

DEPARTMENT OF WATER RESOURCES

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October 1, 2020

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Via electronic mail: dvang@wwd.ca.gov

Westlands Water District Groundwater Pumping and Conveyance Project Draft Initial Study/Negative Declaration State Clearinghouse # 2020090040

Dear Mr. Vang:

The California Department of Water Resources (DWR) has reviewed the Draft Initial Study/Negative Declaration for Westlands Water District Groundwater Pumping and Conveyance Project dated September 2020 and provides the enclosed comments. DWR appreciates the opportunity to comment and looks forward to working with Westlands Water District as it moves forward.

If you have any questions, please contact me at Ted.Craddock@water.ca.gov or your staff may contact Daniel Whisman, Program Manager for California Aqueduct Subsidence Program, at Daniel.Whisman@water.ca.gov.

Sincerely,

Ted Craddock for

Ted Craddock
Deputy Director
State Water Project

Enclosure

Enclosure: Department of Water Resources' (DWR) Comments on the September 2020 Draft Initial Study / Negative Declaration (IS/ND) for Westlands Water District (District) Groundwater Pumping and Conveyance Project (Project)

15.10 Hydrology and Water Quality – Reliance Upon the Westside Subbasin Groundwater Sustainability Plan

1. The IS/ND Hydrology and Water Quality subsidence discussion and determination rely on compliance with the Westside Subbasin Groundwater Sustainability Plan (Westside GSP). While the Westside GSP contains important subsidence data, it does not contain sufficient information to be the sole basis for an adequate subsidence discussion for the Project. The IS/ND needs to provide an updated discussion on subsidence which considers recent subsidence studies and addresses DWR's concerns cited below prior to a finding of less than significant impacts on subsidence may be considered. DWR offers to provide recent subsidence studies and data as well as engineering and geological expertise to assist with strengthening the Project's subsidence analysis.
2. The IS/ND relies upon the Westside GSP measurable objective for land subsidence along the SLC as 0.1 feet per year. DWR's data shows the 0.1 feet per year objective is not a sustainable residual rate of subsidence and would significantly impact the functioning capacity and operational flexibility of the SWP. This degree of subsidence has resulted in considerable damage to the SLC for many years, impacting the overall functional capacity and operational flexibility of the State Water Project (SWP). The IS/ND determination that a 0.1 feet per year subsidence rate must be re-analyzed. DWR offers to provide available data for the re-analysis of the threshold for subsidence rate.

15.10 Discussion of Ground Subsidence Monitoring and Protection

3. The Westside GSP utilized Westside Groundwater Model (WSGM) which is calibrated to historical conditions from water years 1987 through 2015 to estimate groundwater level impacts of the Project. The IS/ND should extend the calibration and simulation period through 2019 to provide the most recent hydrology and agricultural water use conditions. This will provide the most accurate modeling results to support the Project's finding that it would be managed in a sustainable manner, including avoiding significant and unreasonable land subsidence that substantially interferes with surface land uses as required by SGMA. In addition to the Project's proposed well monitoring, land surface elevation surveys prior to, during, and after the Project should be incorporated into the proposed Project's ground subsidence monitoring plan. DWR offers its assistance in developing a complete monitoring plan.

15.10 Discussion of Water Quality Monitoring and Protection

4. The IS/ND in some instances only mentions that participating wells need to meet Title 22 standards while in other instances the IS/ND mentions the wells need to meet the Water Quality Monitoring Plan criteria. Wells participating in this Project need to meet the water quality criteria presented in the USBR's San Luis Canal Non-Project Water Pump-in Program – Water Quality Monitoring Plan for that year. As stated in Appendix B of the IS/ND, San Luis Canal Non-Project Water Pump-in Program 2020 Water Quality Monitoring Plan, the maximum allowable Electrical Conductivity changes in the SLC caused by the addition of non-project

groundwater should be less than 100 $\mu\text{S}/\text{cm}$ between check points 13 and 21 and not to exceed 700 $\mu\text{S}/\text{cm}$ at check point 21.

The IS/ND Hydrology and Water Quality section must include the requirements and potential protective actions to confirm that the groundwater from the participating wells meets the Water Quality Monitoring Plan by testing water before it is transferred via the SLC. Based on this monitoring data, USBR, the District and DWR would need to determine whether inflows to the SLC distribution system are of acceptable water quality.

15.10.1 Existing Setting - Subsidence Occurrences along the San Luis Canal

5. The IS/ND identifies two existing subsidence areas of concern within the Westside GSP that may significantly impact the SLC and asserts subsidence along the SLC is limited to these two discrete subsidence areas of concern. Both DWR and the Westside GSP identify additional significant subsidence in the Project area likely to impact the SLC. An updated subsidence impacts analysis that includes all the Westside GSP subsidence data occurring along the SLC is needed.

15.10.1 Existing Setting - Subsidence Monitoring Plan

6. The Project's subsidence monitoring plan needs to include a continuous monitoring program (i.e., a program utilizing interferometric data that expeditiously quantifies the impact when subsidence occurs and measures the growth of a subsidence hotspot and its impact on the SLC).

15.10.1 Existing Setting - Groundwater and Subsidence Monitoring Buffer

7. The IS/ND states that groundwater pumping related to subsidence would be managed and monitored in compliance with the Westside GSP. The Westside GSP asserts that a one-mile buffer is adequate for monitoring groundwater pumping based on the modeling included in its Appendix I. Most of the Project extraction wells are located very close to the SLC. Groundwater levels and ground subsidence must be monitored very closely to avoid subsidence which would result in significant adverse impacts on the structural integrity of SLC. A subsidence hotspot can form, expand, and subsequently damage the canal with great speed. The IS/ND should contain additional data analysis apart from that contained in the Westside GSP to demonstrate that a one-mile buffer will provide adequate protection to the SLC. DWR offers to work with the District to evaluate whether a one-mile buffer next to the SLC is sufficient.

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

CALIFORNIA AQUEDUCT SUBSIDENCE STUDY
San Luis Field Division
San Joaquin Field Division



June 2017

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

CALIFORNIA AQUEDUCT SUBSIDENCE STUDY

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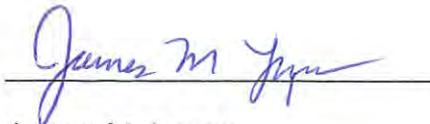
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State of California
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DEPARTMENT OF WATER RESOURCES
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CALIFORNIA AQUEDUCT SUBSIDENCE STUDY

ENGINEERING CERTIFICATION

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

A handwritten signature in blue ink, reading "James M. Lopes", is written over a horizontal line.

James M. Lopes
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Abbreviations and Acronyms

Aqueduct	California Aqueduct
bgs	below ground surface
BVPP	Buena Vista Pumping Plant
CAISO	California Independent System Operator
cfs	cubic feet per second
Corcoran clay	Corcoran Clay Member of the Tulare Formation
DAPP	Dos Amigos Pumping Plant
Delta	Sacramento-San Joaquin Delta
DMC	Delta-Mendota Canal
DOE	DWR Division of Engineering
DWR	California Department of Water Resources
gpm	gallons per minute
GPS	Global Positioning System
GSA	groundwater sustainability agency
GSP	groundwater sustainability plan
HEC-RAS	Hydrologic Engineering Center's River Analysis System
InSAR	Interferometric Synthetic Aperture Radar
JPL	Jet Propulsion Laboratory
NASA	National Aeronautics and Space Administration
NGVD	National Geodetic Vertical Datum
NGS	National Geodetic Survey
O&M	DWR Division of Operations and Maintenance
Reclamation	U.S. Bureau of Reclamation
SCADA	supervisory control and data acquisition
SGMA	Sustainable Groundwater Management Act
SJFD	San Joaquin Field Division
SLFD	San Luis Field Division
STA	station
SWP	State Water Project
UAVSAR	Unmanned Aerial Vehicle Synthetic Aperture Radar
USACE	U.S. Army Corps of Engineers
USGS	U.S. Geological Survey
Valley	San Joaquin Valley
WSE	water surface elevation

Executive Summary

In 2006, the San Luis Field Division of the California Department of Water Resources' Division of Operations and Maintenance (O&M) began to see a reduction in flow capacity through the California Aqueduct (Aqueduct) Pools 20 and 21. Subsidence had lowered portions of the Aqueduct and caused the concrete liner freeboard (the vertical distance between the water surface and the top of the concrete liner) to be reduced from its normal of 3 feet, to less than 1 foot. Subsidence had also decreased the ability to store water in those pools, which is normally done to add operational flexibility and to manage pumping at the Aqueduct's pumping plants. While subsidence has reduced the amount of freeboard and flow capacity at specific locations, contracted deliveries have not been curtailed through 2016. Additional work, to be addressed in the next phase of the project, will quantify how hydraulic limitations have impacted operations and will estimate future impacts to deliveries, based on forecasted subsidence rates.

The purpose of this project is to research and study past and present subsidence reports and data, and to understand and summarize the magnitude, location, and effects on the Aqueduct. This report summarizes the significant information found, and presents the results of the data that were analyzed.

Historic and Current Subsidence Summary

Subsidence in the San Joaquin Valley (Valley) has been recorded, analyzed, and studied since the 1920s. Before the construction of the Aqueduct in the mid to late 1960s, subsidence of 20 feet to 30 feet had been recorded in portions of the Valley. Subsidence factored into the planning and the design of the Aqueduct. Different alignments were considered to steer around areas that were thought to have a high potential for subsidence. Consolidation ponds were constructed to induce hydrocompaction (shallow subsidence) before Aqueduct construction. The Aqueduct embankments and concrete-lined freeboard were built higher than normally required to accommodate for future subsidence. During the planning of the Aqueduct, it was thought that most of the future subsidence would occur during Aqueduct construction, and that once the Aqueduct began delivering water the overdraft of groundwater would stop and subsidence would cease.

Subsidence continued during the construction of the Aqueduct (as planned) at an average rate of 6.4 inches per year in Pool 17 through Pool 20 of the San Luis Field Division, with the highest rate of 18.2 inches per year in Pool 19. In most areas, the higher embankments and freeboard were adequate to accommodate the active subsidence. But, in 1969 and 1970, the embankment and liner were raised 4 feet near Check 17 to restore the required amount of freeboard.

After water deliveries from the Aqueduct began, subsidence rates decreased to an average of less than 0.1 inch per year during the normal to wet hydrologic years. But, during dry to critical hydrologic years, subsidence increased to an average of 1.1 inches per year in Pool 17 through Pool 20. The slow, but ongoing, subsidence decreased the Aqueduct's concrete freeboard to a point that canal embankment and liner raises were required in 1982 in the San Luis Field Division, and in 1989 and 1996 in the San Joaquin Field Division.

The current drought has produced subsidence rates similar to those seen before the Aqueduct began delivering water. Between 2013 and March 2015, the average subsidence in Pool 17 through Pool 20 was

4.0 inches per year, with as much as 7.5 inches per year near the southern end of Pool 20. From March 2015 through August 2015, Pool 20 experienced an average of 6.9 inches of subsidence.

The review of collected data suggests there are three “bowls” of subsidence along the alignment of the Aqueduct in the San Luis Field Division and the San Joaquin Field Division. The first bowl is the largest, located in Pool 15 through Pool 18, in the San Luis Field Division. In that area, 1 foot to 9 feet of extra lined freeboard was constructed with the Aqueduct and then the liner was raised 24 inches to 58 inches in 1982 to maintain adequate freeboard. In that segment, the Aqueduct crosses five alluvial fans and the alignment arcs east into the Valley, to maintain the design gradient. The second bowl of subsidence is located in Pool 19 through Pool 21 in the San Luis Field Division. In that segment, 2 feet to 7 feet of extra freeboard was constructed with the Aqueduct. Portions of the liner in Pool 21 were raised as much as 36 inches in 1982. The alignment of this segment crosses one large alluvial fan and arcs out into the Valley. The third bowl is in Pool 23 through Pool 25 in San Joaquin Field Division. In that area, there was an extra 1.3 feet to 3.8 feet of lined freeboard built into the Aqueduct. Pools 22 and 23 were raised 30 inches and 39 inches, respectively in 1989. Pool 24 was raised 30 inches in 1996.

