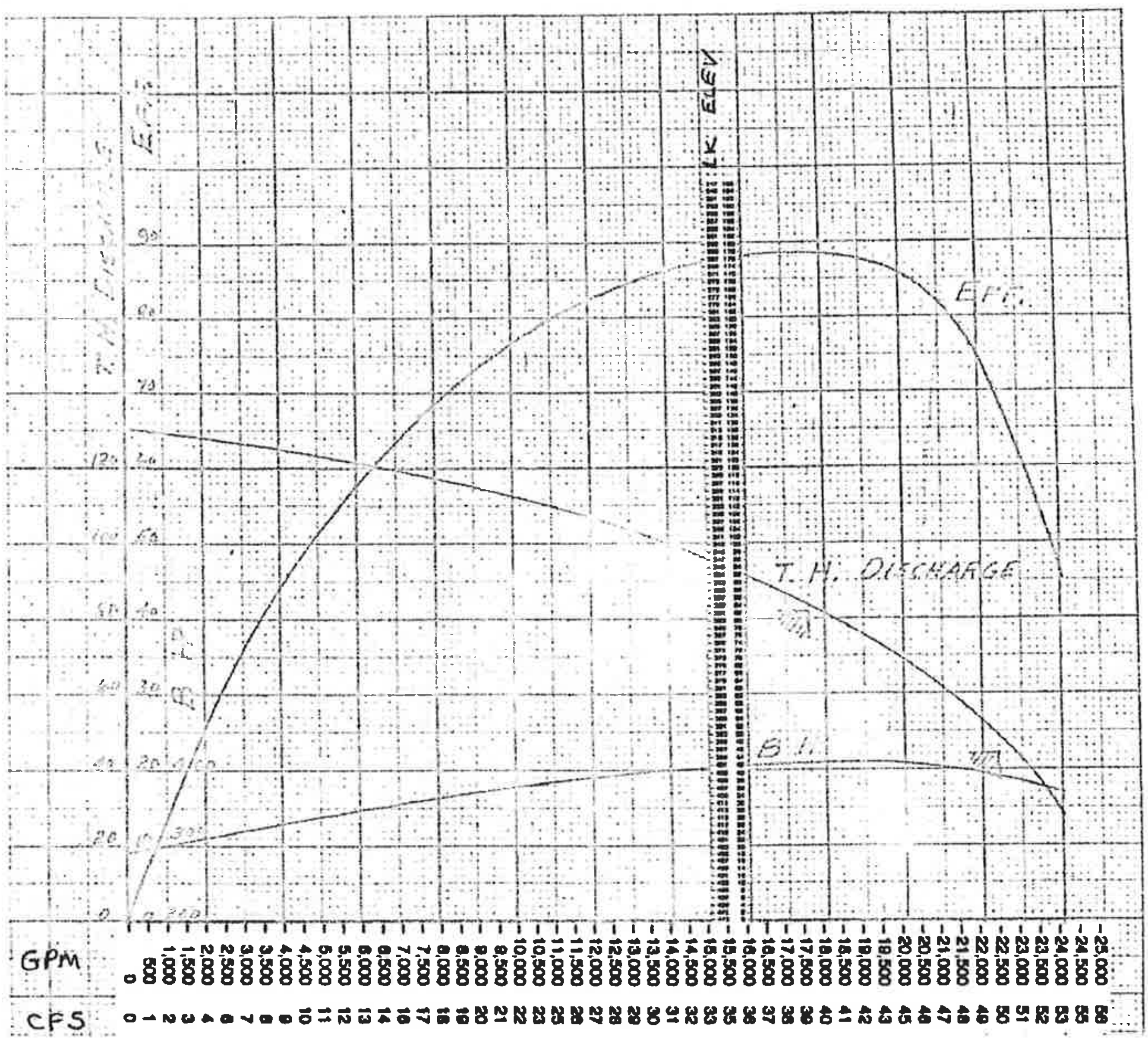
Pump Curves



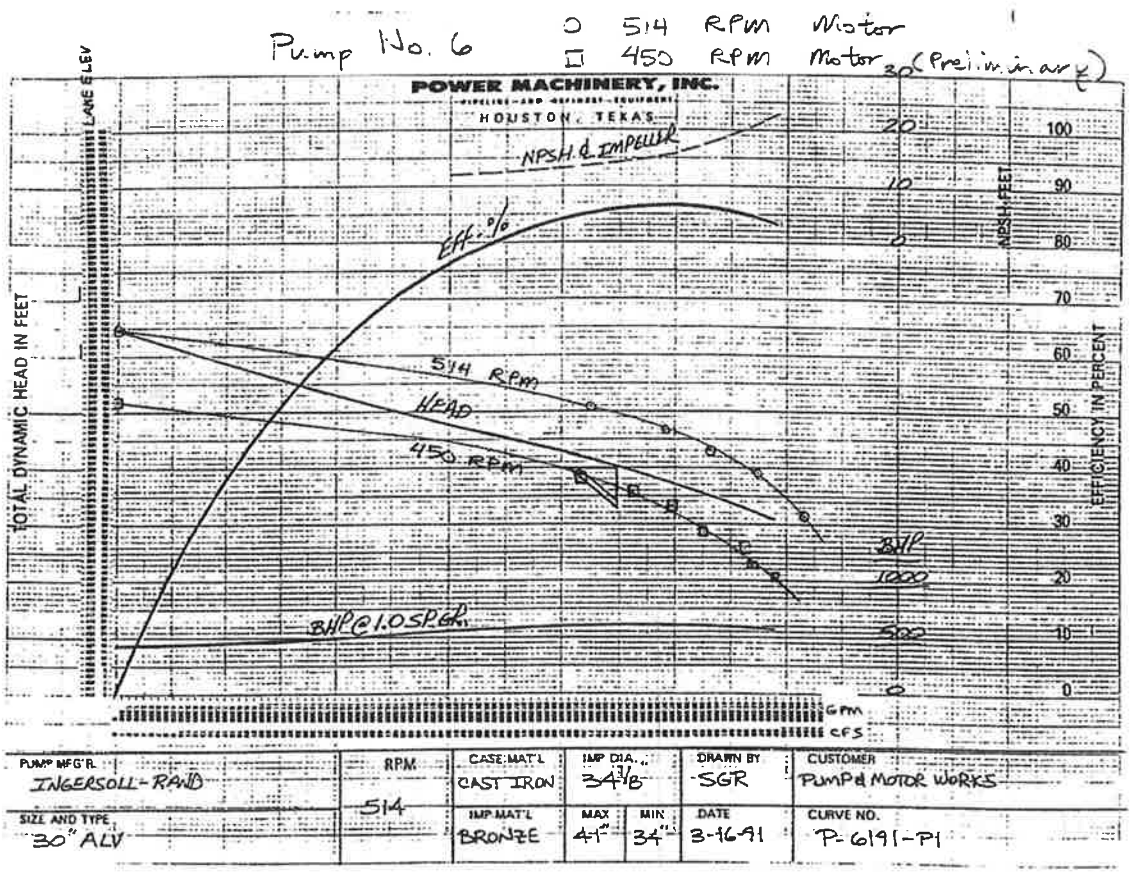
Operating Range for Pump #2



Operating Range for Pump #3



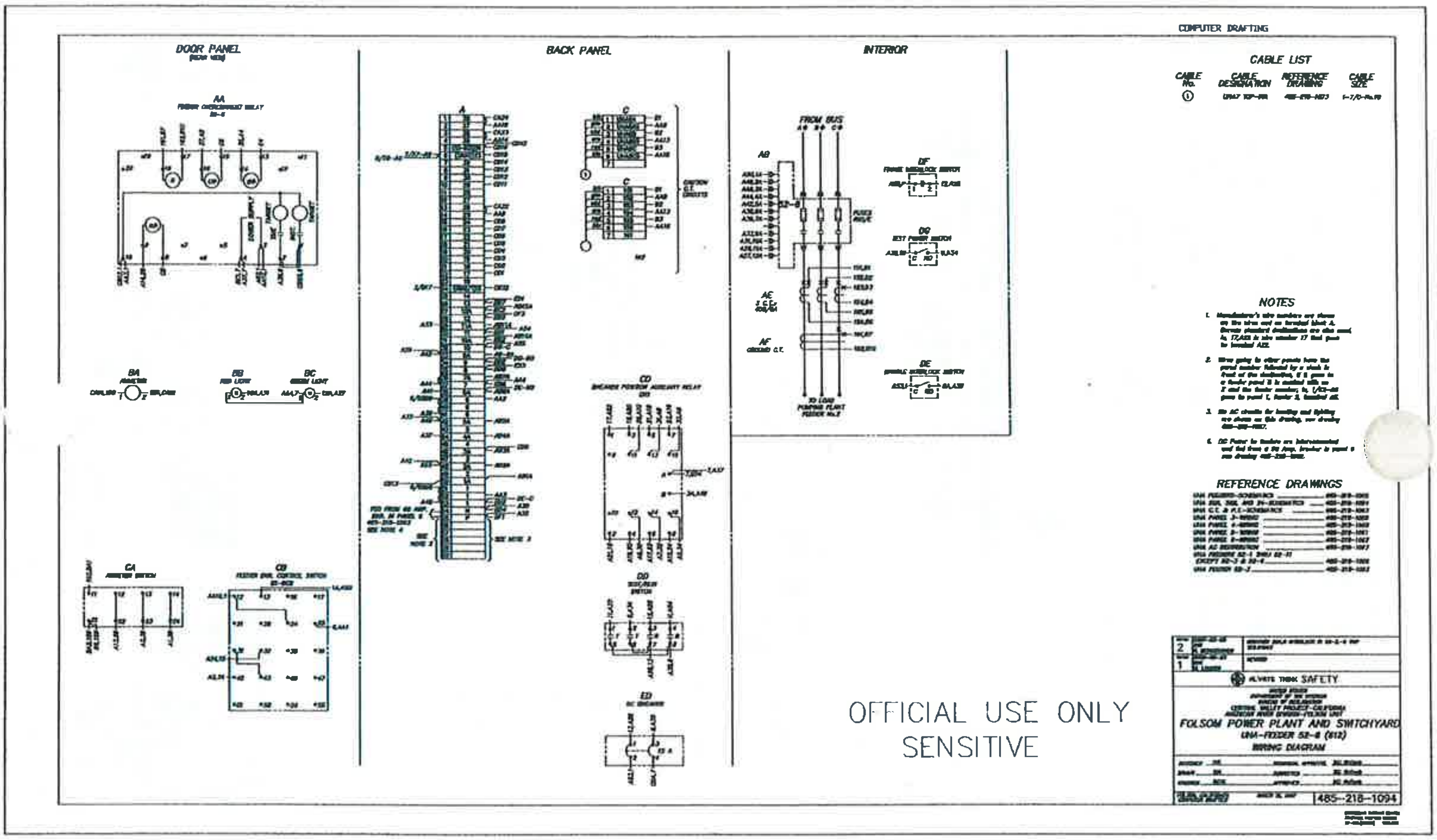
Operating Range for Pumps #4 and #5



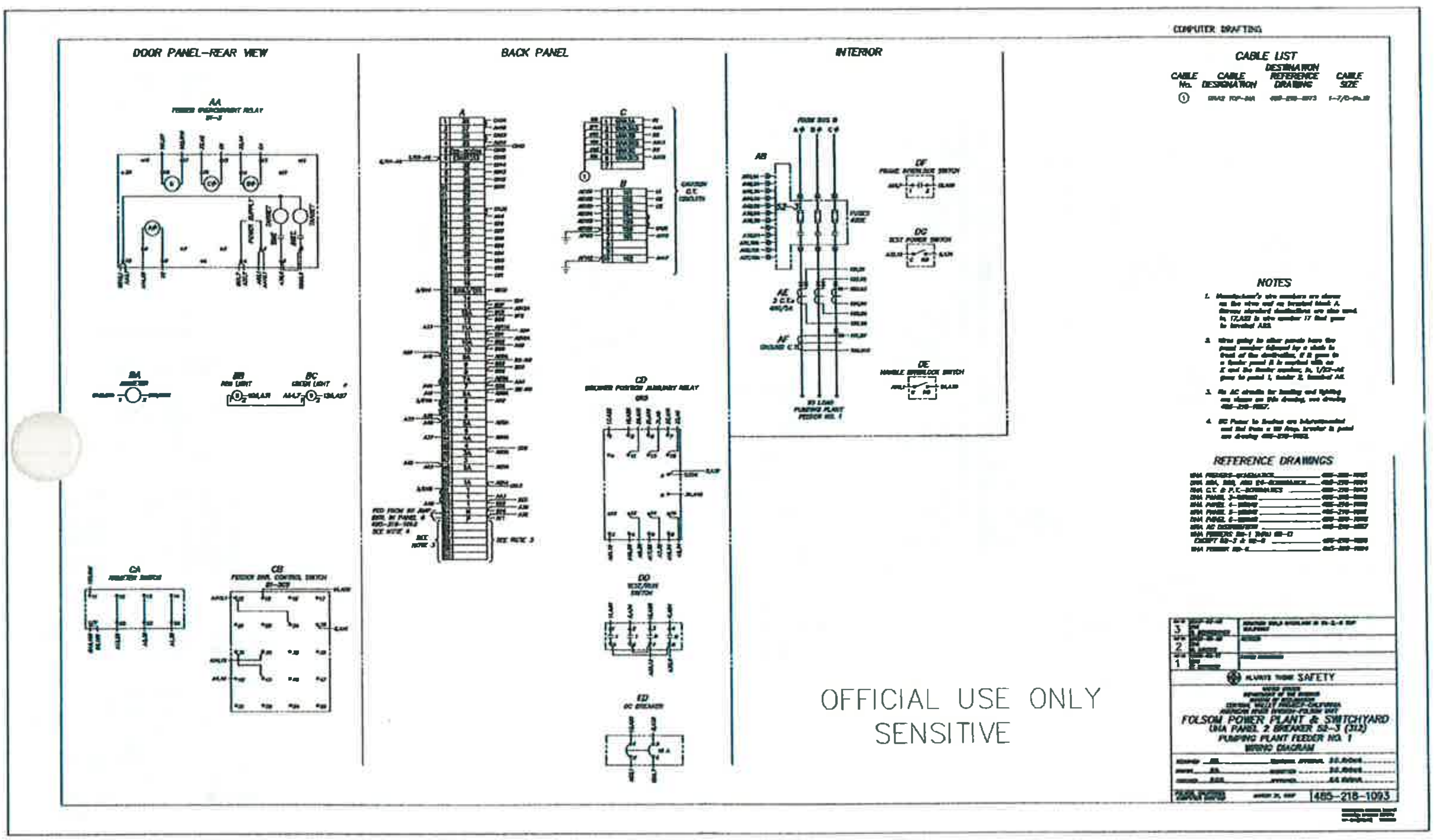
Operating Range for Pump #6 and Emergency Pump