Operational Impacts

In addition to the observed reduction in capacity of the Aqueduct because of subsidence, there have been other impacts to the operation:

- Subsidence has decreased the availability of lined freeboard on long stretches of the Aqueduct. One of the functions of the lined canal freeboard is to provide erosion protection from waves in the Aqueduct. It also provides protection from fluctuations in the flow, where water levels can rise.
- The decrease in lined freeboard has decreased or eliminated the potential to store additional water in some pools. The Aqueduct freeboard is used as a reservoir, storing water during low-cost high-pumping period (nighttime) and drafting water for downstream delivery during high-cost low-pumping period (daytime). The reduced storage forces more pumping during more expensive periods to meet direct downstream demand.
- To be able to flow the required water out of the bowls of subsidence, the water needs to become deeper at the center of the bowl. At those locations there are canal structures that have become submerged. The overchute at Milepost 208.11 was designed to have 2 feet to 3 feet of clearance above the water surface and is now down in the water.
- Check structures 17 and 24, located at the bottom of two of the bowls, are being operated with most of their radial gates out of the water to minimize the head loss across them. Their radial gate trunnions are submerged under the checked up flow.
- The water surface elevation sensor gauges are on the downstream end of the pools. Because of the non-uniformity of the subsidence along the Aqueduct, in some pools, the upstream end is lower than the downstream end. In those cases, the downstream level sensor readings do not represent the true condition of the pool.

Hydraulic Impacts

For this study, in order to simulate, quantify, and project future hydraulic impacts to the Aqueduct caused by subsidence, a one-dimensional flow computer model was constructed using the Hydrologic Engineering Center's River Analysis System (HEC-RAS), which is the standard software for river and canal flow analysis. To construct the model, the as-built geometries of the Aqueduct of Pools 14 through

30 were entered into HEC-RAS. The Aqueduct cross sections were adjusted vertically based on the magnitude of subsidence from the latest survey data, 2015 for the San Luis Field Division, and 2013 for the San Joaquin Field Division. Two separate models were created based on the subsided model. One model with smooth, newly-formed concrete to model as-built conditions (best case scenario) and a second model with slightly higher friction factors to model the current conditions. The higher friction factor is consistent with an unlined, uniform channel, which is consistent with the observed sediment layer that has build-up on the canal liner and periodic debris on the invert.

For the model runs, the original design flowrates for each pool were used to evaluate the capacity of the subsided canal. The flowrates start at 13,100 cubic feet per second (cfs) in Pool 14 and step down to 5,350 cfs in Pool 30. The decrease in design capacity going downstream is the result of water being diverted at canal turnouts and side channels along the canal. After each model run, the water surface elevations were compared to the most recent survey elevations on the top of the concrete liner. The design criterion for lined freeboard was 3 feet for the San Luis Field Division and 2.5 feet for the San Joaquin Field Division.

The hydraulic model, using the current condition friction factors, showed that the water surface overtopped the concrete liner in four locations: Pools 17, 20, 24, and 25, totaling 9.5 miles of canal. The model showed that there were a total of 28.3 miles of canal (including the 9.5 miles overtopped) that had less than 1 foot of lined freeboard and 68.6 miles that had less than 2 feet of lined freeboard. In all, there were 95.8 miles of canal that had less than the designed amount of freeboard.

To evaluate the Aqueduct's current capacity, the flowrates in each pool were reduced from their design values until there was 0.5 foot of freeboard. A freeboard of 0.5 foot was selected as a minimum value to obtain a maximum flowrate within the canal. Under these conditions, waves or flow fluctuations could still overtop the canal liner. To obtain this minimal freeboard, the flowrates of 12 Aqueduct pools were reduced. Pools 16, 17, and 18's capacities were reduced by 400 cfs, from 11,800 cfs to 11,400 cfs (-3.4 percent). Pools 20 and 21's capacities were reduced by 1700 cfs, from 8,350 cfs to 6,650 cfs (-20.3 percent). This reduction in flow was carried through Pools 22, 23, and 24 because it was less than the design flowrate. To meet the 0.5 foot of freeboard, the flowrate in Pool 25 was reduced by 850 cfs, from 6,350 cfs to 5,500 cfs (-13.3 percent), which was also carried downstream to Pools 26, 27, and 28.

If more than 0.5 foot of freeboard is required, then the flowrates will need to be reduced further. A focused study was done for Pool 20 by using the model. The flowrate was reduced further to obtain the resulting freeboard. To have 1.5 feet of freeboard, the flowrate would need to be reduced by 2,350 cfs, from 8,350 cfs to 6,000 cfs (-28.1 percent). To obtain 2.5 feet of freeboard, the flowrate would need to be reduced by 2,900 cfs, from 8,350 cfs to 5,450 cfs (-34.7 percent). To obtain more than 2.6 feet of freeboard, the water surface in Pool 20 dropped below the pool's minimum operating level, which could leave some of the Pool 20 turnouts inoperable because of pump cavitation.

Future Study

Upon completion of Phase 1 of this study, it is recommended that Phase 2 commence as planned. Phase 2 will consist of evaluating future operational schemes of the Aqueduct to see if there are other ways to operate the Aqueduct to meet the required deliveries. Phase 2 will then take those schemes and develop preliminary alternatives to achieve them. The development of these preliminary alternatives will include

costs, constructability, implementability, and effectiveness. The next phase will also engage external stakeholders to ensure that the project is consistent with other projects, programs, and objectives. Lastly, the next phase will select a preferred alternative for future operation, if different, and provide scope, schedule, and costs for implementation.

Chapter 1. Introduction

1.1 Background

Land subsidence is a gradual settling or sudden sinking of the land surface from changes that take place underground. Between the 1920s and the construction of the California Aqueduct (Aqueduct) in the 1960s, several locations in the San Joaquin Valley (Valley) experienced nearly 30 feet of subsidence because of aquifer overdraft. As much as 20 feet of subsidence was recorded along the alignment of the San Luis Canal in the same time period. Anticipating that subsidence would continue during the relatively short time of construction, additional freeboard was added to portions of the Aqueduct. As anticipated, the newly built Aqueduct continued to subside during construction and through the beginning of operation. During design, it was predicted that when water deliveries from the Aqueduct began, ground water pumping would decrease and subsidence would slowly cease, and would not resume.

Since the construction of the Aqueduct there have been dry and critically dry water years when State Water Project water deliveries have been curtailed. During those periods there has been a corresponding increase in groundwater withdrawal, which has caused an increase in the rate of land subsidence. In areas relatively close to the Aqueduct, extensive groundwater withdrawal has caused areas of the ground surface to subside nearly 6 feet.

In 2006, the California Department of Water Resources (DWR) Division of Operations and Maintenance (O&M) started to see a decrease in conveyance capacity of the Aqueduct in the San Luis Field Division (SLFD). The flow capacity of the Aqueduct at Pool 20 was estimated at 7,300 cubic feet per second (cfs) (according to personnel from Operations and Control Office), a decrease of 1,000 cfs from its design capacity. Low spots in the canal liner, caused predominately by subsidence, limited the maximum water surface elevations (WSEs) at some locations. Aqueduct turnout operation limited the minimum WSEs at other locations.

During the current drought (2013 to 2015) parts of the Valley are sinking rapidly because of excessive groundwater pumping. This large-scale and rapid rate of subsidence is causing damage to water delivery infrastructures as documented later in this report using data from the UAVSAR flights (NASA 2015). Some areas are experiencing nearly 1.25 inches of sinking per month, which is similar to the subsidence rates that occurred before and during the construction of the Aqueduct.

1.2 Purpose and Scope

The purpose of this project is to collect, research and study previous studies, by DWR, and others, related to recent and historical subsidence along the west side of the Valley; use the historic survey data to understand and summarize the locations, magnitudes, and rates of subsidence; and describe and quantify how subsidence has impacted the operation of the Aqueduct. This report summarizes the significant information found, and presents the results of the data that was analyzed.

To achieve this, reports and studies were collected from DWR Archives, the U.S. Bureau of Reclamation (Reclamation) Library, University of California, Davis libraries, and the State Archives. Historic and new survey data were collected from DWR's Precise Surveys Unit. Survey data collection procedures were reviewed for understanding of changes in datum. Construction history was researched, repairs noted, and

information documented. Workshops were held with the Operations Control Office, the SLFD, and the San Joaquin Field Division (SJFD) to understand and document the current operation of the Aqueduct and how it is being affected by subsidence.

A series of large summary sheets were developed to summarize the subsidence, operational data, and geological information. The sheets were made for Pool 14 to Pool 21 (SLFD) and Pool 22 to Pool 38 (SJFD). The data illustrate historical and recent rates of subsidence and can be used in estimating potential future subsidence along the Aqueduct.

In order to quantify hydraulic impacts of subsidence, a hydraulic model was developed by DWR's Division of Engineering using the U.S. Army Corps of Engineers' Hydrologic Engineering Center's River Analysis System (HEC-RAS). The model covered the Aqueduct sections from the outlet of the Dos Amigos Pumping Plant (DAPP), Pool 14, to the forebay of the Buena Vista Pumping Plant (BVPP), Pool 30. The model factors include the 164.2 miles of Aqueduct, the turnouts, and pumping plants located within the study reach. One of the unknown factors is the Manning's n . As a result, the model was run multiple times by varying the n values from low to high.

Additional data were reviewed from the DWR contract with the National Aeronautics and Space Administration (NASA) Jet Propulsion Laboratory (JPL), where satellite-based Interferometric Synthetic Aperture Radar (InSAR) and Unmanned Aerial Vehicle Synthetic Aperture Radar (UAVSAR) measured relative changes in land surface elevation in the Valley from 2007 through 2015.