APPENDIX E

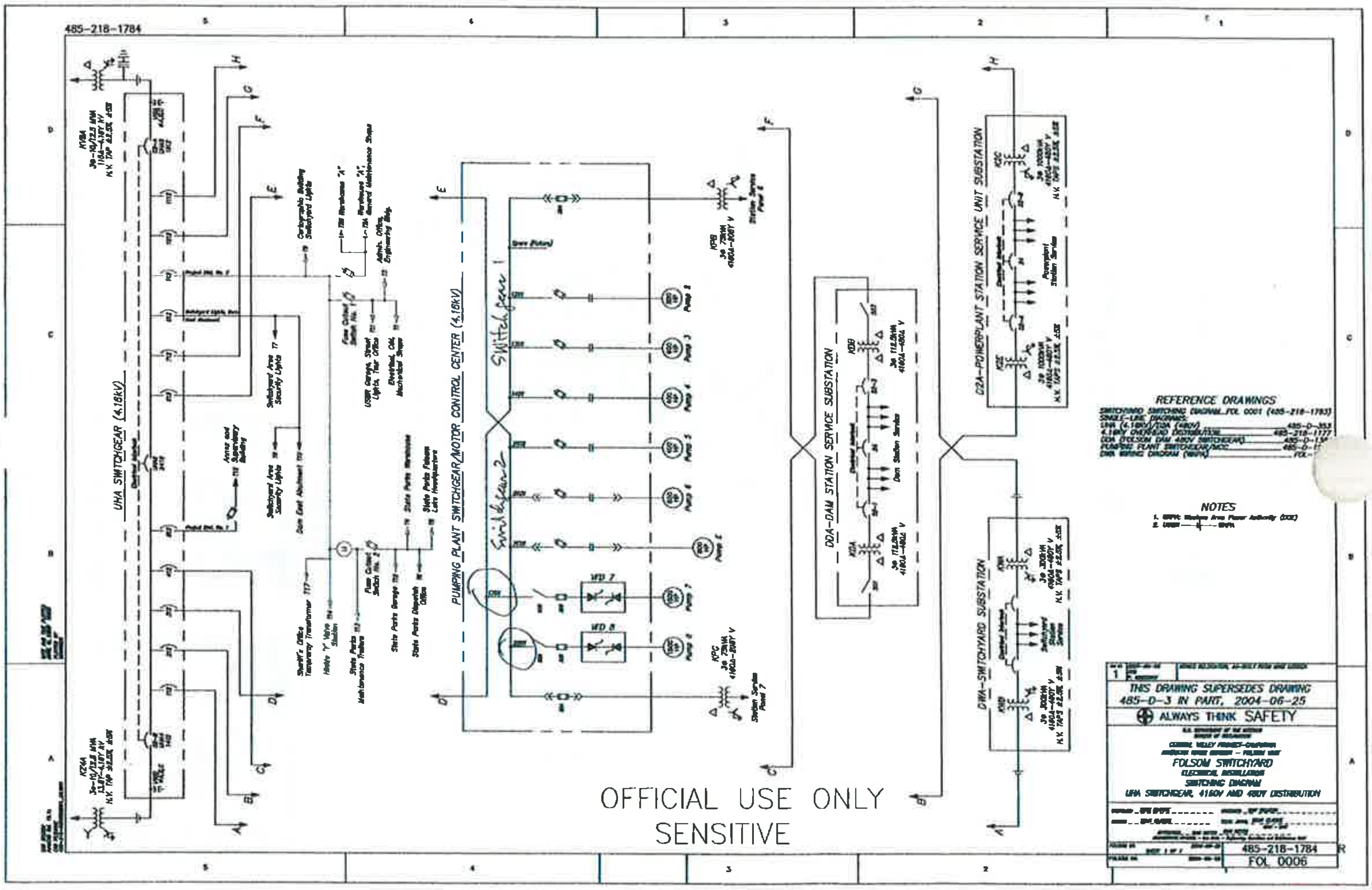
Referenced Electrical/Control Drawings



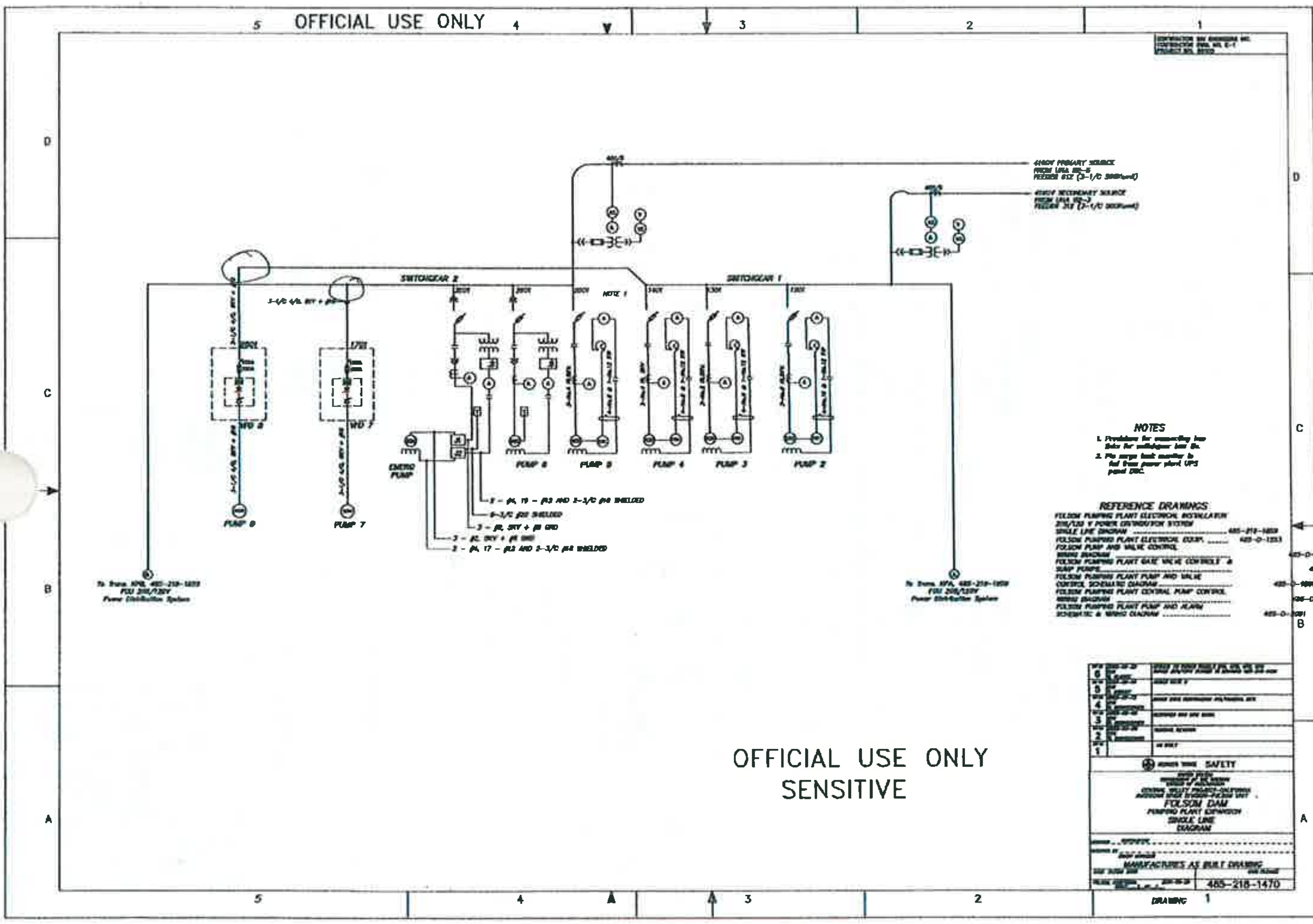
OFFICIAL USE ONLY
SENSITIVE



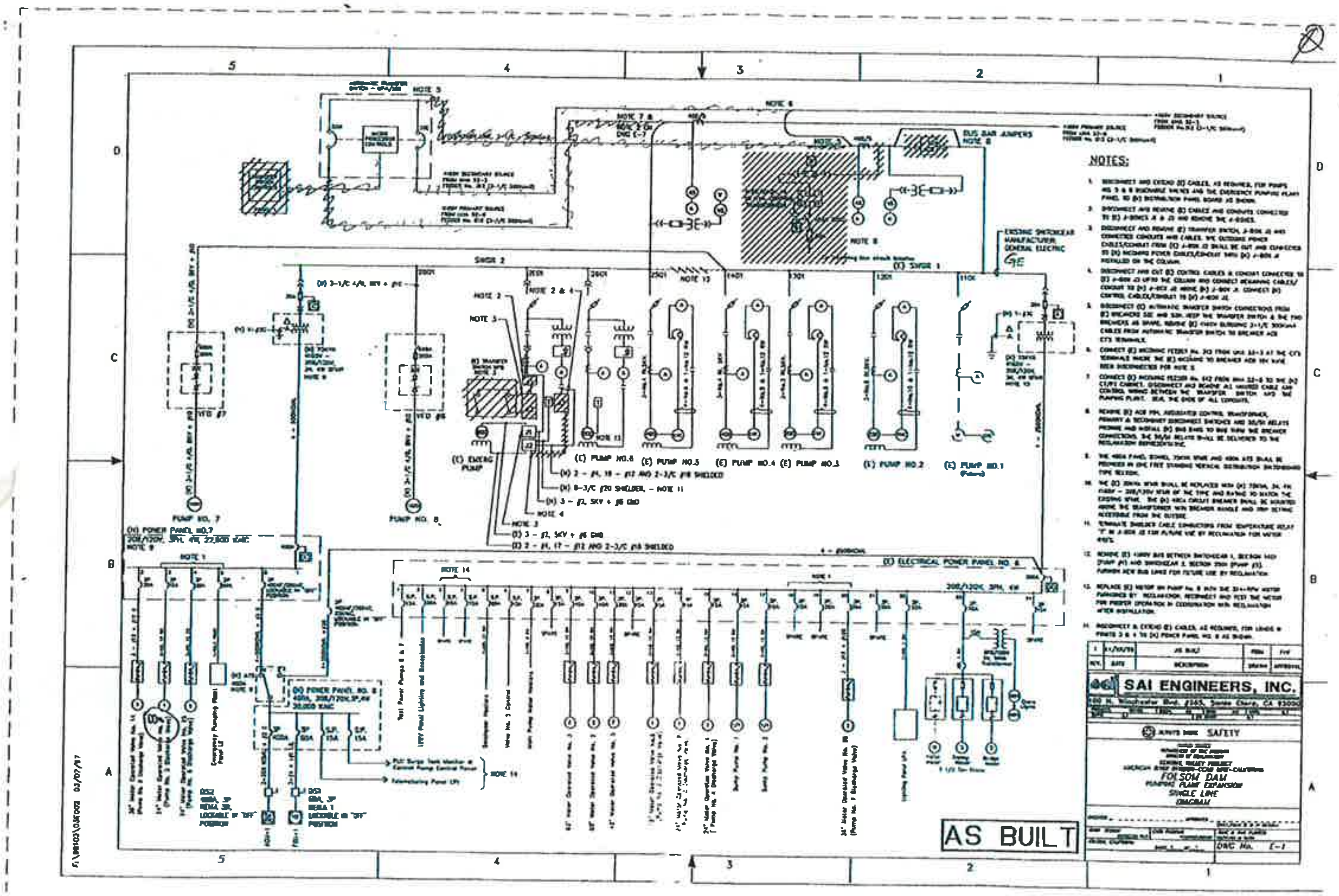
OFFICIAL USE ONLY
SENSITIVE



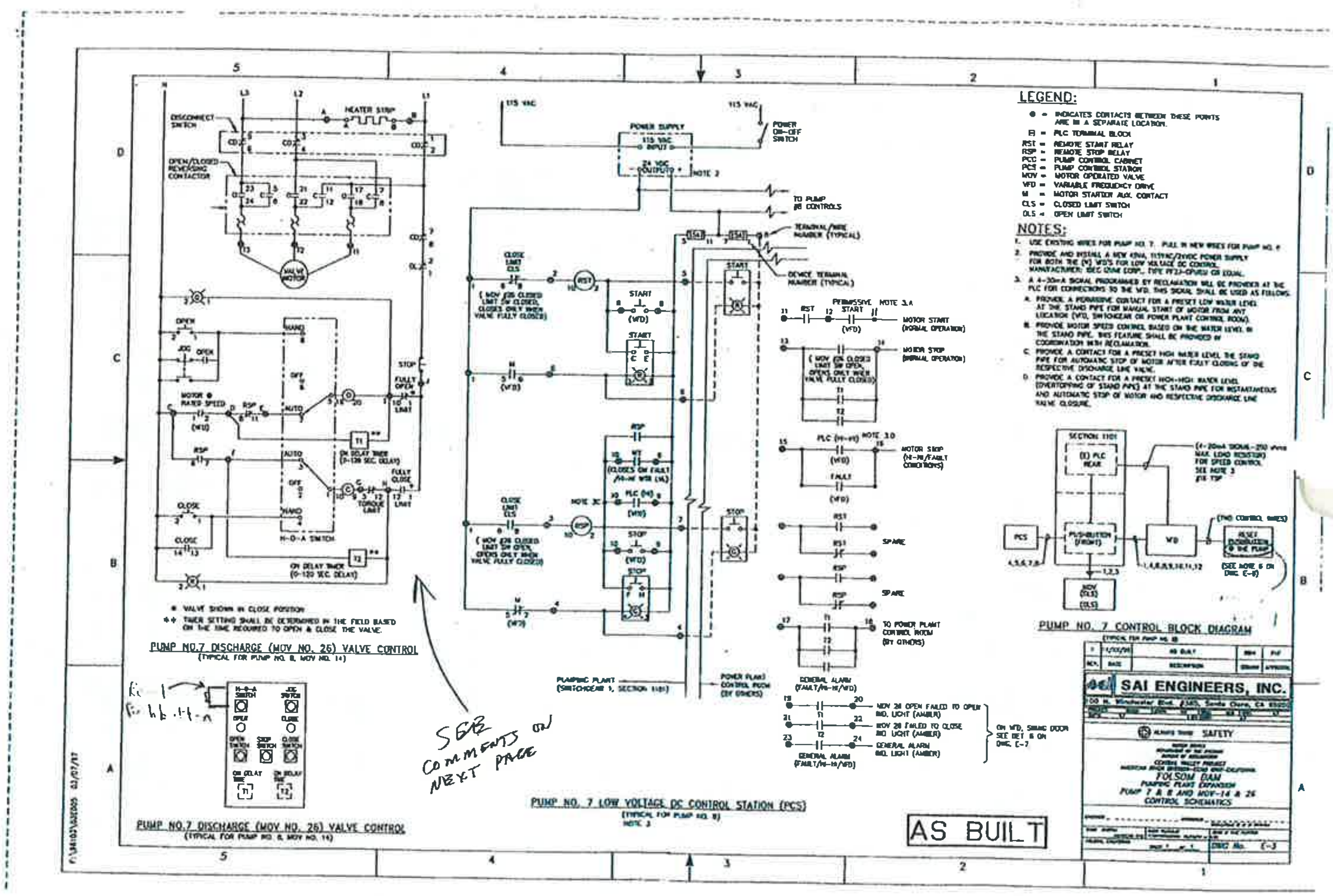
OFFICIAL USE ONLY
SENSITIVE



OFFICIAL USE ONLY
SWITCHGEAR 2
SWITCHGEAR 1
PUMP 8
PUMP 7
PUMP 6
PUMP 5
PUMP 4
PUMP 3
PUMP 2
EMERG PUMP
NOTES
REFERENCE DRAWINGS
SAFETY
FOLSOM DAM
PUMPING PLANT EXPANSION
SINGLE LINE
DIAGRAM
MANUFACTURES AS BUILT DRAWING
485-218-1470
DRAWING
OFFICIAL USE ONLY
SENSITIVE



NOTES:
SAI ENGINEERS, INC.
SAFETY
FOLSOM DAM
AS BUILT
DWG No. E-1
PUMP NO. 7
PUMP NO. 8
(E) PUMP NO.5
(E) PUMP NO.4
(E) PUMP NO.3
(E) PUMP NO.2
(E) PUMP NO.1