1.3 Description of Study Area

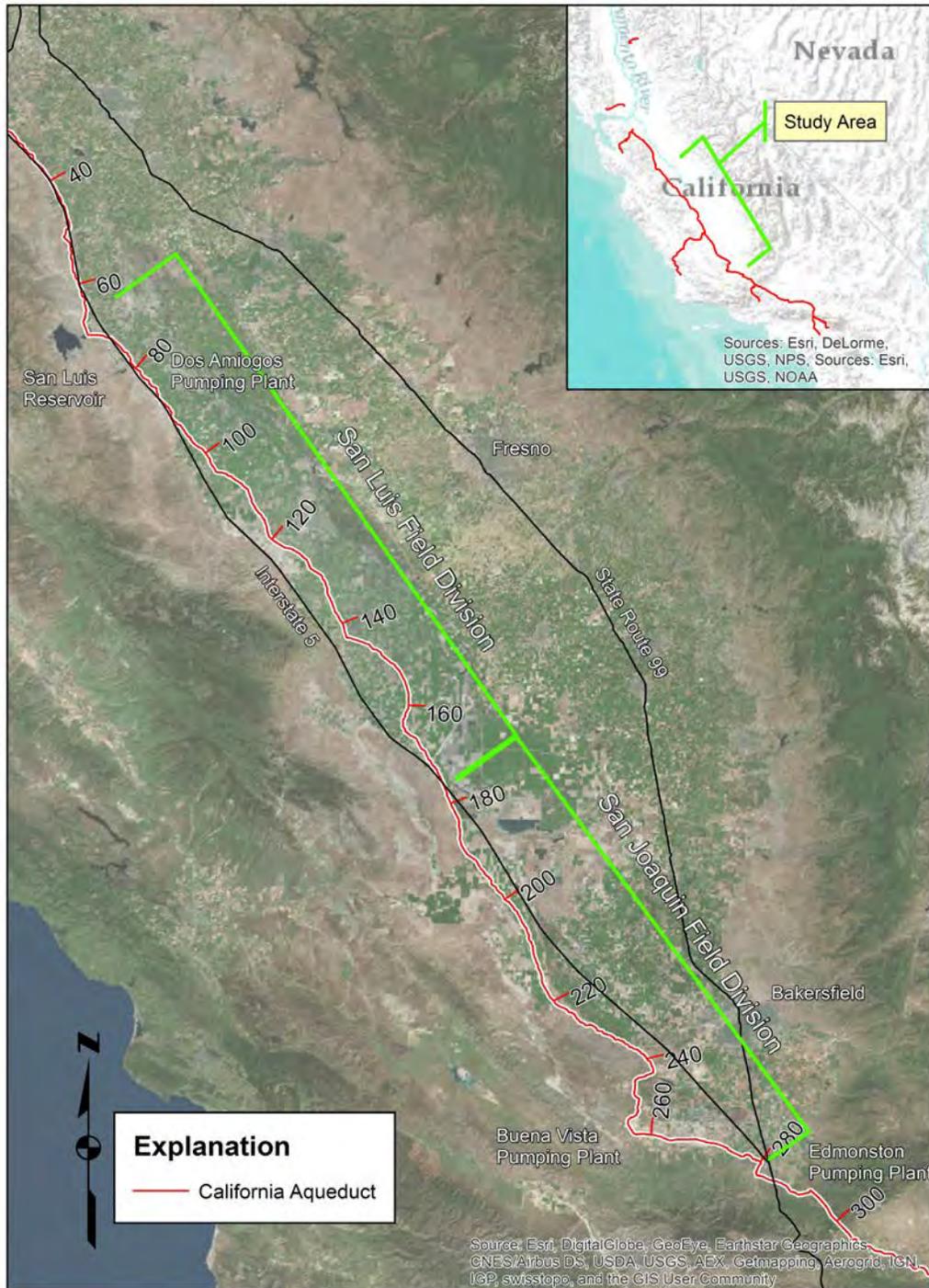
The Valley covers about 10,000 square miles and represents the southern part of the Central Valley, also known as the Great Valley of California. The Valley is bounded by the Sacramento-San Joaquin Delta (Delta) and Sacramento Valley on the north, the Sierra Nevada on the east, the Tehachapi Mountains on the south, and the Coast Ranges on the west.

Generally, the land surface has low relief. Its configuration is the result of millions of years of sediment deposition from the bordering mountain ranges. Most of the Valley is close to sea level with elevation increasing along the Valley margins. It is about 500 feet above sea level along the eastern edge of the Valley. Most of the western edge ranges from 50 feet to 350 feet above sea level.

The study area in this report includes sections of the Aqueduct from the San Luis Field Division and San Joaquin Field Divisions as shown in Figure 1-1. The area is dominated by agricultural land uses with several significant population centers and many other smaller communities. Climate in the study area is arid-to-semiarid and is generally characterized by hot, dry summers and damp, mild winters. Precipitation during an average year ranges from 5 inches to 20 inches. Most rain falls from November through April; about half falls from December through February. It should be noted that there are times when significant deviations from average conditions occur, which are expressed as droughts or floods.

Most of the streamflow is draining the western Sierra Nevada on the east side of the Valley. Historically, much of the precipitation in the Sierra Nevada has been snow. The San Joaquin River system conveys this water from the mountains to the Valley, where it takes a northerly turn into the Delta. The southern

Figure 1-1 Location of the California Aqueduct Subsidence Study in the San Joaquin Valley



Valley receives streamflow from the Kings, Kaweah, and Kern rivers. During many thousands of years, the natural flow of these rivers distributed networks of streams and washes on the slopes of the alluvial fans and terminated in topographically closed basins, such as Tulare Lake, Kern Lake, and Buena Vista Lake. The streams draining the drier eastern slopes of the Coast Ranges adjacent to the Valley are intermittent or ephemeral.

Surface water (provided by the Aqueduct) is used in the Valley when it is available. Essentially, all natural flows in area streams have been diverted for agricultural and/or municipal use (Moore and others 1990, Faunt 2009). The Valley also relies heavily on groundwater, which accounts for about 30 percent to 40 percent of the annual supply for agricultural and urban purposes (California Department of Water Resources 2003, Faunt 2009). During periods of drought, the groundwater usage increases to make up for the lack of surface water.

Chapter 2. Background — Land Subsidence

2.1 Previous Subsidence Studies

Land subsidence in the Valley was documented in many reports from 1956 to 1986 by the U.S. Geological Survey (USGS) and DWR as part of a cooperative subsidence program. Two important references from the cooperative subsidence program include:

1. USGS Professional Paper Series 437 A–I (Bull 1964, Lofgren and Klausning 1969, Bull 1972, Lofgren 1975, Bull and Miller 1975, Bull 1975, Bull and Poland 1975, Poland and others 1975, Ireland and others 1984), collectively referred to as the Poland Reports, after Dr. Joseph F. Poland, who led the program.
2. USGS Professional Paper Series 497 A–E and G (Johnson and others 1968, Meade 1964, Meade 1967, Meade 1968, Miller and others 1971, Riley 1970).

An additional report was published as part of the proceedings from the “Dr. Joseph F. Poland Symposium on Land Subsidence,” which was held in 1995 (Swanson 1998). It provides a brief update of land subsidence in the Valley from the early 1980s through 1995, when subsidence data collection in the Valley was sharply reduced. The downscaled monitoring focused largely on selected extensometers and surveys points along the Aqueduct and other important canals.

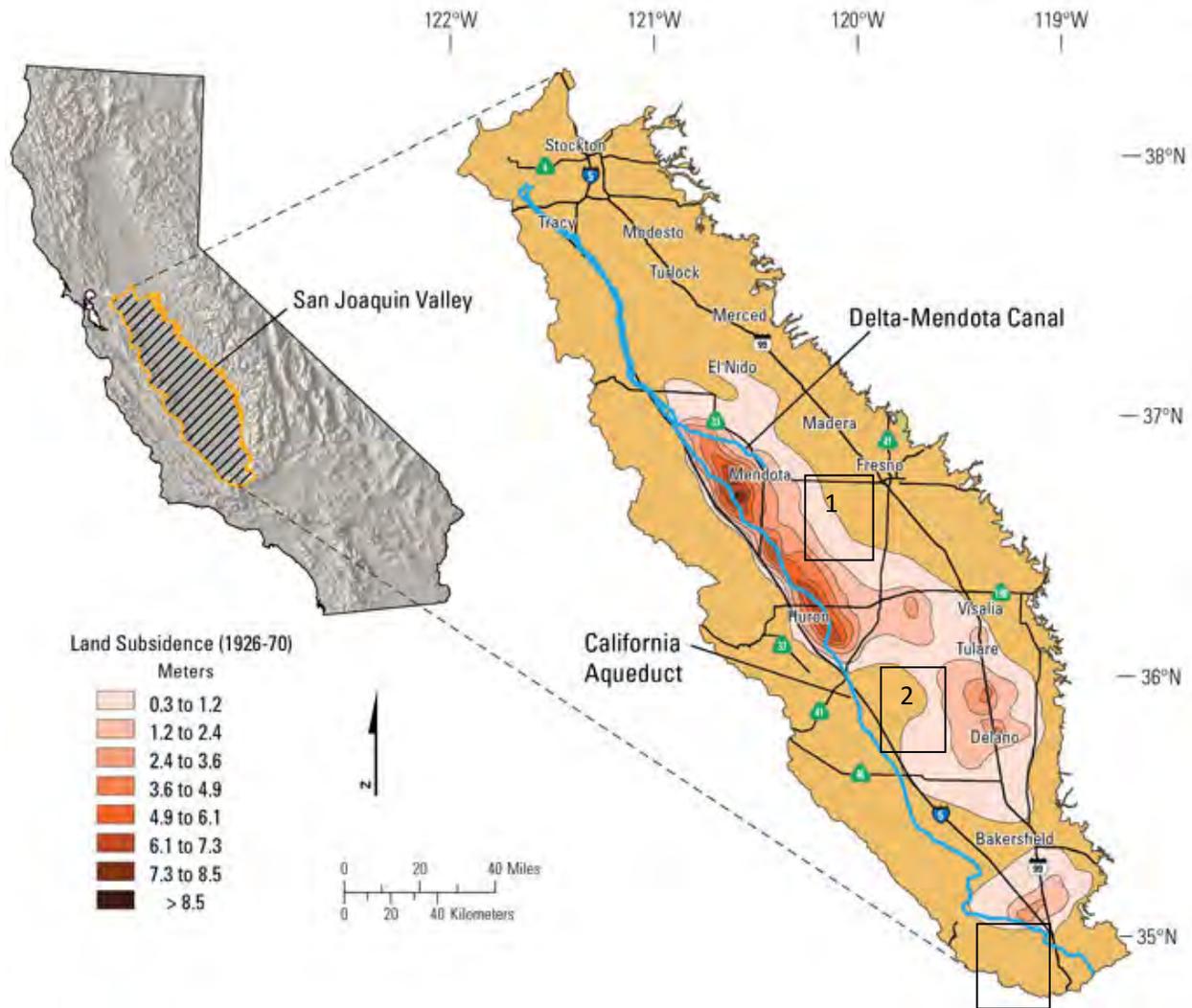
The previous studies documented that aquifer-system compaction brought on by groundwater overdraft causes the largest magnitude and areal extent of land subsidence in the Valley (Poland and others 1975, Ireland and others 1984, Farrar and Bertoldi 1988, Bertoldi and others 1991, Galloway and Riley 1999).

Land subsidence from groundwater pumping began in the mid-1920s (Poland and others 1975, Bertoldi and others 1991, Galloway and Riley 1999). By 1970, there had been more than 1 foot of land subsidence in about half of the Valley, or about 5,000 square miles (Poland and others 1975).

The areal extent of three principal areas of subsidence caused by groundwater withdrawals are delineated in the Poland Reports as (1) Los Banos-Kettleman City (2400 square miles), (2) Tulare-Wasco (1400 square miles), and (3) Arvin-Maricopa (700 square miles) (Poland and others 1975, Thomas and Phoenix 1976, Ireland and others 1984). These areas are shown in Figure 2-1.

The earlier reports showed that the historic withdrawal of groundwater from the unconsolidated deposits in the Valley caused widespread land subsidence, with some areas exceeding 28 feet, between 1926 and 1970 (Poland and others 1975) and reaching 29.5 feet by 1981 (Ireland 1986). They also documented that long-term groundwater level declines result in the one-time release of “water of compaction” from compacting silt and clay layers (aquitards). That results in a loss of aquifer-system storage (Poland and others 1975, Ireland and others 1984, Ireland 1986, Galloway and Riley 1999).

Figure 2-1 Location of the Areal Extent of Three Principal Areas of Subsidence Caused by Groundwater Withdrawals



The release of water of compaction in the Los Banos-Kettleman City area has been substantial. It has been estimated that by the mid-1970s, about one-third of the volume of water pumped from storage in this area came from compaction of fine-grained beds (Poland and others 1975, Faunt 2009). Hydraulic head declines of more than 400 feet in the confined part of the aquifer system have caused the inelastic (permanent) compaction of the clayey beds beneath the Corcoran clay.

Regional monitoring showed that subsidence was greatly slowed (or even stopped in some spots) in the Los Banos-Kettleman City area after the importation of surface water (particularly through the Aqueduct beginning in the early 1970s) and subsequent recovery of groundwater levels. But, the droughts of 1976-1977, 1987-1992, and 2007-2010 resulted in diminished deliveries of imported water. The lack of surface water resulted in increased groundwater pumping, which caused rapid lowering of groundwater levels. Consequently, the rate of subsidence increased (Swanson 1998, Galloway and Riley 1999).