(E) ELECTRICAL POWER PANEL NO. 6
208/120V, 3PH, 4W
BUS BAR JUMPERS
NOTE 6
NOTE 14
GE


LEGEND:
RST = REMOTE START RELAY
RSP = REMOTE STOP RELAY
PCC = PUMP CONTROL CABINET
PCS = PUMP CONTROL STATION
MOV = MOTOR OPERATED VALVE
VFD = VARIABLE FREQUENCY DRIVE
CLS = CLOSED LIMIT SWITCH
OLS = OPEN LIMIT SWITCH
NOTES:
PUMP NO.7 DISCHARGE (MOV NO. 26) VALVE CONTROL
PUMP NO. 7 LOW VOLTAGE DC CONTROL STATION (PCS)
PUMP NO. 7 CONTROL BLOCK DIAGRAM
SAI ENGINEERS, INC.
FOLSOM DAM
AS BUILT
SEE COMMENTS ON NEXT PAGE

APPENDIX F

Proposed Pump Selection Schedule

Folsom Pumping Plant

Proposed Pump Operation Schedule for Pumped Raw Water Deliveries

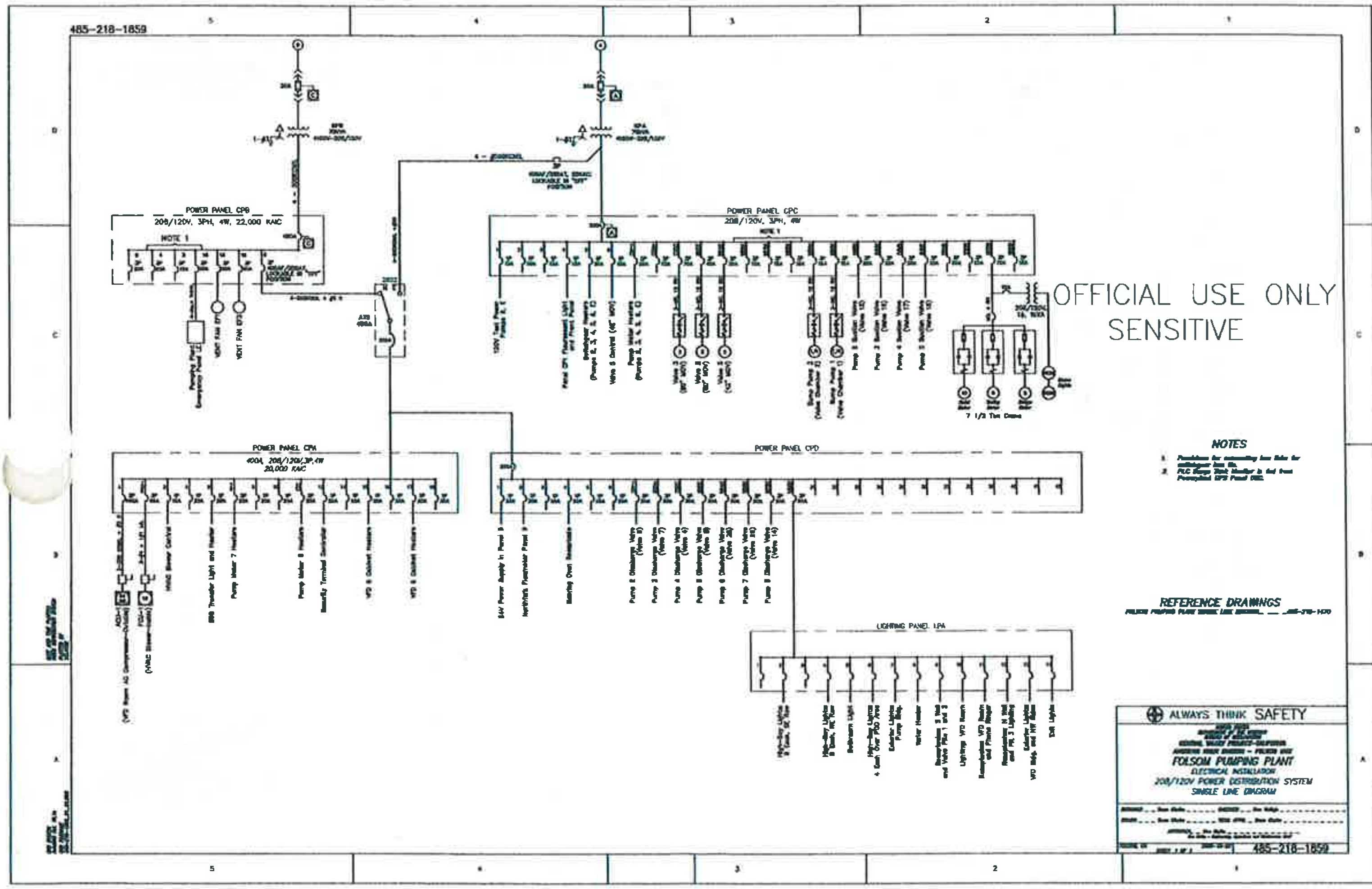
1. Determine Folsom Reservoir water level (ft elevation)

2. Determine total pumping demand (cfs)

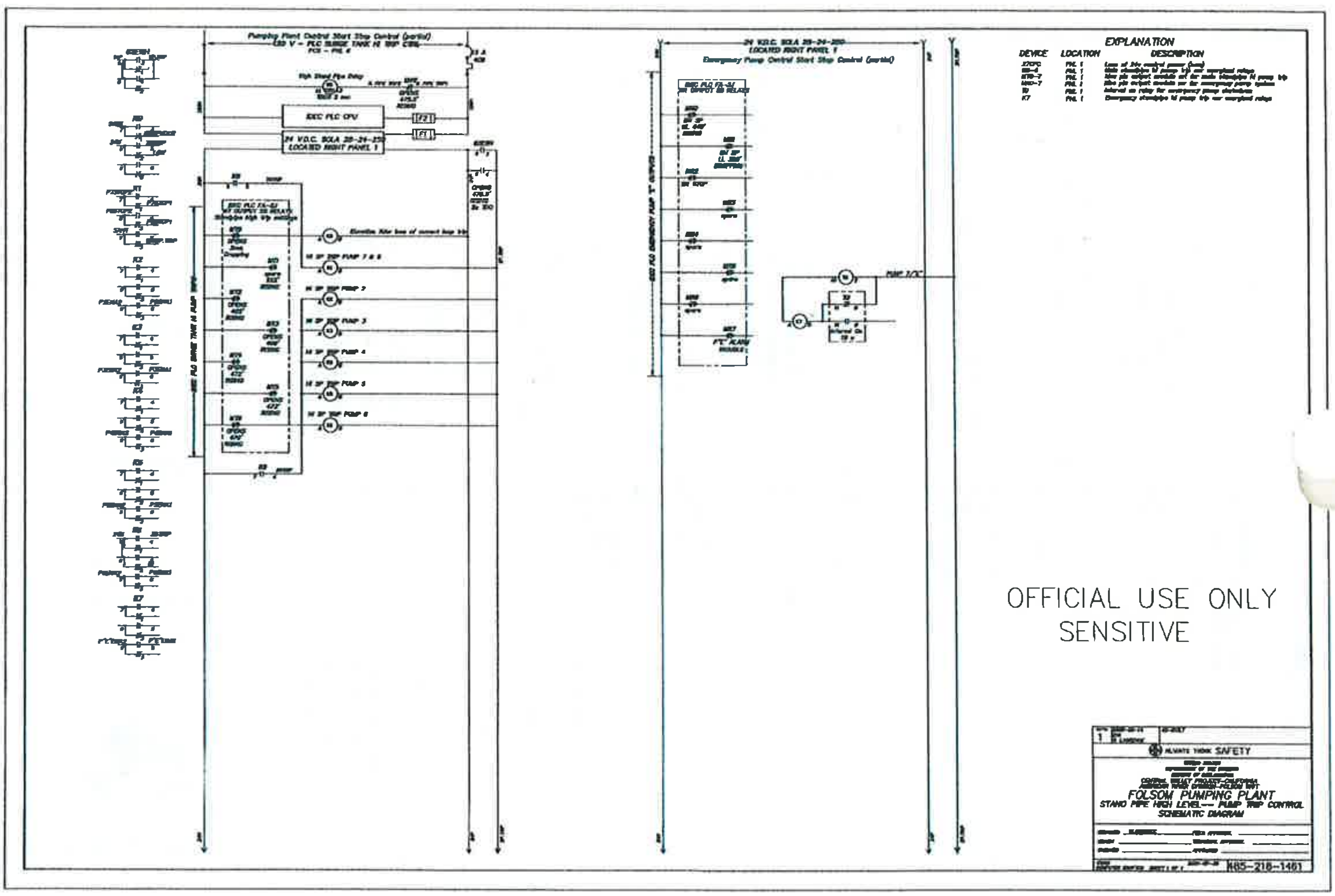
3. Select spreadsheet tab from graph below:

4. Select pumps and settings from appropriate tab

Notes: Pumping not feasible for combinations of reservoir level and total demand that fall outside the "tab" areas
Pumps 7 and 8 can be used interchangeably (i.e., using pump 7 where it says 8 and visceversa will not affect operations)
Pumps 4 and 5 can be used interchangeably (i.e., using pump 4 where it says 5 and visceversa will not affect operations)



OFFICIAL USE ONLY
SENSITIVE

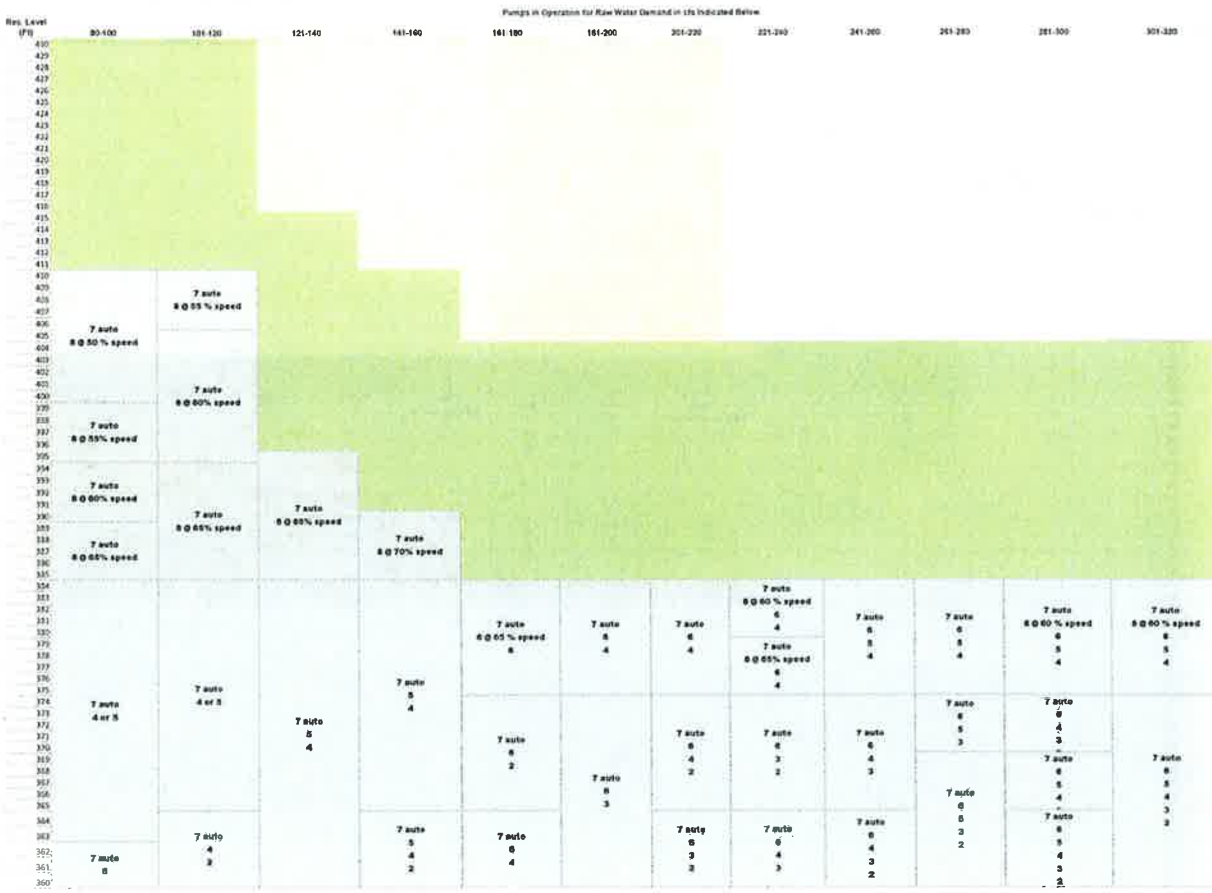


EXPLANATION
DEVICE
LOCATION
DESCRIPTION
24 V.D.C. SOLA
LOCATED RIGHT PANEL 1
IDEC PLC CPU
OFFICIAL USE ONLY
SENSITIVE
ALWAYS THINK SAFETY
FOLSOM PUMPING PLANT
STAND PIPE HIGH LEVEL— PUMP TRIP CONTROL
SCHEMATIC DIAGRAM

Folsom Pumping Plant

Proposed Pump Operation Schedule for Pumped Raw Water Deliveries

TAB 1: North Fork Surge Tank at 435 Feet (VFD Setpoint 0.65)



Folsom Pumping Plant

Proposed Pump Operation Schedule for Pumped Raw Water Deliveries

TAB 2: North Fork Surge Tank at 455 Feet (VFD Setpoint 0.76)

Res. Level (Ft)	Pumps in Operation for Raw Water Demand in cfs Indicated Below											
	80-100	101-120	121-140	141-160	161-180	181-200	201-220	221-240	241-260	261-280	281-300	301-320
430												
429												
428		7 auto										
427		8 @ 55 % speed										
426	7 auto											
425	8 @ 50 % speed											
424												
423												
422												
421												
420		7 auto										
419		8 @ 60% speed										
418	7 auto											
417	8 @ 50% speed											
416												
415												
414												
413	7 auto	7 auto										
412	8 @ 60% speed	8 @ 65% speed										
411												
410			7 auto									
409			8 @ 65% speed									
408				7 auto								
407				8 @ 70% speed								
406												
405												
404												
403												
402			7 auto								7 auto	7 auto
401								7 auto	7 auto	7 auto	8 @ 60 % speed	8 @ 60 % speed
400					7 auto	7 auto	7 auto	8 @ 60 % speed	6	6	6	6
399			5		8 @ 65 % speed	6	6	6	5	5	5	5
398			4	7 auto	5	4	4	4	4	4	4	4
397				5								
396				4								
395												
394										7 auto	7 auto	
393										6	6	
392										5	4	7 auto
391							7 auto	7 auto	7 auto	3	3	6
390					7 auto	7 auto	6	6	5		2	5
389					6	6	4	3	4	7 auto	7 auto	4
388					2	3	2	2	3	6	6	3
387										3	5	2
386										3	4	
385										2	3	
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Folsom Pumping Plant

Proposed Pump Operation Schedule for Pumped Raw Water Deliveries

TAB 3: Valve Throttling (North Fork Surge Tank at 455 Feet, VFD Setpoint 0.76)

Res. Level (Ft)	Pumps in Operation for Raw Water Demand in cfs Indicated Below											
	80-100	101-120	121-140	141-160	161-180	181-200	201-220	221-240	241-260	261-280	281-300	301-320
430												
429			**7 auto** **8 @ 65% speed** Discharge valves on pumps 7 and 8 throttled to get pressure readings on downstream gages that are 22 psi higher than pressure readings on suction side	**7 auto** **6** Discharge valves on pumps 6 and 7 throttled to get pressure readings on downstream gages that are 20 psi higher than pressure readings on suction side	**7 auto** **8 @ 65% speed** **6** Discharge valves on pumps 6, 7, and 8 throttled to get pressure readings on downstream gages that are 20 psi higher than pressure readings on suction side	**7 auto** **8 @ 65% speed** **6** Discharge valves on pumps 6, 7, and 8 throttled to get pressure readings on downstream gages that are 22 psi higher than pressure readings on suction side						
428												
427												
426												
425												
424												
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APPENDIX G

Reclamation Comments on First Draft of Report and WRE Responses

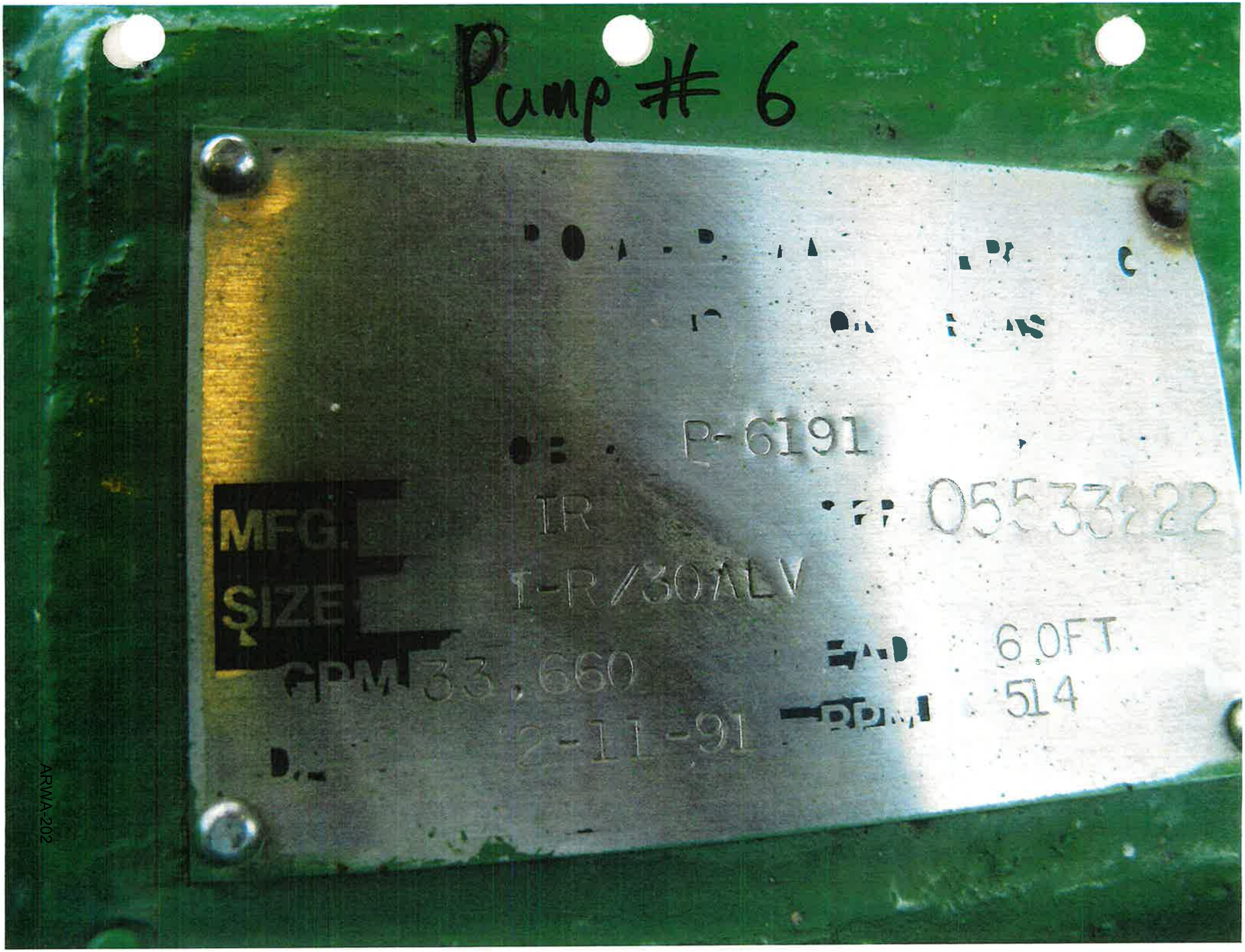
Folsom Pumping Plant System Capacity Evaluation