2.2 Monitoring Subsidence

Several methods are available to monitor land subsidence. They include: (1) baseline and repeated surveys of benchmarks using precise-leveling survey methods, (2) Global Positioning System (GPS), (3) extensometers, and (4) InSAR.

Benchmarks or “geodetic stations” are used along a transect. The land surface elevations are initially surveyed and then re-surveyed over time to track changes in elevation at the benchmarks and monitor trends. The majority of surveying currently used to monitor subsidence across greater distances, or at a regional scale, is done with GPS. At a 95 percent confidence level, 95 percent of repeat measurements are expected to be within 0.13 foot (40 mm), or less, according to the USGS (2013).

Extensometers provide site-specific measurements of land subsidence. They are made of pipe or cable, anchored at the bottom of a borehole, through geologic layers that are susceptible to compaction of the ground surface. The device is connected to a recorder that measures changes of distance between the bottom of the borehole and the ground surface. If the inner pipe and casing go through the entire thickness of compressible sediments, then the device measures actual land subsidence. If, in addition, both groundwater levels and sediment compaction are measured, the data can be analyzed to determine which properties relate to the compaction. Changes in the trend of this data can also be used to predict future subsidence.

InSAR is a satellite- or aircraft-based remote sensing technique that uses radar to collect surface images. The individual radar images of a given area, collected at different times, are compared and interferograms are produced. The interferograms then are processed to show changes in land surface elevation.

2.3 Regional Perspective

Shortly after the completion of the Delta-Mendota Canal (DMC) by Reclamation in 1951, subsidence caused by withdrawal of groundwater in the northern Valley had begun to raise concerns on the impending threat to the DMC. Because of this threat to the DMC, and in order to help plan other major canals and structures proposed for these subsiding areas, the USGS, in cooperation with the DWR, began an intensive investigation into land subsidence in the Valley. The objectives were to determine the causes, rates, and extent of land subsidence, and to develop scientific criteria for the estimation and control of subsidence. The USGS concurrently began a federally funded research project to determine the physical principles and mechanisms governing the expansion and compaction of aquifer systems resulting from changes in aquifer hydraulic heads.

2.3.1 Groundwater Pumping and Water Level History

In 1955, about one-fourth (almost 8 million acre-feet) of the total groundwater extracted for irrigation in the United States was pumped in the Valley. As of 1960, water levels in the deep aquifer system were declining at a rate of about 10 feet per year. The maximum changes in water levels occurred in the western and southern portions of the Valley, in the deep confined aquifer system. More than 400 feet of water-level decline occurred in some west side areas of this system. Until 1968, irrigation water in these areas was supplied almost entirely by groundwater.

Water levels in the southeastern and eastern portions of the Valley were generally less affected because some surface water was also available for irrigation. In the water table aquifer, few areas exceeded 100 feet of water level decline, but a large portion of the Tulare Basin did experience declines of more than 40 feet. In some areas on the northwest side of the Tulare Basin, the water table aquifer rose as much as 40 feet because of infiltration of excess irrigation water.

Accelerated groundwater pumping and water level declines, principally in the deep aquifer system during the 1950s and 1960s, caused about 75 percent of the total volume of land subsidence in the Valley. By the late 1960s, less-expensive surface water was being diverted to agricultural interests from the Delta through federal and State water projects. This largely supplanted groundwater for crop irrigation.

With the imported surface water for irrigation, groundwater levels began a period of recovery, and subsidence slowed or was arrested over a large part of the affected area. Water levels in the deep aquifer system recovered as much as 200 feet in the six years from 1967 to 1974 (Ireland and others, 1984). It should be noted that although water levels began to recover in the deep aquifer system, aquifer-system compaction and land subsidence continued, but at a lesser rate.

For example, during the period from 1968 to 1974, water levels measured in an observation well near Cantua Creek recovered more than 200 feet, but the ground surface subsided another 2 feet. This apparent contradiction is the result of the time delay in the compaction of the aquitards in the aquifer system. It takes more time for pore-fluid pressures in the aquitards to equilibrate with the pressure changes occurring in the aquifers, than it takes for the aquifer to respond to the volume of groundwater being pumped (or not pumped) from the aquifer system.

The time needed for pressure equilibration of the aquitards depends largely on their thickness and permeability. It may take centuries for some aquitard pressure equilibration to occur, and the ultimate compaction to be realized. Swanson (1998) states that “Subsidence is continuing in all historical subsidence areas..., but at lower rates than before....”

Springhorn (2014) reported that accelerated groundwater pumping and water level declines, principally in the deep aquifer system during the 1950s and 1960s, caused about 75 percent of the total volume of land subsidence in the Valley. Springhorn (2014) looked at historical groundwater levels at discrete locations to evaluate recent groundwater lows compared to historical lows. Since spring 2008, groundwater levels are at all-time historical lows in most areas of the state, especially in the southern Valley. Some areas exhibit groundwater levels more than 50 feet below previous historical lows experienced sometime prior to 2000. There are many areas of the Valley where recent groundwater levels are more than 100 feet below previous historical lows and correspond to areas of recent subsidence, as shown in the compilation by Springhorn (2014).

The relationship between subsidence and water level decline is well documented near the center of subsidence west of Fresno. There, the increased pumping of groundwater began about 1940, and since that time water levels declined about 300 feet. In this specific area, 1 foot of subsidence has occurred for each 16 feet of decline in water level. Throughout the Valley, this ratio varies from about 8 feet of water level decline in areas of maximum subsidence to more than 25 feet of water level decline per foot of subsidence in perimeter areas. In the Tulare area, groundwater was intensively pumped until 1951 when

surface water deliveries began. Pumping levels declined 235 feet from 1930 to 1951, causing nearly 9 feet of subsidence, for an overall rate of 1 foot of subsidence for 26 feet of water level decline.

2.3.2 Predicted Subsidence with Surface Water Deliveries (Active versus Residual)

During the planning and design of the San Luis Canal, the historic subsidence rates in the vicinity of the canal alignment were analyzed and studied by Reclamation in the *Ultimate Amounts of Deep Subsidence* study (U.S. Bureau of Reclamation 1963A). That study focused on predicting future subsidence that would need to be incorporated into the design of the Aqueduct. In the study, two types of subsidence, active and residual, were combined to predict future subsidence.

Active subsidence was defined as the subsidence caused by direct pumping and groundwater overdraft. The general concept was that the active subsidence around the canal would cease when the Aqueduct started delivering water. It was recognized that active subsidence would continue during Aqueduct design, Aqueduct construction, during the beginning of operation, and during the construction of the distribution systems that would deliver water to those who were currently pumping.

Residual subsidence was an additional subsidence that occurs after the time of groundwater overdraft, as water pressures slowly reach equalization or drain in the clays that are being overdraft. The study assumed that the residual subsidence (called “lag”) would be 10 percent of the active deep subsidence. That percentage was added to the active subsidence that was anticipated during and after Aqueduct construction. For the study, residual subsidence was also added for all of the active subsidence that occurred before 1962-1963 (the time of the study).

The predicted subsidence results from the study varied along the length of the Aqueduct as the subsidence rates varied before 1962-1963 along the length of the Aqueduct. Section 4.2.1 discusses extra freeboard that was added to the design of the Aqueduct as it was constructed. Tables 4-1 and 4-2 list the extra freeboard per segment of the Aqueduct. As discussed in Section 4.4.1, a segment of the Aqueduct around Check 17 required two canal liner and embankment raises in 1969 and 1970 to prevent overtopping because the active subsidence during construction was greater than predicted. Another 27-mile canal liner and embankment raise was completed in 1982 to address areas where subsidence had reduced Aqueduct freeboard below standard design levels. Figures 6-1 and 6-2 show the location and magnitude of the extra freeboard, the location of canal raises, and the current elevation of the top of Aqueduct liner for comparison.

Chapter 3. Regional Setting

3.1 Regional Geology and Groundwater

The Valley occupies a trough created by tectonic forces related to the collision of the Pacific and North American plates. The trough is filled with marine sediments overlain by continental sediments (in some places thousands of feet deep) deposited largely by streams draining the mountains, and partially in lakes that inundated portions of the Valley floor from time to time. The marine and continental deposits form a large wedge that thickens from east to west and from north to south. More than half the thickness of the continental sediments is composed of fine-grained (clay, sandy clay, sandy silt, and silt) stream (fluvial) and lake (lacustrine) deposits susceptible to compaction.

The Valley was formed chiefly by tectonic movement during the late Tertiary and Quaternary, which included westward tilting of the Sierra Nevada block. Quaternary deformation has been principally along the southern and western borders of the Valley, where the marine and continental sediments are tightly faulted and folded, and stream terraces are elevated (Lofgren 1976). A detailed discussion of the geology of the Central Valley is given by Page (1986).

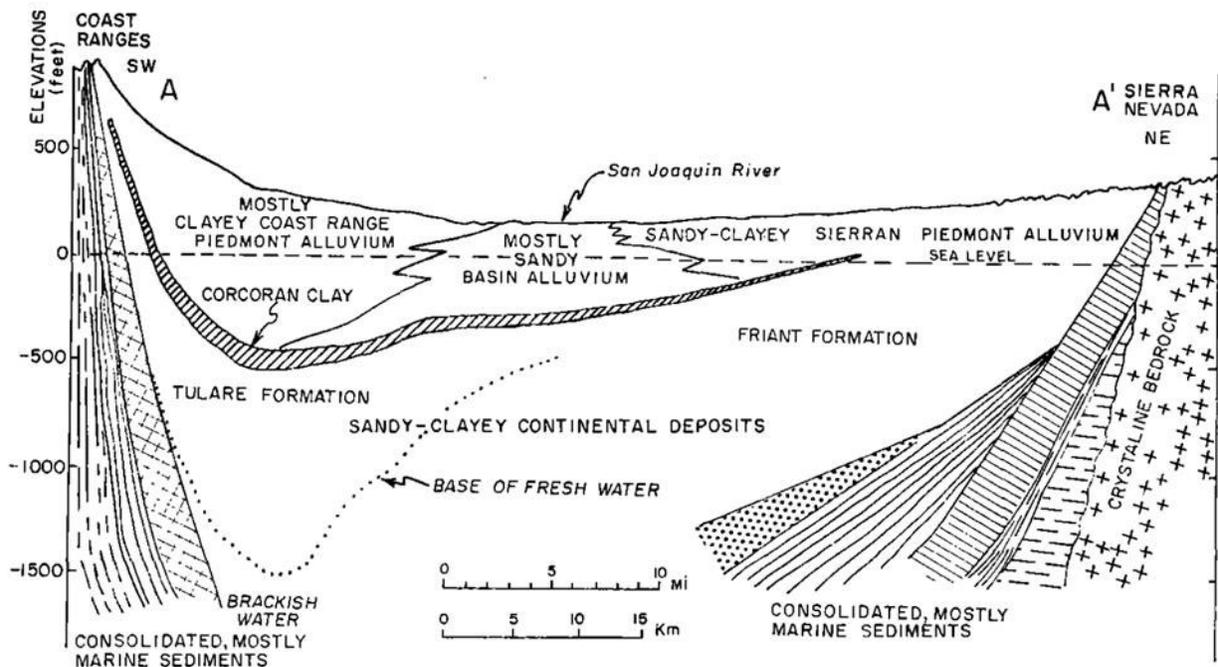
Three distinct groundwater bodies have been delineated in the Valley and include (with increasing depth): (1) unconfined to semi-confined freshwater in alluvial deposits overlying a widespread lacustrine confining bed known as the Corcoran Clay Member of the Tulare Formation (Corcoran clay), (2) freshwater confined beneath the Corcoran clay in alluvial and lacustrine deposits, and (3) saline water, contained primarily in marine sediments, which underlie freshwater throughout most of the Valley (Page 1986). A typical cross section across the Valley is shown in Figure 3-1.