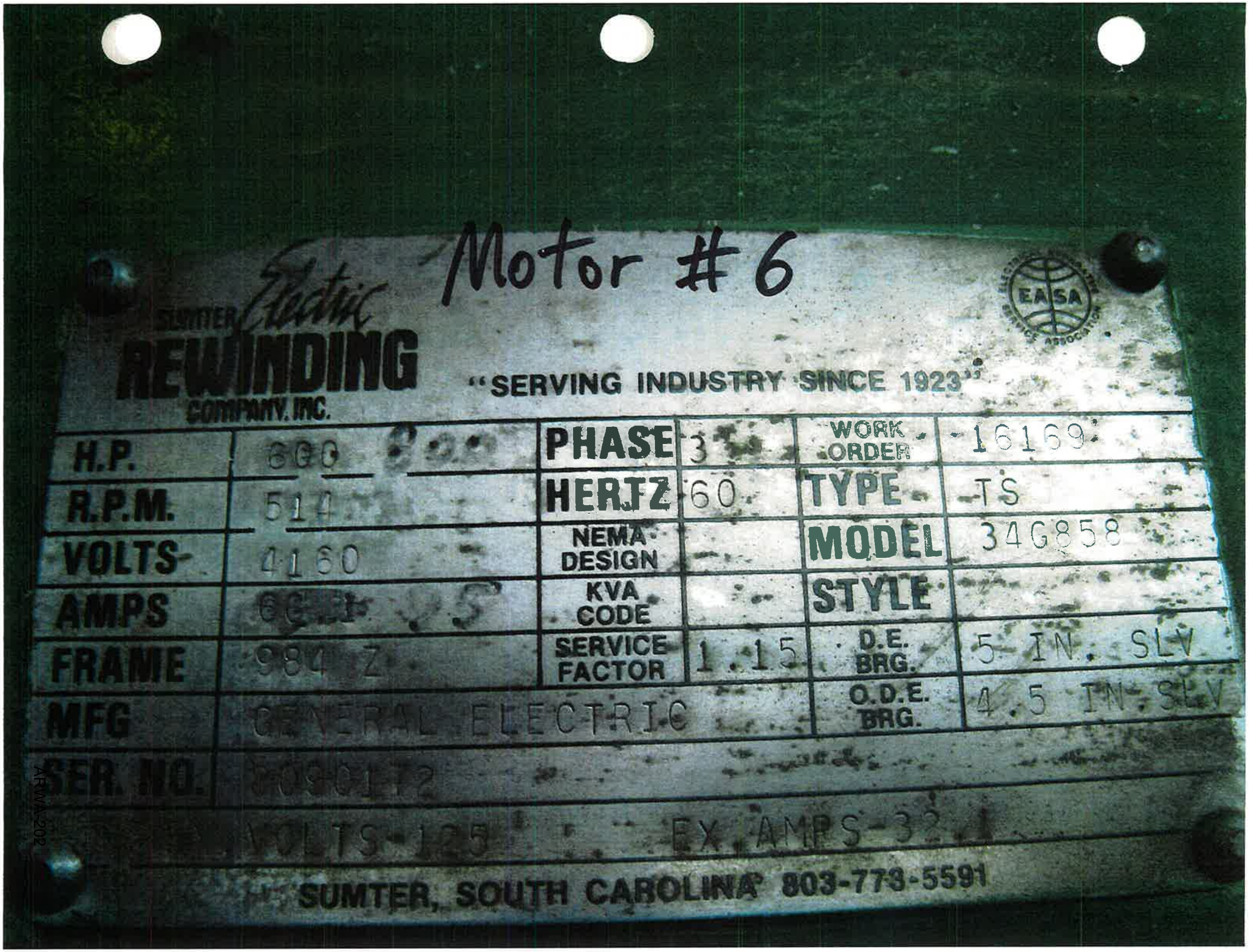
Comments on First Draft - Final Report

Ref. No.	Document	Section	Page	Full Comment Text	Comment Author	Source	Response	Response Author
1		Corrective Measures	ES-2	The report recommends installing butterfly valves on the discharge of pumps for purposes of throttling Reclamation typically does not recommend throttling pump discharges Attempts to regulate above 50-foot of differential head using butterfly valves has resulted in severe cavitation	Melavic	Reclamation	We understand Reclamation prefers not to throttle discharge valves, and it is a good practice when possible. But in this case it results in highly inefficient likely damaging, pump operation. Cavitation is a real problem (it is occurring now and that is one of our major concerns), and if throttling valves are installed they should be selected carefully. You might refer to http://www.valmatic com/pdfs/Cavitation_in_Valves_7-22-08 pdf for a good treatise on valve cavitation As indicated there, manufacturers typically recommend that no throttling be done if the cavitation index is under 2 5 The cavitation index for pumps 2 to 6 would be above 5 if operated at peak efficiency. Note that when operated at very low heads, with the discharge head set by the VFDs, the cavitation index falls <0	Gustavo Arboleda, WRE
2		3 3 2	3-5	Second sentence doesn't make sense The author most likely intends to state that Flowserve recommends operations within 20% of peak efficiency, but not 80% Within 20% is also consistent with Table 3-4	Melavic	Reclamation	We avoided saying "within 20% of peak efficiency" because that would imply that if peak efficiency was 80% then it would be acceptable to operate from 60% to 100% efficiency (or plus or minus 20 percentage points) But what we intend to convey is that the pumps should be operated at "no less than 80% of their peak efficiency, or "no less than 0 8 * Peak Efficiency." Therefore, if peak efficiency is 80%, then the pump should not be operated at less than 0 8 * 80% = 64% efficiency We will clarify wording in the report	Gustavo Arboleda, WRE
3		3 3 4	3 8	States that the pump will not operate at peak efficiency unless valve throttling is introduced. Throttling introduces a head loss and is energy inefficient because the pump will work against a higher head The pump may run smoother by adjusting the system curve closer to the pump curve, but it is not more efficient from the perspective of energy efficiency Was impeller diameter considered?	Melavic	Reclamation	Absolutely right. We say so in the report You might note that the summary table (Table 6-5) has it in capital letters - Would NOT reduce power consumption but rather increase it Changing impellers was considered We discussed it with Brian Zewe during our most recent trip to the plant. It was concluded that changing impellers repeatedly would be impractical	Gustavo Arboleda, WRE
		Appendix C	all	Appendix C does not include the pump curve for the E-Pump which is fairly identical to Pump #6	Melavic	Reclamation	Pump curve data on Page C-13 of Appendix C does indicate that the data applies to both Pump #6 and the Emergency Pump The actual pump curves are presented in Appendix D and there is no curve for the emergency pump We will change the caption under the Pump #6 curve to indicate that it applies to the emergency pump as well	Gustavo Arboleda, WRE

ARWA-202

Pump # 6




Motor #6
SUMTER Electric
REWINDING
COMPANY, INC.
"SERVING INDUSTRY SINCE 1923"
EASA
H.P.
600
R.P.M.
514
VOLTS
4160
AMPS
FRAME
984 Z
MFG
GENERAL ELECTRIC
SER. NO.
PHASE
3
HERTZ
60
NEMA DESIGN
KVA CODE
SERVICE FACTOR
1.15
WORK ORDER
16169
TYPE
TS
MODEL
34G858
STYLE
D.E. BRG.
5 IN. SLV
O.D.E. BRG.
4.5 IN. SLV
VOLTS 125
EX. AMPS
SUMTER, SOUTH CAROLINA 803-773-5591

ARWA-202


Emergency Pump
POWER MACHINE
HOUSTON
JOB #
SER.
MFG.
SIZE 30 ALV
EAD
GPM 33,700
RPM
DATE 8/27/91


Emergency
Motor
INDUCTION MOTOR
RATED HP 800
VOLTS 4160
AMP 86
SEC. VOLTS 125
HP
RPM
PHASE 3
FREQ
FRAME
SEC. AMP
RISE
LUBRICATION-OIL
CAUTION BEFORE INSTALLING OR OPERATING READ
INSTRUCTIONS
CONN. DIAG
WHEN ORDERING RENEWAL PARTS, GIVE THIS MOTOR MODEL
SER NO

RECLAMATION
Managing Water in the West

PM Details Report
PM: 12260 - FOU EMERGENCY LOW LEVEL PUMP INSPECTION_PM

Parent:					
Asset:				**Interruptible?**	No
Location:	FO-PMPPLT-PMP-EMER. PUMPING PLANT, EMERGENCY PUMP			**Outage Required?**	No
Routes:					
Reference:	FIST Vol. 4-1A Sec. 2.11-15 thru 2.11-19				
Frequency:		**FIST Frequency:**		**Variance?**	No
Last Start Date:	5/28/15	**Estimated Next Due Date:**	5/28/16	**Last Completion Date:**	8/7/15
Lead Craft:	CC-MECH	**FBMS Work Order:**			
Work Type:	PM	**Sub Work Type:**	O&M	**Current Counter:**	8
Supervisor:	CASTRO, JESSE			**Priority:**	3
Lead Person:				**Crew:**	
Next Job Plan:	25192				
PM Master:					

Job Plan: 25192, FOU PUMPING PLANT LOW LEVEL PUMP INSPECTION - MINOR (O&M)

Op	Op Description	Task Duration
10	JHA/HECP	0:30
	Energy Source Determination: ***WARNING*** CLEARANCE REQUIRED- 4160v pump disconnect (72 hr. notice required for outage request) Clearance Points: 1. 4160 VAC Disconnect Switch #2E01, (Ref. Drawing 485-218-1470) 2. Discharge Valve #31, Closed (Ref. Drawing 485-218-688 3. Suction Valve #30, Closed; (Ref. Drawing 485-218-688) Energy Source: Water pressure: pump suction Valve / discharge valve. Rotating Shaft: Motor/Pump Coupling Lockout: CLEARANCE- Required for pump/motor coupling maintenance Additional comments: ***CAUTION*** Execise Extreme CAUTION accessing the Low Level Pump Building- -Steps and Ladders may be wet and/or slippery -Grating Deck is HEADBANGER when using ladders past penstock *NOTIFY OPERATIONS UPON ARRIVAL AND DEPARTURE AT LOW LEVEL PUMP* -BEWARE of SNAKES or OTHER DANGEROUS WILDLIFE beneath penstocks - Watch for wasps during summer, bring spray Completed By ____________________ Reviewed By ____________________	

Qty	Craft	Hours	Item #	Store-room	Qty	Service Item	Qty	Tool #	Qty	Tool Hours
1	CC-MECH	5	0		0		0		0	0

ARWA-202

RECLAMATION
Managing Water in the West

Job Plan Details Report // **25192 - FOU PUMPING PLANT LOW LEVEL PUMP INSPECTION - MINOR (O&M)**

Status:	ACTIVE	**Interruptible?**	No	**Lead Craft:**	CC-MECH
Duration:	5.00			**Crew:**	
Planned By:				**Lead Person:**	

Job Plan Tasks

Sequence	Task ID	Description	Duration	Meter Name
	10	JHA/HECP	0.50	

Energy Source Determination: ***WARNING*** CLEARANCE REQUIRED- 4160v pump disconnect (72 hr. notice required for outage request) Clearance Points: 1. 4160 VAC Disconnect Switch #2E01, (Ref. Drawing 485-218-1470) 2. Discharge Valve #31, Closed (Ref. Drawing 485-218-688 3. Suction Valve #30, Closed; (Ref. Drawing 485-218-688) Energy Source: Water pressure: pump suction Valve / discharge valve. Rotating Shaft: Motor/Pump Coupling Lockout: CLEARANCE- Required for pump/motor coupling maintenance Additional comments: ***CAUTION*** Execise Extreme CAUTION accessing the Low Level Pump Building- -Steps and Ladders may be wet and/or slippery -Grating Deck is HEADBANGER when using ladders past penstock *NOTIFY OPERATIONS UPON ARRIVAL AND DEPARTURE AT LOW LEVEL PUMP* -BEWARE of SNAKES or OTHER DANGEROUS WILDLIFE beneath penstocks - Watch for wasps during summer, bring spray Completed By ______________________ Reviewed By ______________________

Sequence	Task ID	Description	Duration	Meter Name
	20	Drawings, Instructions and Tools	0.50	

Drawings or Instructions: 485-D- 551 485M-208-1966-1969 485M-218-392L 485M-218-748-L 485-208-(853-855) 485-218-(1478-1480) 485-218-688 *****NOTICE***** LOW LEVEL PUMP COUPLING is METRIC Materials: Chevron Ultra-Duty "O" Grade Grease with small button head. (Pump Bearings and Couplings) Rags Lectra clean solvent

Sequence	Task ID	Description	Duration	Meter Name
	30	Inspect/Clean/Lubricate	2.00	

____1. Operational check of bearings before lubricating (SEE OP 40, 1, a.) ____2. Lubricate pump bearings (Chevron Ultra-Duty "O" Grade) COUPLING: ***WARNING*--Rotating Shaft--CLEARANCE REQUIRED ON PUMP ____1. Inspect coupling for general condition, and hardware security ____2. Lubricate pump and motor coupling (Chevron Ultra-Duty "O") (5/16" Allen wrench and 7/16" NF long neck zirk fitting) (Grease adapter is METRIC THREAD- 16mm x 1.5 lead) DRAIN LINES: ____1. Clean out all pump drain lines

Sequence	Task ID	Description	Duration	Meter Name
	40	Inspect FOU Emergency Pipeline	3.00	

ARWA-202

RECLAMATION
Managing Water in the West

Job Plan Tasks

Sequence	Task ID	Description	Duration	Meter Name
		OPERATIONAL CHECK OF EQUIPMENT (Accomplish during pump ops test) ____1. Check Pump for Proper Operation ___a. Check pump bearings for noise / heat / vibration. Annotate any abnormalities ___b. Check pump /motor coupling for noise / vibration. Annotate any abnormalities ____2. Check pump packing for cooling water flow rate. ____3. 36" BFV (V30) and flex coupling ___ a. Operate valve open and closed ___ b. Inspect flex coupling for leakage and hardware security ____4. 36" Check Valve (V31) and flex coupling __ _ a. inspect and adjust flapper shaft packings ___ b. Inspect flex coupling for leakage and hardware security ____5. 36" Motor operated BFV (V32) ___a. Operate valve open and closed INSPECT AND SERVICE: VALVES, COUPLINGS, AND FLANGED CONNECTIONS ____1. 6" drain V51 ___ a. Operate valve open and closed ____2. 36" BFV (V29) and flex coupling ___ a. Operate valve open and closed ___ b. Inspect flex coupling for leakage and hardware security ____3. 36" BFV (V33) and flex coupling ___ a. Operate valve open and closed ___ b. Inspect flex coupling for leakage and hardware security ____4. 1" gate valve _ _ a. Operate valve V53 open and closed ___ b. Operate valve V54 open and closed ____5. Inspect reinforcement nozzle ____6. 24" manway at base of emergency standpipe ____7. 24" manway at top of emergency standpipe		

Work Assets

Location	Asset	Item	Descripton	Work Type
FO-PMPPLT-PMP-8			PUMPING PLANT, PUMP #8	
FO-PMPPLT-PMP-7			PUMPING PLANT, PUMP #7	
FO-PMPPLT-PMP-6			PUMPING PLANT, PUMP #6	
FO-PMPPLT-PMP-5			PUMPING PLANT, PUMP #5	
FO-PMPPLT-PMP-4			PUMPING PLANT, PUMP #4	
FO-PMPPLT-PMP-3			PUMPING PLANT, PUMP #3	
FO-PMPPLT-PMP-2			PUMPING PLANT, PUMP #2	
FO-PMPPLT-PMP-EMER			PUMPING PLANT, EMERGENCY PUMP	

Planned Labor

Task ID	Craft	Skill Level	Vendor	Contract	Labor	Qty	Hours	Rate	Line Cost
	CC-MECH					1	5.00	114.36	571.80
								Total Planned Labor Cost:	571.80

Grand Total for all Costs: **$571.80**

ARWA-202

RECLAMATION
Managing Water in the West

PM Details Report
PM: 11881 - FOU PUMP E MOTOR INSPECTION _EL

Parent:					
Asset:				**Interruptible?**	No
Location:	FO-PMPPLT-PMP-EMER PUMPING PLANT, EMERGENCY PUMP			**Outage Required?**	No
Routes:					
Reference:	FIST Vol. 3-4 Sec.2.2				
Frequency:		**FIST Frequency:**		**Variance?**	No
Last Start Date:	5/1/15	**Estimated Next Due Date:**	5/1/16	**Last Completion Date:**	10/8/15
Lead Craft:	CC-ELECT	**FBMS Work Order:**			
Work Type:	PM	**Sub Work Type:**	O&M	**Current Counter:**	8
Supervisor:	SEMONEIT, KARL			**Priority:**	3
Lead Person:				**Crew:**	
Next Job Plan:	15647				
PM Master:					

ARWA-202

RECLAMATION
Managing Water in the West

Job Plan: 15647, FOU - PUMPING PLANT PUMP EMERGENCY MOTOR INSPECTION - MINOR (O&M)

Op	Op Description	Task Duration
10	JHA/HECP	0:00

Energy Source Determination
Electrical Energy / Lockout Points:
4160 VAC Fused Contactor Disconnect Switch #2E01, Racked
Out And Removed From PCCE Cabinet, Locked Out Under Clearance;
208 VAC Motor Heaters / Panel Le Brkr. #8/10;
208 VAC/ V32 Discharge Valve / Panel LE Brkr. #2/4/6;
120 VAC Control Power And Cabinet Heaters;
120 VAC Test Power / Panel CPC Brkr. #1;
24 VDC Remote Control Power/ Panel 1 Rear, Fuse DB7;
64 VDC Alarm Power / Switch In PCCE Cabinet, Left Side;

Hydraulic Energy:
Water: V32 Discharge Valve, Locked Out Under Clearance;
Water: V30 Suction Valve, Locked Out Under Clearance;

Clearance Points:
1. Locking Device Placed In Position To Block High Voltage Shutters
   In Closed Position, PCCE Cabinet;
2. V32 Discharge Valve Disconnect Switch / Ervcs Panel;
3. V32 Discharge Valve Manual Valve Operator;
4. V30 Suction Valve (Optional);

Qty	Craft	Hours	Item #	Store-room	Qty	Service Item	Qty	Tool #	Qty	Tool Hours
1	CC-ELECT	2	0		0		0		0	0
1	CC-ELECT	2	0		0		0		0	0
1	CC-ELECT	2	0		0		0		0	0
1	CC-ELECT	2	0		0		0		0	0

ARWA-202

RECLAMATION
Managing Water in the West

Job Plan: 15647, FOU - PUMPING PLANT PUMP EMERGENCY MOTOR INSPECTION - MINOR (O&M)

Op	Op Description	Task Duration
10	JHA/HECP	0:00

Energy Source Determination
Electrical Energy / Lockout Points:
4160 VAC Fused Contactor Disconnect Switch #2E01, Racked
Out And Removed From PCCE Cabinet, Locked Out Under Clearance;
208 VAC Motor Heaters / Panel Le Brkr. #8/10;
208 VAC/ V32 Discharge Valve / Panel LE Brkr. #2/4/6;
120 VAC Control Power And Cabinet Heaters;
120 VAC Test Power / Panel CPC Brkr. #1;
24 VDC Remote Control Power/ Panel 1 Rear, Fuse DB7;
64 VDC Alarm Power / Switch In PCCE Cabinet, Left Side;

Hydraulic Energy:
Water: V32 Discharge Valve, Locked Out Under Clearance;
Water: V30 Suction Valve, Locked Out Under Clearance;

Clearance Points:
1. Locking Device Placed In Position To Block High Voltage Shutters
   In Closed Position, PCCE Cabinet;
2. V32 Discharge Valve Disconnect Switch / Ervcs Panel;
3. V32 Discharge Valve Manual Valve Operator;
4. V30 Suction Valve (Optional);

Qty	Craft	Hours	Item #	Store-room	Qty	Service Item	Qty	Tool #	Qty	Tool Hours
1	CC-ELECT	2	0		0		0		0	0
1	CC-ELECT	2	0		0		0		0	0
1	CC-ELECT	2	0		0		0		0	0
1	CC-ELECT	2	0		0		0		0	0

ARWA-202

RECLAMATION
Managing Water in the West

Job Plan Details Report // **15647 - FOU - PUMPING PLANT PUMP EMERGENCY MOTOR INSPECTION - MINOR (O&M)**

Status:	ACTIVE	**Interruptible?**	No	**Lead Craft:**	CC-ELECT
Duration:	2.00			**Crew:**	
Planned By:				**Lead Person:**	

Job Plan Tasks

Sequence	Task ID	Description	Duration	Meter Name
	10	JHA/HECP	0.00	

Energy Source Determination
Electrical Energy - Lockout Points:
4160 VAC Fused Contactor Disconnect Switch #2E01, Racked
Out And Removed From PCCE Cabinet, Locked Out Under Clearance;
208 VAC Motor Heaters / Panel Le Brkr. #8/10;
208 VAC/ V32 Discharge Valve / Panel LE Brkr. #2/4/6;
120 VAC Control Power And Cabinet Heaters;
120 VAC Test Power / Panel CPC Brkr. #1;
24 VDC Remote Control Power/ Panel 1 Rear, Fuse DB7;
64 VDC Alarm Power / Switch In PCCE Cabinet, Left Side;

Hydraulic Energy
Water: V32 Discharge Valve, Locked Out Under Clearance;
Water: V30 Suction Valve Locked Out Under Clearance;

Clearance Points:
1. Locking Device Placed In Position To Block High Voltage Shutters
   In Closed Position, PCCE Cabinet;
2. V32 Discharge Valve Disconnect Switch / Ervcs Panel;
3. V32 Discharge Valve Manual Valve Operator;
4. V30 Suction Valve (Optional);

Sequence	Task ID	Description	Duration	Meter Name
	20	Drawings, Instructions & Tools	0.00	

ARWA-202

Job Plan Tasks

Sequence	Task ID	Description	Duration	Meter Name
		Single line: 485-218-1470 Schematics: 485-218-1296; Wiring: 485-D-1867, 1868 485-218-1557 485-218-1466 Piping: 485-218-688 Manual: GEH-3102D Spare Parts: Motor Contactor Coil; Item No.: 1D300G102 Catalog Title: Coil, Motor Contactor Closing Location: 22A07D01 Field Contactor Coil; Catalog Title: Coil, Electrical, GE 15D22G2 Location: Vidmar Cabinet In VFD Room; Item No.: W5950008957483 Also At Nim. Location: 55A04B01 Bearing Oil: Catalog Title: Chevron GST ISO 46 OIL Location: Oil Shed Item No.: GSTISO46 Carbon Brushes, Slip Ring (4): Catalog Title: Brush, Carbon, For Pump 6 And E Slip Rings Item No.: 1010517260201 Location: 15C01B01;		

Sequence	Task ID	Description	Duration	Meter Name
	30	Inspect Slip Rings and Brushes	0.00	

ARWA-202

Job Plan Tasks

Sequence	Task ID	Description	Duration	Meter Name
		INSPECT SLIP RING AND BRUSHES		

_____ 1. Check condition of the slip ring for grooving and excessive wear. Clean, polish or resurface as needed.
_____ 2. Inspect the brushes for wear, freedom of movement, and for proper spring tension. Replace as necessary.
_____ 3. Inspect the brush pigtail connection for tightness and any damage
_____ 4. Inspect the slip ring area for signs of excess carbon, clean as needed.
_____ 5. Inspect for the presence of oil on the slip ring or brushes.
_____ 6. Check motor bearing oil level.