Numerous lenses of fine-grained sediments are distributed throughout the Valley and generally constitute more than 50 percent of the total thickness of the valley fill. These lenses are highly compressible and account for nearly all aquifer-system compaction and subsequent land subsidence. In general, the lenses are not vertically extensive or laterally continuous, with the exception of the Corcoran clay.

The Corcoran clay was deposited during Pleistocene time when more than 6,000 square miles of the Valley was inundated by lakes. This diatomaceous/lacustrine clay is a low-permeability, areally extensive, lacustrine deposit (Johnson and others 1968) that varies in thickness from 0 feet to 200 feet (Davis and others 1959, Page 1986). A detailed description of sediment texture in the Valley aquifer system is given in Faunt (2009).

3.2 Causes of Subsidence/Mechanisms of Compaction

Land subsidence is a gradual settling or sudden sinking of the land surface from changes that take place underground. Four causes of subsidence are known to occur within the study area. In order of decreasing magnitude they are: (1) deep confined aquifer-system compaction caused by groundwater overdraft (water level decline), (2) hydrocompaction of moisture-deficient sediments, (3) fluid extraction from oil and gas fields, and (4) tectonic/crustal movement.

Figure 3-1 A Typical Cross Section of the Valley

The areas affected by subsidence caused by aquifer-system compaction and hydrocompaction are principally in the western and southern parts of the Valley where runoff from surface streams is minimal. Hydrocompaction and groundwater overdraft have significantly lowered the land surface in the Valley since about the 1920s.

Both shallow and deep processes accompanied the expansion of irrigated agriculture onto the arid, gentle slopes of the alluvial fans along the west side and south end of the Valley. Initially, the distinction between them and their relative contributions to the overall subsidence problem was not fully recognized. Both processes are described in the following paragraphs.

3.2.1 Groundwater Extraction (Deep Subsidence)

Deep subsidence is land subsidence that results from the compaction of deep, overpumped aquifer systems. The aquifer system of the Valley has both unconfined and confined parts caused by alternating layers of coarse- and fine-grained sediments. Water in the coarse-grained, unconfined, water table aquifers can be extracted or recharged easily. Compaction within these layers is generally minor and elastic, which is reflected as seasonal subsidence and rebound of water levels and the land surface.

Most of the subsiding area in the Valley is underlain by a continuous and extensive confining bed. Most of the groundwater overdraft and compaction caused by head decline in groundwater levels occurs in the confined aquifer system beneath this bed. The confining layer is the Pleistocene age, Corcoran clay. The boundary of the confining layer conforms fairly closely with the area affected by subsidence. For convenience, the unconfined to semiconfined water-bearing deposits above the Corcoran clay are referred to as the upper water-bearing zone. The confined system beneath it is referred to as the lower water-bearing zone.

Most water wells in the Valley exploit the deeper confined aquifers (lower water-bearing zones); withdrawal of water from them causes drainage of the fine-grained confining layers called aquitards. Significant water is available in the aquitards. But, those drain slowly and compact both elastically and inelastically. In general, if water levels are not drawn too low, when pumping ceases, water recharges the aquitards and their structure expands.

In many aquifers, groundwater is pumped from pore spaces between grains of sand and gravel. If an aquifer has beds of clay or silt within or next to it, the lowered water pressure in the sand and gravel causes slow drainage of water from the clay and silt beds. The reduced water pressure represents a loss of support for the clay and silt beds. Because these beds are compressible, they compact (become thinner), and the effects are seen as a lowering of the land surface.

Land subsidence from this process is permanent. If water levels are drawn too low, an irreversible compaction of the fine grained aquitards may occur. Recharging the aquifer until groundwater is returned to the original level will not result in an appreciable recovery of the land surface elevation (Galloway and others 1999, Bertholdi et al. 1991).

As mentioned in Chapter 2, three major subsiding areas related to groundwater withdrawal have been recognized in the Valley: (1) the Los Banos-Kettleman City area, largely in western Fresno County, (2) the Tulare-Wasco area, mostly in Tulare County, and (3) the Arvin-Maricopa area, in Kern County (Figure 2-1).

3.2.2 Hydrocompaction (Shallow Subsidence)

Hydrocompaction is compaction due to wetting. It is a near-surface phenomenon. Hydrocompaction produces land-surface subsidence through a mechanism entirely different from the compaction of deep aquifer systems.

In the 1940s and 1950s, farmers bringing virgin valley soils under cultivation found that standard techniques of flood irrigation caused an irregular settling of their carefully graded fields. That resulted in an undulating surface of hollows and hummocks with local relief around 3 feet to 5 feet. Where water flowed, or ponded continuously for months on susceptible soils, very localized settlements of 10 feet or more were witnessed.

The mechanisms and necessary conditions for hydrocompaction (initially known as “near-surface subsidence”) were investigated by means of laboratory tests on soil cores from depths of at least 100 feet, and by continuously flooded test plots equipped with subsurface benchmarks at various depths. In some cases, soil-moisture probes were used.

The combined field and laboratory studies demonstrated that hydrocompaction occurred only in alluvial-fan sediments above the highest prehistoric water table, and in areas where sparse rainfall and ephemeral runoff had never penetrated below the zone subject to summer desiccation by evaporation and transpiration. Under those circumstances, the initial high porosity of the sediments (often enhanced by numerous bubble cavities and desiccation cracks) is sun-baked into the deposits and preserved by their high dry strength, even as they are subjected to the increasing load of at least 100 feet of accumulated overburden.

In the Valley, such conditions are associated with areas of very low average rainfall and infrequent, flashy, sediment-laden runoff from small, relatively steep upland watersheds that are underlain by easily erodible shales and mudstones. The resulting muddy debris flows, and poorly sorted stream sediments, typically contain montmorillonite clay in proportions that cause it to act, when dry, as a strong interparticulate bonding agent.

When water is first applied in quantities sufficient to penetrate below the root zone, the clay bonds are drastically weakened by wetting, and the weight of the overburden crushes out the excess porosity. The process of densifying to achieve the strength required to support the existing overburden may reduce the bulk volume by as much as 10 percent.

Most of the potential hydrocompaction contained in anomalously dry, low-density sediments is realized as soon as the sediments are thoroughly wetted. The progression of a hydrocompaction event is controlled largely by the rate at which the wetting front of percolating water can move downward through the sediments. A site underlain by a thick sequence of poorly permeable sediments may continue to subside for months, or years, as the slowly descending wetting front weakens progressively deeper deposits. If the surface water source is seasonal or intermittent, the progression is further delayed.

Localized compaction beneath a water-filled pond or ditch often leads to vertical shear failure at depth between the water-weakened sediments and the surrounding dry material. At the surface, this process surrounds the subsiding flooded area with an expanding series of concentric tensional fissures having considerable vertical offset. This can lead to severe damage when it occurs beneath an engineered structure.

The hazards presented by hydrocompaction are somewhat mitigated by the fact that the process goes rapidly to completion with the initial thorough wetting, and is not subject to reactivation through subsequent cycles of decreasing and increasing moisture content. But, if the volume of water that infiltrates the surface on the first wetting cycle is insufficient to wet the full thickness of susceptible deposits, then the process will propagate to greater depths on subsequent applications, resulting in renewed subsidence. Also, an increase in the surface load such as a bridge footing or a canal full of water can cause additional compaction in pre-wetted sediments.

Subsidence due to hydrocompaction has occurred in two areas west and southwest of Mendota (Bull 1964a), a small area just south of Kettleman City, and in five areas south and southwest of Bakersfield (California Department of Water Resources 1964, pi. 2; Lofgren 1975, pi. 3C). The total area known to be susceptible to hydrocompaction is about 225 square miles, of which about 145 square miles is north of Kettleman City (Prokopovich 1970).

Studies undertaken in the mid-1950s led to a better understanding of hydrocompaction and to the identification of long reaches of the Aqueduct alignment that were underlain by deposits susceptible to hydrocompaction. Construction of the Aqueduct through many of these reaches was preceded by pre-wetting the full thickness of susceptible deposits beneath the Aqueduct alignment. Those measures added more than two years and tens of millions of dollars to the cost of the project (Prokopovich and Marriott 1983).

3.3 Historical Subsidence and Groundwater Levels

Land subsidence in the Valley was first noted near the Delano (Tulare-Wasco) area by I. H. Althouse (a consulting engineer) in 1935. Ingerson (1941) used a comparison of leveling data from 1902, 1930, and 1940 to create profiles used to describe the process of land subsidence in the Valley.

Accelerated groundwater pumping and water level declines, principally in the deep aquifer system during the 1950s and 1960s, caused about 75 percent of the total volume of land subsidence in the Valley.

By the late 1960s, surface water was being diverted to agricultural interests from the Delta and the San Joaquin River through federal reclamation projects, and from the Delta through the newly completed State Water Project. Less expensive water from the DMC, the Friant-Kern Canal, and the Aqueduct largely supplanted groundwater for crop irrigation.

At about the same time, groundwater levels began a dramatic period of recovery, and subsidence stopped, or at least slowed, to a lesser rate over a large part of the affected area. Water levels in the deep aquifer system recovered as much as 200 feet from 1967 to 1974 (Ireland and others 1984).

After 1974, land subsidence was demonstrated to have slowed or largely stopped. But, land subsidence remained poised to resume under certain conditions. Such an example includes the severe droughts that happened between 1976 and 1977 and between 1987 and 1991. Those droughts lead to diminished deliveries of imported water, which prompted some water agencies and farmers (especially in the western Valley) to refurbish old pumps, drill new water wells, and begin pumping groundwater to make up for cutbacks in the imported water supply. The decisions to renew groundwater pumping were encouraged by the fact that groundwater levels had recovered to near-predevelopment levels.

During the 1976-1977 drought, groundwater levels rapidly declined more than 150 feet over a large area, and subsidence resumed (after only one-third of the peak annual pumping volume of the 1960s had been reached). Nearly 0.5 foot of subsidence was measured in 1977 near Cantua Creek. That scenario was repeated during the 1987-1991 drought. It underscores the sensitive dependence between land subsidence and the dynamic state of imported water availability and use.

The sensitivity to a relatively small amount of renewed pumping causing such a rapid decline in water levels reflects the reduced groundwater storage capacity (lost pore space) caused by subsidence and aquifer-system compaction. It also demonstrates the nonrenewable nature of the “water of compaction,” which is available only on the first cycle of large scale drawdown.

Under natural conditions before development, groundwater in the alluvial sediments was replenished primarily by infiltration through stream channels near the Valley margins. The eastern Valley streams carrying runoff from the Sierra Nevada provided most of the recharge for Valley aquifers. Some recharge also occurred from precipitation falling directly on the Valley floor and from stream and lake seepage occurring there.

Over the long-term, natural replenishment was dynamically balanced by natural depletion through groundwater discharge, which occurred primarily through evapotranspiration and contributions to streams flowing into the Delta. The areas of natural discharge in the Valley generally corresponded with the areas

of flowing, artesian wells mapped in an early USGS investigation (Mendenhall and others 1916). Direct groundwater outflow to the Delta is thought to have been negligible.

It has been about 150 years since water was first diverted on the Kings River and more than 120 years after the first irrigation groups were established in the Valley. Intensive development of groundwater resources for agricultural uses has significantly altered the Valley's water budget. The natural replenishment of the aquifer systems has remained about the same, but more water has been discharged than recharged; the deficit may have amounted to as much as 800,000 acre-feet per year during the late 1960s (Williamson et al. 1989).