Sequence	Task ID	Description	Duration	Meter Name
	40	Inspect Motor Bearings and Heaters	0.00	

_____ 1. Check outboard bearing oil level.
_____ 2. Check inboard bearing oil level.
_____ 3. Verify motor heaters are hot while motor is shut down
_____ 4. Run motor and check bearing noise
_____ 5. Log current run hour meter reading ___________ HRS.

Sequence	Task ID	Description	Duration	Meter Name
	50	Megger HV Cables Motor Windings	0.00	

MEGGER HV CABLE AND MOTOR WINDINGS @ 5000 V
PERFORM 10 MIN. TEST FOR P. I.
_____ 1. Record and download megger readings. TEST #______
_____ 2. Final megger reading: __________ ____OHMS

Work Assets

Location	Asset	Item	Descripton	Work Type
	13048		PUMP #5 MOTOR	
	13067		PUMP #6 MOTOR	
	13039		PUMP #4 MOTOR	
	13030		PUMP #3 MOTOR	
	13096		EMERGENCY PUMP MOTOR	
	13021		PUMP #2 MOTOR	

ARWA-202

Planned Labor

Task ID	Craft	Skill Level	Vendor	Contract	Labor	Qty	Hours	Rate	Line Cost
	CC-ELECT					1	2.00	116.42	232.84
								Total Planned Labor Cost:	232.84

Grand Total for all Costs:	$232.84

ARWA-202

—Obsolete—

DESIGNER'S OPERATING CRITERIA

AND STANDING OPERATING PROCEDURE

FOLSOM DAM EMERGENCY PUMPING PLANT

FOLSOM DAM
AMERICAN RIVER DIVISION
CENTRAL VALLEY PROJECT
CALIFORNIA

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FOREWORD

Of primary concern to the Bureau of Reclamation is the safety of the general public and of the operating and maintenance personnel. Careful consideration also should be given to the conservation and protection of the Bureau of Reclamation's facilities. Therefore, safety, conservation, and protection should be the theme of the operating instructions.

The Reclamation Safety and Health Standards, Design Standards No. 1, Chapter 3, and OSHA Safety and Health Standards (29CFR 1910) are standards for safety. Please READ them and FOLLOW their instructions and recommendations.

The Avoidance of Accidents

is an Essential Requirement

of Every Operation.

DO NOT TAKE CHANCES

DESIGNERS' OPERATING CRITERIA

AND STANDING OPERATING PROCEDURE

FOLSOM DAM EMERGENCY PUMPING PLANT

FOLSOM DAM
AMERICAN RIVER DIVISION
CENTRAL VALLEY PROJECT
CALIFORNIA

CHAPTER I GENERAL

A. Location.

Folsom Dam and its appurtenant facilities are located approximately 2 miles north of Folsom in Sacramento County, California. The dam and its facilities are on the American River about 2 miles upstream of Folsom, California, as shown on the Location Map, Drawing No. 1 (485-208-949).

B. Purpose.

These operating criteria are confined to the operation of the emergency pumping plant. Folsom Dam was constructed by the Army Corps of Engineers for flood control and power generation as authorized by the American River Basin Development Act of 1949. The Folsom Powerplant was constructed by the Bureau of Reclamation for power generation. The Folsom Dam Pumping Plant (pumping plant) was constructed by the Army Corps of Engineers for irrigation uses on both sides of the American River and a water supply for Folsom Prison. Later, project water was delivered for domestic, municipal, and industrial uses for the city of Folsom, San Juan Suburban Water District, and the city of Roseville.

The Folsom Dam Emergency Pumping Plant (emergency pumping plant) was constructed to provide water to the cities of Roseville and Folsom, San Juan Suburban Water District, and Folsom Prison during drought years when Folsom Reservoir levels do not allow the delivery of water from the reservoir by gravity through the existing 84-inch pipeline to the pumping plant or by use of the primary pumping plant for water deliveries down to approximately 330 foot reservoir elevation.

CHAPTER II EMERGENCY PUMPING PLANT

A. Prefabricated Metal Building

1. Purpose - The purpose of the metal building is to protect the pumping unit from the elements while the pumping unit is installed in the emergency pumping plant at the toe of Folsom Dam. The pumping unit is not weatherproof.

2. Description - The building is a prefabricated rigid frame metal building manufactured by "United Structures of America, Inc." of Houston, Texas. The building was designed for 20 lb/ft^2 live roof load and 80 mph wind load in accordance with the Uniform Building Code. The building was furnished with the manufacturer's standard paint system, one access door and wall vents as specified.

3. Operation - The roof is equipped with four lifting lugs, one near each corner of the roof. The hole diameter in each lifting lug is 2 inches. When the pumping unit is to be moved in or out of the building, the roof rafters are to be unbolted from the columns and the roof lifted as a single unit, using all four lifting eyes concurrently. There are four bolted rafter-column connections, each consisting of six 1/2-inch diameter by 1-1/4-inch long ASTM Designation: A325 high-strength bolts.

When the roof is placed back on top of the building after being removed, the 24 attachment bolts shall be reinstalled and tightened in accordance with the instructions of the building manufacturer.

4. Maintenance - All metalwork should be inspected, cleaned, and repainted as necessary.

B. Emergency Pumping Plant.

1. Purpose - The emergency pumping plant conveys water from Penstock No. 1 to the 84-inch pipeline which feeds the pumping plant.

2. Description - The emergency pumping plant consists of a pumping unit, 36-inch diameter pipeline, and a 48-inch diameter standpipe. The pumping unit is Unit No. 7 of the pumping plant relocated within the emergency pumping plant. The pumping unit is shown on Drawing Nos. 485-208-846, 847, 848, and 853. The pipeline is a 36-inch-diameter steel pipe extending from Penstock No. 1 to the existing 84-inch-diameter pipeline and the new 48-inch-diameter standpipe. The top of the standpipe is at elevation 447.25. Manual 36-inch butterfly valves are located on each end of the pipeline to isolate the 36-inch pipeline from Penstock No. 1 and the 84-inch pipeline when the emergency pumping plant is not in use. There is a 36-inch swing check valve in the 36-inch pipeline and a 6-inch drainline downstream of the pumping unit. An electric motor-operated butterfly valve is located on the 36-inch pipeline downstream of the swing check valve. The pipeline is shown on Drawing Nos. 485-208-846, -853, -854 and -855.

3. Operation - The emergency pumping plant shall not be operated with the reservoir elevation above 330.00 or below 307.00. The initial design criteria were for operation of the emergency pumping plant between Elevation 340.00 and 325.00. The lower limit may be adjusted to as low as Elevation 307.00 depending on actual field conditions. The upper limit should be adjusted to Elevation 330.00 unless field conditions do not allow delivery through the existing system at this low an elevation. Unit No. 7 of the pumping plant shall be relocated to the emergency pumping plant. The pipe jig in the emergency pumping plant shall be removed to allow the installation of the pumping unit. All electrical and control connections shall be made as described in Section II.C.2. The butterfly valves shall be opened to allow water to fill the pipeline when the pumping unit is in the operating position. The butterfly valve at the Penstock No. 1 tap and the motor operated butterfly valve shall be opened to equalize the water level in the pipeline and standpipe with the reservoir water level. After the water level has equalized, open the 84-inch pipeline tap manual butterfly valve and close the motor operated butterfly valve. The pumping unit can be energized. After the pumping unit has reached full speed, the butterfly valve near the swing check valve will open automatically. The gate valve in the 84-inch pipeline upstream of the 36-inch pipeline connection then shall be closed.

After the reservoir has risen to El. 330.00 and the emergency pump is no longer needed, the gate valve in the 84-inch pipeline shall be opened and the pump shall be deenergized. The manual butterfly valves shall be closed and the motor operated butterfly valve opened. The 6-inch drainline shall be opened to drain the pipeline. After the pipeline is drained, the 6-inch gate valve and the motor-operated butterfly valve shall remain open to drain possible valve leakage. The pumping unit shall be removed from the emergency pumping plant and reinstalled within the pumping plant. All electrical and control connections shall be made. The pipe jig shall be reinstalled in the emergency pumping plant.

4. Maintenance - The pipeline shall be inspected for leakage when the pipe is filled with water. The valves shall be checked and operated annually. When the valves are operated, the reservoir elevation shall not be above 440.00. After the butterfly valves have been operated, the pipeline shall be drained. All maintenance of the valves shall be as recommended by the particular valve manufacturer. Every five years the tell tale ports at the taps for the manual butterfly valves at Penstock No. 1 and the 84-inch pipeline shall be checked for seepage.

C. Electrical System

1. Purpose - The purpose of the electrical system is to provide control of and electrical power for the pumping unit in the emergency pumping plant.

2. Description - The electrical system consists of one motor-pump unit, one motor-operated butterfly valve, one butterfly valve remote control panel, one sectionalizing switch, lighting panelboard, light fixtures, outlet receptacles and wiring, conduit and grounding systems.

The motor for the Emergency Pumping Plant pumping unit is an existing motor from Unit 7 of the pumping plant.

PUMPING PLANT

A 5kV, 200-amp, 3-phase, SF6 puffer-type switch, designated switch No. 1703 (UPB), is installed at Unit 7 in the pumping plant. The switch is to provide power to either Unit 7 in the pumping plant or the unit in the emergency pumping plant. New wires and conduits have been installed between this switch and the unit in the emergency pumping plant.

The existing excitation/control circuits for Unit 7 in the pumping plant will be connected to either Unit 7 in the pumping plant or the pumping unit in the emergency pumping plant. A new terminal strip at Unit 7 and new wires and conduit between the terminal strip and the pumping unit in the emergency pumping plant has been installed.

"A selector switch, designated "SS5", controls which valve (NORMAL or EMERGENCY) and valve controls are actuated, and controls whether the emergency standpipe level protection is activated through the PLC (Programmable Logic Controller) for pump shutdown. The switch is located within the No. 7 motor control cabinet.

MOTOR OPERATED BUTTERFLY VALVE

The butterfly valve with electric motor operator is installed in the 36-inch discharge line downstream of the swing check valve. The operator, designated "E-VCS", includes an electric motor, reduction gears, limit switch mechanism, torque limit switch mechanism, handwheel with declutching mechanism, position indicator, and reversing motor starter with motor overload relays, and a "LOCAL-REMOTE" selector switch, which must remain in the REMOTE position for automatic control.

EMERGENCY PIPELINE STANDPIPE

The 36-inch diameter pipeline is equipped with a standpipe. The standpipe has a pressure transducer connected to the PLC. The PLC is currently programmed to shut down the emergency pumping unit if the water level in the standpipe goes above Elevation 440.00 or below Elevation 325.00.

EMERGENCY PUMPING PLANT

The butterfly valve remote control panel, designated "E-RVCS", includes a disconnect switch, "AUTO-OFF-HAND" selector switch, "OPEN," "STOP," and "CLOSE" pushbuttons, indicator lamps, and an "EMERGENCY STOP" pushbutton which will shut down all running pumps when the SS5 switch is in the EMERGENCY position. The panel is installed within the emergency pumping plant."

A 120/208-volt, 100-amp, 3-phase, 4-wire panelboard with a 50-amp main breaker, designated panel "LE", is installed inside the emergency pumping plant, which provides power to the motor-operated butterfly valve and the motor space heaters. Supply for this panelboard comes from the pumping plant 120/208-volt AC distribution panel. Four 80-watt fluorescent light fixtures are installed inside the building. One 50-watt high pressure sodium light fixture, with a 120-volt high power factor ballast, controlled by a photocell is installed on the wall above the entrance door.

3. Operation - The Unit 7 motor and pump shall be removed from the pumping plant and installed on the pump frame in the emergency pumping plant. Electrical conductors shall be connected to the motor. The excitation/control circuits shall be disconnected from Unit 7 in the pumping plant and connected to the pumping unit in the emergency pumping plant. The sectionalizing switch shall be switched such that electrical power will be conducted to the pumping unit in the emergency pumping plant.

4. Maintenance - Maintenance of the electrical equipment shall be as recommended by the manufacturers of the equipment.

D. <u>Sequence Of Operation</u>

Note: See drawing 485-218-688 for valve designations.

1. Start Sequence

a. Remove the weather proofing from valve no. 31 (36-inch swing check valve located downstream of the emergency pump discharge). Ensure that the counter weights will clear the valve body, the cushion chamber small check valve and orifice on the bottom of the chamber are clear, the inside of the cylinder is lubricated with light oil, and that the valve mechanism is free to operate.

b. Close the 6-inch drain valve located downstream of the 36-inch swing check valve.

c. Open valve no. 32 (36-inch motor operated butterfly valve).

d. Open valve no. 30 (36-inch manual butterfly valve at FU-1 penstock tap).

e. The water level in the system will equalize with the reservoir level. Examine the system visually for leaks or movement.

f. Open valve no. 29 (36-inch manual butterfly valve located in the valve pit where the 36-inch emergency pump pipeline connects to the 84-inch pipeline).

g. After the emergency pump pipeline is watered up, close valve no. 32, verify the motor operated butterfly valve local selector is in the REMOTE position, and the E-RVCS valve control panel is in the AUTO position.

h. Verify that the emergency pump standpipe water level gage reads properly.

i. Verify disconnect 1703 (pump no. 7 motor feeder disconnect adjacent to the normal motor location) and switch SS5 (valve control switch in the pump no. 7 motor starter cabinet) are in the EMERGENCY pump position.

j. Close pump no. 7 motor starter disconnect 1701 and bump start pump and check that motor rotation is proper.

k. Start and operate the emergency pump and verify that valve no. 32 has opened.

l. Close valve no. 9 (60-inch gate valve at the 84-inch outlet from Folsom Dam).

2. Stop Sequence

a. Open valve no. 9.

b. If the system is to be shut down for a short time, the only requirement is to stop the pump. If the system is to be secured for the season or longer, continue with the following steps.

c. Close valve no. 29.

d. Close valve no. 30.

e. Open valve no. 32.

f. Open the 6-inch drain valve located downstream of valve no. 31.

g. After the system is drained the 6-inch drain valve and valve no. 32 shall remain open to drain possible valve leakage.

h. Clean, lubricate, and weather proof valve no. 31 (36-inch swing check valve).

3. Test Operation With The Reservoir Water Level Above 330.00 And Below 440.00

Same as Start Sequence above except keep valve no. 9 open (step 1).

CHAPTER III REFERENCE MATERIAL

A. Bureau of Reclamation Specifications

The following specifications are available for reference purposes in the Regional and Project Offices:

Number	Title
20-C0338	Emergency Pumping Plant - Phase I Folsom Dam Pumping Plant, American River Division, Central Valley Project, California
20-C0404	Emergency Pumping Plant - Phase III Folsom Dam Pumping Plant, American River Division, Central Valley Project, California

B. Bureau of Reclamation Publications

Paint Manual, Third Edition, 1976

Reclamation Safety and Health Standards

Design Standards No. 1, Chapter 3

OSHA Safety and Health Standards (29 CFR 1910), revised January 1976

Irrigation O&M Bulletin No. 60, "Pumping Plant Maintenance Schedules and Records," Revised 1970.

C. Manufacturers' Data

"DeZurik Installation, Operation & Maintenance Manual," DeZurik, A Unit of General Signal, Sartell, Minnesota.

"Operation Instructions," GA Industries Inc., Mars, Pennsylvania.

"Operating and Maintenance Manual," Joslyn Power Products Corporation, Alsip, Illinois.

D. Bureau of Reclamation Drawings

Latest revised prints of all Bureau of Reclamation drawings mentioned in the text have been included as part of these criteria.

NO.	DRAWING NO.	TITLE

GENERAL

1.
(a)485-208-949-- Location Map
(b)485-208-744-- Right Abutment Surface Treatment--Plan, Section and
Detail (location map only)

PUMPING PLANT

2.
(a)485-208-846 -- General Plan and Installation
(b)485-208-847 -- Grading Plan
(c)485-208-848 -- Tap Thrust Block and Pump Slab
(d)485-208-849 -- Tap Valve Access Stairway
(e)485-208-853 -- Pumping Unit Installation

PIPELINE

3.
(a)485-208-1147 -- General Plan and Tap Installation
(b)485-208-1148 -- Penstock Tap Installation
(c)485-208-1149 -- 84-Inch Pipe Tap Installation
(d)485-208-850 -- Pipe Anchorage Details
(e)485-208-854 -- 36-Inch Pipe Installation in Valve Vault

STANDPIPE

4.
(a)485-208-851 -- Standpipe Support Details
(b)485-208-855 -- Standpipe

ELECTRICAL

5.
(a)485-208-852 -- Electrical Installation

REFERENCE

6.
(a)485-D-65 -- Steel Penstocks--Plan and Profiles
(b)485-D-1293 -- Main Concrete Dam--Plan
(c)485-D-1294 -- Main Concrete Dam--Elevations
(d)485-D-1295 -- Main Concrete Dam--Typical Sections
(e)485-D-1324 -- Natoma Pressure Pipe Line--Plan, Profile and Details
(f)485-D-1415 -- Pumping Plant--Plan, Elevations and Details
(g)485-D-1416 -- Pumping Plant--Reinforcement Details
(h)485-D-1417 -- Pumping Plant--Equipment Arrangement
(i)485-D-1420 -- Pumping Plant--Power Conduit Plan

(j)485-D-1551 -- Pumping Plant Equipment--Mechanical--Pump Installation
(k)485-D-1552 -- Pumping Plant Equipment--Electrical Installation --Power Single Line Diagram
(l)485-D-1553 -- Pumping Plant Equipment--Electrical--Switchboard & Power Panel--Sheet 1
(m)485-D-1553 -- Pumping Plant Equipment--Electrical--Switchboard & Power Panel--Sheet 2
(n)485-D-1866 -- Folsom Pumping Plant--Electrical Installation--Pump and Valve Controls--Schematic Diagram
(o)485-D-1868 -- Folsom Pumping Plant--Electrical Installation--Electrical Power Panel
(p)485-D-2061 -- Folsom Pumping Plant--Bristol Recorder and Control Circuits--Schematic and Wiring Diagram
(q)485-208-562 -- Penstock Access Stairway and Walkway--General Plan and Elevations
(r) 485-218-688 -- Folsom Pumping Plant -- Water Distribution -- Flow Diagram

STANDARD DRAWINGS

7.
(a)40-D-5913 -- Valve Support
(b)40-D-6003 -- 18" Steel Ladder
(c)40-D-6022 -- 42" Two Rail Handrail--Details
(d)40-D-6248 -- Flange Support
(e)104-D-254 -- Equipment Enclosures
(f)104-D-286 -- Metal Conduit Bends

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Work Order: 3815978

3815978

CC (CENTRAL CALIFORNIA AREA OFFICE)

WO Description: FOU EMERGENCY PUMP MOTOR WINDING TEMP PROTECTION

Long Description: This work order is created to engineer pump motor winding over-temperature protection for Emergency Pump.

Location:	FO-PMPPLT-PMP-EMER (PUMPING PLANT, EMERGENCY PUMP)			**WO Priority:**	3
Asset:	-			**Asset Priority:**	4
FBMS Work Order:	R2358630	**Crew:**		**Calc Priority:**	7
WBS Element:	RX.03538841.3220000	**Reported Date:**	10/28/2015	**Work Type:**	MOD
Fund:	15XR0680A4	**Target Start:**	10/27/2015	**Sub Work Type:**	NONE
Reported By:	BRIZUELA, LEONARDO	**Target Finish:**		**Status:**	APPR
On Behalf Of:		**Scheduled Start:**		**Outage Required?:**	N
Supervisor:	LAWSON, DAVID	**Scheduled Finish:**		**PM:**	
Lead:	LY, HUE	**Actual Start:**	10/27/2015	**PM Compliance Range:**	
Lead Craft:	CC-EENG	**Actual Finish**			
Reference:	-				
Classification:	-				

Child Work Orders

No Child Work Orders

Safety Plan Information

No Safety Plan

Job Plan

No Job Plan

Tasks

Task ID	Description	Completed?
10	ESTIMATE JOB HAZARDS AND DEVELOPE JHA	☐
20	ENGINEER WIRING SCHEMATICS	☐
30	PROCURE MATERIALS	☐
40	INSTALL WIRING	☐
50	PROGRAM, TEST, AND COMMISSION OVER TEMP RELAY	☐
60	COMPLETE WORK ORDER, UPDATE FILE PRINTS	☐

Labor

Task	Craft	Labor	Qty	Hours
	CC-C&I		2.00	0.00
	CC-CCOPER		1.00	0.00
	CC-EENG		1.00	0.00
	CC-ELECT		2.00	0.00

Materials

No Material Records

Tools

No Tool Records

Work Log

No Work Log Records

Remarks

Lead Signature: ____________________ Date: ____________________

Lead Print Name: ____________________

Supervisor Signature: ____________________ Date: ____________________

Supervisor Print Name: ____________________

Total Time Charged: ____________________

Work Order: 2844691

CC (CENTRAL CALIFORNIA AREA OFFICE)

2844691

WO Description: FOU DROUGHT TEMPORARY PUMP STATION

Long Description: This WO is for all work associated with a temporary pump station that will be floated in the lake and connected to our current raw water system to feed the water customers.

Location:	FO-PMPPLT (PUMPING PLANT)			**WO Priority:**	3
Asset:	-			**Asset Priority:**	4
FBMS Work Order:	R3786519	**Crew:**		**Calc Priority:**	7
WBS Element:	RX.03538842.3221000	**Reported Date:**	01/23/2014	**Work Type:**	MOD
Fund:	16XR0680A4	**Target Start:**		**Sub Work Type:**	NONE
Reported By:	CASTRO, JESSE	**Target Finish:**		**Status:**	APPR
On Behalf Of:		**Scheduled Start:**		**Outage Required?:**	N
Supervisor:	KINSEY, ANDERS	**Scheduled Finish:**		**PM:**	
Lead:	SANTANA, JOSE	**Actual Start:**	10/30/2015	**PM Compliance Range:**	
Lead Craft:		**Actual Finish**			
Reference:	-				
Classification:	-				

Child Work Orders

No Child Work Orders

Safety Plan Information

No Safety Plan

Job Plan

No Job Plan

Tasks

No Planned Tasks

Labor

No Labor Records

Materials

No Material Records

Tools

No Tool Records

Work Log

No Work Log Records

Remarks

Lead Signature: ____________________ Date: ____________________

Lead Print Name: ____________________

Supervisor Signature: ____________________ Date: ____________________

Supervisor Print Name: ____________________

Total Time Charged: ____________________

Work Order: 3718893

CC (CENTRAL CALIFORNIA AREA OFFICE)

3718893

WO Description: FOU AUXILIARY PUMPING SYSTEM

Long Description:

The purpose of this project [Auxiliary Pumping System (APS)] is to provide a target total flow of 80 cfs split between Folsom Prison, City of Folsom, San Juan Water District and the City of Roseville under drought/low lake elevations. This is planned as a permanently installed project. The project is phased as follows:

Phase I	Initiation
Phase II	Planning
Phase III	Design and Development
Phase IV	Procurement
Phase V	Execution
Phase VI	Closeout

The project needs to be operational by 5/2/2016, and closeout by 9/30/2016.

Location:	FO-PMPPLT (PUMPING PLANT)			**WO Priority:**	3
Asset:	-			**Asset Priority:**	4
FBMS Work Order:	R3786519	**Crew:**		**Calc Priority:**	7
WBS Element:	RX.03538842.3221000	**Reported Date:**	08/24/2015	**Work Type:**	ENG
Fund:	16XR0680A4	**Target Start:**	07/31/2015	**Sub Work Type:**	MAJ MOD
Reported By:	ZEWE, BRIAN	**Target Finish:**	09/30/2016	**Status:**	APPR
On Behalf Of:		**Scheduled Start:**		**Outage Required?:**	N
Supervisor:	KINSEY, ANDERS	**Scheduled Finish:**		**PM:**	
Lead:	ZEWE, BRIAN	**Actual Start:**	07/31/2015	**PM Compliance Range:**	
Lead Craft:		**Actual Finish**			
Reference:	-				
Classification:	-				

Child Work Orders

No Child Work Orders

Safety Plan Information

No Safety Plan

Job Plan

No Job Plan

Tasks

No Planned Tasks

Labor

No Labor Records

Materials

No Material Records

Tools

No Tool Records

Work Log

No Work Log Records

Remarks

Lead Signature: ______________________ **Date:** ______________________

Lead Print Name: ______________________

Supervisor Signature: ______________________ **Date:** ______________________

Supervisor Print Name: ______________________

Total Time Charged: ______________________