The time needed for pressure equilibration depends largely on the thickness and permeability of the aquitards. Decades to centuries may be required for most of the pressure equilibration to occur and for the ultimate compaction and subsidence to be realized.

Most of the surface water now being imported is transpired by crops or evaporated from the soil. The amount of surface water outflow from the Valley has been reduced compared to pre-development conditions. Groundwater in the Valley has generally been depleted and redistributed from the deeper aquifer system to the shallower aquifer system. That has created problems of groundwater quality and drainage in the shallow aquifer system, which is infiltrated by excess irrigation water that has been exposed to agricultural chemicals and natural salts concentrated by evapotranspiration.

Chapter 4. Aqueduct Construction

4.1 Background

During the planning stages of the Aqueduct, it was recognized that subsidence was a major problem in the Valley; a problem that would affect the design, construction, and operation of the canal. In the study area, 200 of the 280 miles of Aqueduct were in areas of subsidence. Three design measures (Sections 4.2.1, 4.2.2, and 4.2.3) were taken to deal with this problem: (1) extra freeboard was added at several locations along the canal to compensate for future subsidence until surface water became available, (2) pre-consolidation ponding was done in several areas that were susceptible to hydrocompaction, and (3) in areas of more severe subsidence, such as the area southwest of Mendota in Fresno County, where 28 feet of subsidence occurred between 1926 and 1972, Reclamation analyzed three different alignments from approximately Milepost 241 to Milepost 279 to determine which alternative would be most cost effective to construct and maintain.

4.2 Pre-Construction

Subsidence in the Valley has been a problem since at least the 1920s. But, it was not until DWR and the U.S. Department of the Interior became interested in constructing the California Aqueduct that sufficient interest, staffing, and funds became available for a full-scale investigation of subsidence in the Valley.

In May 1954, a joint conference in Washington, D.C. established a cooperative program to study subsidence. This conference led directly to the formation of the Inter-Agency Committee on Land Subsidence in the San Joaquin Valley, in Sacramento in December 1954. Members of the committee included representatives from DWR, USGS, Reclamation, U.S. Coast and Geodetic Survey, U. S. Army Corps of Engineers (USACE), Soil Conservation Service, California Division of Highways, and the University of California, Davis.

In January 1955, the committee began topographic mapping of some subsidence areas in the Valley. At about the same time, the U.S. Coast and Geodetic Survey was in the process of releveling many of the subsidence areas.

In July 1955, a field reconnaissance was made to determine the extent and nature of shallow subsidence. The results lead to a core drilling and sampling program in the fall of 1955. From late 1955 to early 1957, survey networks were established. During the summer of 1956, in addition to the survey and exploration work, large test plots were constructed to measure water intake and the amount of shallow subsidence.

Investigations by the committee are detailed in the *Progress Report on Land Subsidence in the San Joaquin Valley, California, through 1957*. The report focused on three main topics: (1) vertical control and topographic mapping, (2) shallow subsidence, and (3) deep subsidence.

Early in 1957, it became clear that increased emphasis in obtaining field data relating to subsidence would be required in order to meet the design and construction schedules of the Aqueduct. Accordingly, DWR's Division of Resources Planning organized a program for an expanded subsidence investigation. The Division of Design and Construction became responsible for DWR's subsidence investigations in June 1958.

4.2.1 Extra Freeboard

Subsidence along the Aqueduct (which affected 80 miles of the alignment) was considered during the design process. This was accomplished by estimating the amount of probable future deep subsidence in the area, and adding extra freeboard to the normal 3 feet. It was not possible to substantially reduce the heavy pumping of groundwater until the canal was built and imported water became available. As a result, the additional amount of freeboard built into the system, when subsided, would keep the operation and deliveries of the Aqueduct feasible.

The exact location of the canal alignment was not established before design. Because of that, no permanent leveling route was available along the Aqueduct. Subsidence contour maps (based on periodic leveling of an established leveling network) from 1956 to 1958, from 1958 to 1960, and 1:24,000 topographic quadrangles were used to estimate future subsidence. The first estimate was done in April 1961. Strip maps of the canal were drawn that showed the estimated topography at the beginning and at the end of construction. The assumption was made that the estimated subsidence rates would continue through construction. It was also assumed that once the canal was built and in operation there would be no more groundwater overdraft. As a result, there would be no more active subsidence; but a small (continued) amount of residual subsidence would persist.

A second estimate was done in January 1963, using the 1960 to 1963 subsidence rates, and using the same assumptions as the April 1961 estimate. The values for this particular estimate ranged from small traces to 15 feet of subsidence, with an average of about 8 feet of subsidence. Smaller estimates of subsidence were obtained from piezometric decline data and piezometric decline-subsidence ratios. Estimated subsidence ranged from traces to 10 feet, with an average of 5 feet. Comparing the 1963 estimates to those from 1961 (upstream of Milepost 139), the 1963 estimate was as much as 4.5 feet less than the 1961 estimate. Downstream of Milepost 139, both estimates were about the same.

Those estimates were used as guidelines in the design of the San Luis Canal to allow for future subsidence. The design provided from 1 foot to 9 feet of extra lined freeboard, on top of the normal 3 feet, from Milepost 87 (Dos Amigos Pumping Plant) to Milepost 172 (Kettleman City); Pools 16 to 20 being the most critical. In addition, between Mileposts 109.9 and 113.17 (an area of most subsidence) the invert was placed 2 feet above the designed grade. The thought was that in time, subsidence would eventually flatten out this area. Table 4-1 has a summary of pools with increased freeboard along the canal.

In general, the additional freeboard was enough to compensate for the subsidence encountered during construction and post construction except between Milepost 132.19 and Milepost 132.95 (near Check 17). The increased freeboard there was originally 6 feet, including 3 feet of normal freeboard. An extra 2 feet of freeboard was added twice, 2 feet in 1969 (Contract No. 200-C-752) and 2 feet in 1970 (Contract No. DC-6859), right after the end of construction (that area subsided about 5 feet during construction). An additional 3 feet of freeboard was added between Mileposts 128.76 and 132.19, and between Mileposts 132.95 and 137.02 (Contract No. DC-68.59) in 1970.

Table 4-1 Pools Designed with Extra Freeboard in the San Luis Field Division

Pool	Section Number	From Milepost	To Milepost	Water Depth ^a (ft.)	Extra Freeboard (ft.)	Total Freeboard ^a (ft.)	Lined Concrete (ft.)
13	1	70.90	86.47	32.8	1	4	36.8
14	2A	87.02	92.56	24.9	0	3	27.9
14-15	2B	92.57	97.52	24.9	1	4	28.9
15	2C	97.53	105.22	24.9	2	5	29.9
15	2D	105.25	108.50	24.9	5	8	32.9
16	3A	108.59	109.90	24.8	7	10	34.8
16	3A ^b	109.96	113.36	24.8	9	12	36.8
16	3A	113.36	118.63	24.8	7	10	34.8
16-17	3B	118.67	128.73	24.8	5	8	32.8
17	3C	128.76	132.19	24.8	6	9	33.8
17	3D ^c	132.19	132.95	24.8	7	10	34.8
18	3E	132.95	137.02	24.8	6	9	33.8
18	3F	137.02	143.29	24.8	3	6	30.8
19	4A	143.29	145.01	23.9	5	8	31.9
19	4B	145.05	152.70	23.9	7	10	33.9
19	5A	152.70	155.64	23.9	7	10	33.9
20	5B	155.70	164.69	23.9	5	8	31.9
21	5C	164.74	170.07	23.9	2	5	28.9
21	5D	170.11	172.40	23.9	0	3	26.9

Notes:

ft = feet

^aFigured for maximum water surface elevation.^bInvert grade raise due to subsidence.^c1969 and 1970 rehabilitations of freeboard within area.

Subsidence in the southern Valley was also observed for many years. Areas of deep subsidence near State Route 99 (15 miles south of Bakersfield), and areas between Arvin and Wheeler Ridge, and west to Maricopa, had subsided 1 foot to 4 feet according to data obtained from the U.S. Coast and Geodetic Survey, and mapping by the USGS. In an effort to prevent future subsidence from affecting the operation of the Aqueduct (and future water deliveries), extra freeboard was also added to that stretch of the Aqueduct. Normal values along that stretch of the Aqueduct were 2.5 feet of lined freeboard and 2.5 feet of unlined freeboard. An extra 2.5 feet to 6.2 feet of lined freeboard (on top of the 2.5 feet of normal freeboard used in the SJFD) was added in certain areas to compensate for future subsidence. The total freeboard added to accommodate subsidence in that stretch of the canal varied from 2.5 feet to 8.7 feet. Table 4-2 provides a summary of pools with increased freeboard along the canal.

During initial operations, only a couple of areas experienced subsidence. The first area was between Kettleman City and Avenal Gap (downstream of the Interstate 5 crossing). The primary road subsided

Table 4-2 Pools Designed with Extra Freeboard in the San Joaquin Field Division

Pool	Section Number	From Milepost	To Milepost	Water Depth ^a (ft.)	Extra Freeboard (ft.)	Total Freeboard ^a (ft.)	Lined Concrete (ft.)
22	8C/8D	172.44	182.39	26.9	0	2.5	29.4
22	8D ^b	182.39	184.82	26.9	2.5	5	31.9
23	9	184.84	194.94	25.3	1.3	3.8	29.1
23	9 ^b	194.94	197.05	25.3	1.9-2.3	4.4-4.8	29.7-30.1
24	10A	197.07	206.10	25.3	1.3	3.8	29.1
24	10A ^c	206.10	207.94	25.3	3.8	6.3	31.6
25	10A	207.96	210.31	25.3	1.3	3.8	29.1
25	11B	210.31	217.79	24	2.3	4.8	28.8
26	12D	217.81	224.92	23.3	2.5	5	28.3
27	12D/12E	224.94	231.73	23.3	2.5	5	28.3
28	12E	231.75	236.67	23.3	2.5	5	28.3
28	12E	236.67	238.11	21.9	0	2.4	24.3
29	13B	238.13	244.54	20.8	1	3.5	24.3
30	13B	244.56	249.45	20.8	1	3.5	24.3
30	14A	249.46	250.99	22.9	0	2.5	25.4
31	14A	251.18	254.15	22.4	0	2.5	24.9
31	14A	254.15	256.14	22.4	3	5.5	27.9
32	14A	256.18	261.72	22.4	3	5.5	27.9
33	14B	261.77	267.36	22.1	4.5	7	29.1
34	14B	267.43	270.22	22.1	4.5	7	29.1
34	14B	270.22	271.27	22.1	5.5	8	30.1
35	14C/15A	271.33	278.13	21.4	6.2	8.7	30.1
36	15A/16A	278.43	280.36	21.4	4	6.5	27.9
37	16A	280.77	283.95	21	2	4.5	25.5
38	16A	284.01	285.69	21	2	4.5	25.5
38	16A	285.69	287.09	21	0	2.5	23.5
39	16A	287.14	290.21	21	0	2.5	23.5
40	16A/17E	290.23	293.45	21	0	2.5	23.5

Notes:

^aFigured for maximum water surface elevation.^bCanal lining raised in 1989.^cCanal lining raised in 1996.

in the area where a 60-inch diameter culvert crossed under the road. There were some lining cracks, but everything appeared stable after the lining was patched and the sunken area was refilled. Another area of settlement was observed just downstream of the Buena Vista Pumping Plant at a 72-inch diameter concrete culvert undercrossing. The primary road also settled and there were cracks that formed on the

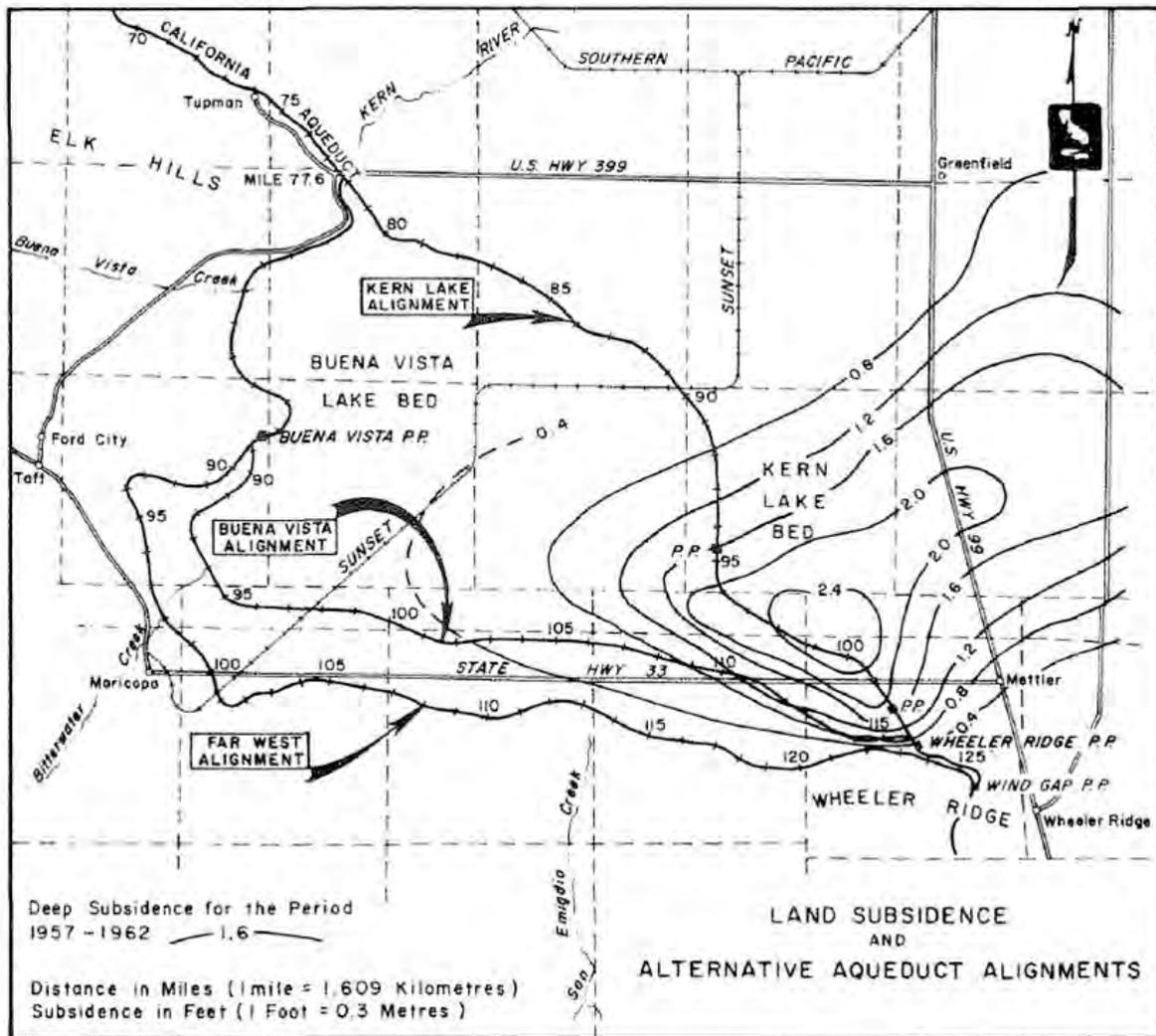
adjacent canal lining, similar to the ones that formed downstream of the Interstate 5 crossing. The cracks were filled with Allcrete and no more settlement was encountered. All other areas of the canal did not experience any notable subsidence during initial operations.

4.2.2 Alignment (Three Alternatives)

Three different canal alignments near the Buena Vista Lake Bed (between Tupman Road and Wheeler Ridge) were considered by the DWR Division of Design and Construction, for the San Joaquin section of the Aqueduct (Figure 4-1). The three alignments included: (1) the Kern Lake Alignment (25 miles long) which crossed through an area of subsidence, (2) the Buena Vista Alignment (38 miles long) which went around the west side of the area and crossed the less severe zones of the subsidence area on the south side, and (3) the Far West Alignment (46 miles long) which skirted the edge of the subsidence area.

The study comparing the three alternatives found that the Kern Lake Alignment would be the most affected by deep subsidence and the alignment would potentially subside locally as much as 18 feet from

Figure 4-1 Location of Three Canal Alignments near the Buena Vista Lake Bed



1970 to 2000. Based on the 1970 pre-subsidence design, the canal would require a trapezoidal shaped canal with 2:1 side slopes, 24 feet of water depth, and a top width of 120 feet. In order to accommodate the predicted subsidence (18 feet by the year 2000), the canal would require a trapezoidal shaped canal with 2:1 side slopes, 42 feet of water depth, and a top width of 192 feet to maintain the water surface at pre-subsidence 1970s level. That was an increase of 18 feet of water depth, and 72 feet of top width. That alternative would have quadrupled the earthwork, increased the cost associated with extra concrete lining, raised structures, and required additional planning to maintain water quality. There would also be additional engineering for other unforeseen problems, plus the costs of controlling plant growth, sediment, and seepage. The Kern Lake Alignment was deemed the least economically feasible.

The Buena Vista Alignment would have been less affected by deep subsidence. The study showed that this alignment would subside approximately 10 feet from 1970 to 2000. Although this alignment was 13 miles longer than the Kern Lake Alignment, the costs associated with subsidence for this alternative were much less.

The Far West Alignment, on the other hand, would not have been affected by deep subsidence. But, its 46-mile length would drastically increase its cost. When the three alternatives were compared, the Buena Vista Alignment was considered the most economical and was chosen over the other two alternatives.

4.2.3 Pre-Construction Ponds

There were three major alluvial fans (Cantua, Panoche, and Little Panoche Creek) that were of concern during pre-construction investigations for the San Luis Canal. The interfans, the areas between the three major alluvial fans, were known to be prone to hydrocompaction. One of the areas was 5.3 miles long, from Milepost 98 to Milepost 103. The other area was 15 miles long, from Milepost 113.5 to Milepost 128.5.

The hydrocompaction problems in two areas between Mileposts 98 and 103 (Little Panoche Creek and Panoche Creek interfan), and between Mileposts 113.5 and 128.5 (Panoche and Cantua Creek interfan) were minimized by pre-construction ponding along the canal alignment. The area between Mileposts 98 and 103 (northern area) experienced an average of three feet of subsidence. The area between Mileposts 113.5 and 128.5 (southern area) experienced the most subsidence, an average of 5 feet.

A total of 128 pre-construction consolidation ponds were constructed. They measured 400 feet to 425 feet wide, varied in length, and were designed to saturate both the canal prism and the areas where the canal embankments were going to be placed. They were generally flooded 1.5 feet to 2 feet deep for 12 to 18 months. A series of gravel packed infiltration wells 75-, 100-, and 125-foot deep (on a 100-foot grid) were also installed to speed up water infiltration in drier areas. The northern subsidence area was done under Reclamation Specification No. DC-5821. The southern area was done under Reclamation Specification Nos. 200C-564 and DC-5862.

The portion of the Aqueduct in the southern part of the Valley also dealt with shallow subsidence-prone areas with pre-construction ponding along the proposed alignment. Major construction contracts by DWR were done to construct pre-consolidation ponds in six areas: (1) the vicinity of Arroyo Pino Creek (Milepost 177.4 to Milepost 177.7), (2) Lerdo Highway to Tupman Road (Milepost 215.6 to Milepost 238.9), (3) Buena Vista Pumping Plant to Wheeler Ridge Pumping Plant (Milepost 255.7 to

Milepost 279.2), (4) Sunset Railroad to Maricopa Highway (Milepost 261.6 to Milepost 274.3), (5) Wheeler Ridge Pumping Plan to Standard Oil Road (Milepost 279.2 to Milepost 283.9), and (6) Standard Oil Road to Grapevine Creek (Milepost 283.9 to Milepost 288.7). Table 4-3 has details.

Table 4-3 Pre-consolidation Ponds in the San Joaquin Field Division

Milepost		Length (miles)	Start of Construction	End of Construction	Spec.	Start of Ponding	End of Ponding	Maximum Subsidence ^a (feet)
From	To							
177.4	177.7	0.3	June 17, 1965	July 1, 1965	65-16	July 7, 1965	Oct.4, 1965	0.7
215.6	238.9	23.3	Nov.18, 1964	Nov. 10, 1965	64-41	June 1965	April 1967	1.23
255.7	279.2	23.5	Jan. 2, 1965	March 11, 1966	64-46	Sept. 1, 1965	March 10, 1968	9
261.6	274.3	12.7	March 5, 1966	July 29, 1966	66-12	Aug. 11, 1966	March 10, 1968	7.5
279.2	283.9	4.7	June 22, 1964	April 2, 1965	64-21	Aug. 25, 1964	April 11, 1968	5.4
283.9	288.7	4.8	Nov. 8, 1963	March 31, 1965	63-32	Aug.25, 1964		1.7

Notes:

Spec. = specification number

^aMaximum subsidence after ponding.

The first stretch of pre-construction ponding was done in the vicinity of Arroyo Pino Creek, between Mileposts 177.4 and 177.7. It consisted of constructing four consolidation ponds with turnouts, in 0.3 mile of the canal prism and embankments. Construction started on May 5, 1965, and was completed on July 1, 1965 (Specification No. 65-16). Ponding operations began on July 7, 1965, and ended on October 4, 1965. As much as 0.7 foot of subsidence occurred at this location after ponding.

The second stretch of pre-construction ponding was done between Lerdo Highway and Tupman Road between Mileposts 215.6 and 238.9. It consisted of constructing 273 subsidence ponds with infiltration wells, along 23.3 miles of the Aqueduct. The ponds were usually 200 feet wide by 500 feet long, and typically each contained 15 gravel-packed infiltration wells, 40 feet to 80 feet deep. Construction started on November 18, 1964, and was completed on November 10, 1965 (Specification No. 64-41). Ponding operations began in June 1965, and ended in April 1967. As much as 1.23 feet of subsidence occurred at this location after ponding.

The third stretch of pre-construction ponding was done between Buena Vista Pumping Plant and Wheeler Ridge Pumping Plant, between Mileposts 255.7 and 279.2. It consisted of initially constructing a v-ditch along a 7-mile stretch west of Maricopa Road. The work included 190 ponds that were constructed along the remaining alignment, which ranged about 16.5 miles along the Aqueduct. Construction started on January 1, 1965, and was completed on March 11, 1966 (Specification No. 64-46). Ponding operations began on September 1, 1965, and ended on March 10, 1968. This was an area where predicted/anticipated subsidence was uncertain. As much as 9 feet of subsidence occurred at this location after ponding.

The fourth stretch of pre-construction ponding was done between Sunset Railroad and Maricopa Highway, between Mileposts 261.6 and 274.3. It consisted of constructing 74 ponds with infiltration wells along the v-ditch of the previous pre-consolidation contract (Specification No. 64-46). Construction started on April 5, 1966, and was completed on July 29, 1966 (Specification No. 66-12). Ponding

operations began on August 11, 1965, and ended on March 10, 1968. As much as 7.5 feet of subsidence occurred at this location after ponding.

The fifth stretch of pre-construction ponding was done between Wheeler Ridge Pumping Plant and Standard Oil Road, between Mileposts 279.2 and 283.9. This work consisted of constructing ponds with infiltration wells, along 4.7 miles of the Aqueduct. Unlike the other stretches, this area was not ponded with a ditch irrigation technique, but with a sprinkler system. Construction started on June 22, 1964, and was completed on April 2, 1965 (Specification No. 64-21). Ponding operations began on August 25, 1964, and ended on April 11, 1968. As much as 5.4 feet of subsidence occurred at this location after ponding.

The sixth stretch of pre-construction ponding was done between Standard Oil Road and Grapevine Creek, between Mileposts 283.9 and 288.7. It consisted of constructing several ponds along 4.8 miles of the Aqueduct. Construction started on November 8, 1963, and was completed on March 31, 1965 (Specification No. 63-32). Ponding operations began on August 25, 1964. As much as 1.7 feet of subsidence occurred at this location after ponding.

4.3 Construction

For the SLFD, there were five major contracts that Reclamation awarded for the construction of the canal. Table 4-4 has a list of the contracts. Table 4-5 has a breakdown of cross section properties. Those contracts were done from Milepost 67.1 to Milepost 174.8 (Pool 13 to Pool 21) and were constructed from 1963 through 1968.

Reach one was built under Reclamation Specification No. DC-5900 (Contract No. 14-06-200-260 A), *Earthwork, Concrete Lining, and Structures; San Luis Canal, Station 37+50 to Station 870+00*. It was awarded to Guy F. Atkinson Company on April 23, 1963. Construction began June 27, 1963, between Stations 110+00 and 190+00, and east of Jensen Road at approximately Station 51+00. The contract was completed on December 6, 1965. Reach one had an additional building contract to construct seven turnouts, a canal drain at the Los Banos Creek crossing, and pump sump covers in the canal invert under Specification No. DC 6333 (Contract No. 14-06-D-5760), *Turnouts, Canal Drain and Pump Sump Covers for San Luis Canal, Reach No. 1*. It was awarded to Syblon-Reid Company on October 14, 1965.

Reach two was built under Specification No. DC-5977 (Contract No. 14-060D-4917), *San Luis Canal, Reach 2, Earthwork, Concrete Lining, and Structures, Station 910+10.09 AH to Station 2053+15*. It was awarded to Morrison-Knudsen Company, Inc., and Utah Construction and Mining Company on August 16, 1963. The notice to proceed was received August 30, 1963. The contract was essentially completed by December 27, 1966.

Reach three was built under Specification No. DC-6148 (Contract No. 14-06-D-52(4)90), *Earthwork, Concrete Lining, and Structures, San Luis Canal, Reach 3, Station 2053+15 to Station 3907+00*. It was awarded to Peter Kiewit Sons' Company on November 9, 1964. The notice to proceed was issued December 5, 1964. On December 29, 1964, the contractor started pre-construction irrigation before excavating the canal prism. The contract was completed on December 29, 1967.

Table 4-4 Construction Contracts for the San Luis and San Joaquin Field Divisions

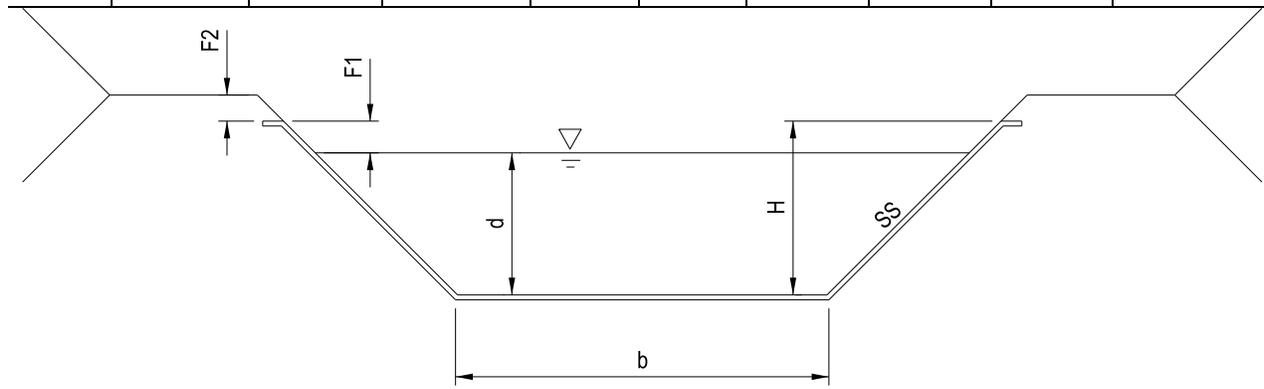
Division	Milepost		Miles	Pools	Start	End	Specification	Reach
	From	To						
San Luis	67.1	86.6	19.5	13	June 5, 1963	Dec. 6, 1965	DC-5900	1
	86.6	108.8	22.2	14-16	Aug. 30, 1963	Dec. 27, 1966	DC-5977	2
	108.8	143.6	34.8	16-19	Dec. 5, 1964	Dec. 29, 1967	DC-6148	3
	143.6	155.7	12.1	19-20	July 7, 1965	July 28, 1967	DC-6280	4
	155.7	174.8	19.1	20-21	Dec, 17, 1965	Feb, 21, 1968	DC-6344	5
San Joaquin	174.8	187.1	12.3	22-23	Feb, 17, 1966	Dec. 12, 1967	66-03	7, 8D, 9
	187.1	220.1	33	23-26	July 24, 1965	May 29, 1968	65-28	9, 10A, 11B, 12D
	220.1	239	18.9	26-29	April 13, 1967	June 12, 1969	67-06	12D, 12E, 13B
	239	251.8	12.8	29-31	Sept. 2, 1967	Sept. 23, 1969	67-37	13B, 14A
	251.8	280	28.2	31-36	Jan. 22, 1968	Aug. 31, 1970	67-69	14A, 14B, 15A, 16A
	280	295.8	15.8	36-40	May 10, 1968	April 15, 1971	68-07	16A, 17E

Table 4-5 Aqueduct Cross Section Properties for the San Luis and San Joaquin Field Divisions

Pool	Section Number	Milepost		d (ft.)	b (ft.)	F1 (ft.)	F2 (ft.)	H (ft.)	SS
		From	To						
13	1	70.90	86.47	32.8	110	4	3	36.8	2:1
14	2A	87.02	92.56	24.9	85	3	3	27.9	2:1
14-15	2B	92.57	97.52	24.9	85	4	3	28.9	2:1
15	2C	97.53	105.22	24.9	85	5	3	29.9	2:1
15	2D	105.25	108.50	24.9	85	8	3	32.9	2:1
16	3A	108.59	109.90	24.8	75	10	3	34.8	2:1
16	3A	109.96	113.36	24.8	75	12	3	36.8	2:1
16	3A	113.36	118.63	24.8	75	10	3	34.8	2:1
16-17	3B	118.67	128.73	24.8	75	8	3	32.8	2:1
17	3C	128.76	132.19	24.8	75	9	3	33.8	2:1
17	3D	132.19	132.95	24.8	75	10	3	34.8	2:1
18	3E	132.95	137.02	24.8	75	9	3	33.8	2:1
18	3F	137.02	143.29	24.8	75	6	3	30.8	2:1
19	4A	143.29	145.01	23.9	60	8	3	31.9	2:1
19	4B	145.05	152.70	23.9	60	10	3	33.9	2:1
19	5A	152.70	155.64	23.9	60	10	3	33.9	2:1
20	5B	155.70	164.69	23.9	50	8	3	31.9	2:1
21	5C	164.74	170.07	23.9	50	5	3	28.9	2:1
21	5D	170.11	172.40	23.9	50	3	3	26.9	2:1
22	8C/8D	172.44	182.39	26.9	32	2.5	2.5	29.4	2:1

California Aqueduct Subsidence Study

Pool	Section Number	Milepost		d (ft.)	b (ft.)	F1 (ft.)	F2 (ft.)	H (ft.)	SS
		From	To						
22	8D	182.39	184.82	26.9	32	5	2.5	31.9	2:1
23	9	184.84	194.94	25.3	32	3.8	2.5	29.1	2:1
23	9	194.94	197.05	25.3	32	4.4-4.8	2.5	29.7-30.1	2:1
24	10A	197.07	206.10	25.3	32	3.8	2.5	29.1	2:1
24	10A	206.10	207.94	25.3	32	6.3	2.5	31.6	2:1
25	10A	207.96	210.31	25.3	32	3.8	2.5	29.1	2:1
25	11B	210.31	217.79	24.0	32	4.8	2.5	28.8	2:1
26	12D	217.81	224.92	23.3	32	5	2.5	28.3	2:1
27	12D/12E	224.94	231.73	23.3	32	5	2.5	28.3	2:1
28	12E	231.75	236.67	23.3	32	5	2.5	28.3	2:1
28	12E	236.67	238.11	21.9	32	2.4	2.5	24.3	2:1
29	13B	238.13	244.54	20.8	32	3.5	2.5	24.3	2.5:1
30	13B	244.56	249.45	20.8	32	3.5	2.5	24.3	2.5:1
30	14A	249.46	250.99	22.9	32	2.5	2.5	25.4	2.5:1
31	14A	251.18	254.15	22.4	24	2.5	2.5	24.9	2:1
31	14A	254.15	256.14	22.4	24	5.5	2.5	27.9	2:1
32	14A	256.18	261.72	22.4	24	5.5	2.5	27.9	2:1
33	14B	261.77	267.36	22.1	24	7	2.5	29.1	2:1
34	14B	267.43	270.22	22.1	24	7	2.5	29.1	2:1
34	14B	270.22	271.27	22.1	24	8	2.5	30.1	2:1
35	14C/15A	271.33	278.13	21.4	24	8.7	2.5	30.1	2:1
36	15A/16A	278.43	280.36	21.4	24	6.5	2.5	27.9	2:1
37	16A	280.77	283.95	21.0	24	4.5	2.5	25.5	2:1
38	16A	284.01	285.69	21.0	24	4.5	2.5	25.5	2:1
38	16A	285.69	287.09	21.0	24	2.5	2.5	23.5	2:1
39	16A	287.14	290.21	21.0	24	2.5	2.5	23.5	2:1
40	16A/17E	290.23	293.45	21.0	24	2.5	2.5	23.5	2:1



Notes:

ft. = feet, F1 = lined freeboard, F2 = unlined freeboard, d = water depth, b = canal bottom width, H = height to top of lining, SS = side slope

Reach four was built under Specification No. DC-6280 (Contract No. 14-06-D-5636), *Earthwork, Concrete Lining, and Structures – San Luis Canal, Station 3907+00 to Station 4404+00*. It was awarded to Granite Construction Company and Gordon H. Ball Enterprises, a joint venture, on June 29, 1965. The notice to proceed was received by the contractor on July 7, 1965. The contract was completed on August 4, 1967.

Reach five was built under Specification No. DC-6344 (Contract No. 14-06-D-5771), *Earthwork, Concrete Lining, and Structures, San Luis Canal, Station 4404+00 to Station 5449+00*. It was awarded to Granite Construction Company and Gordon H. Ball Enterprises, a joint venture, on November 17, 1965. The notice to proceed was received by the contractor on December 17, 1965. Construction began on March 17, 1966, with the excavation of a 6-inch perforated asbestos-cement pipe drain, and a portion of Check 6 structure at Station 4559+00. The contract was accepted as complete on January 19, 1968.

For the SJFD, there were six major DWR contracts (Table 4-4 has contract details, Table 4-5 has section properties) that were awarded for the construction of the canal between Mileposts 174.8 and 295.8 (Pools 22 to 40), and two contracts that were awarded for the construction of turnouts between Kettleman City and 7th Standard Road, and between Tupman Road and the A.D. Edmondston Pumping Plant. Construction was from 1965 to 1971.

Reaches 7, 8D and 9 were built under Specification No. 66-03, *Canal – STA. (Station) 192+00 to STA. 845+30 Kettleman City to Avenal Gap – Mile 174.8 to Mile 187.1*. It was awarded to Gordon H. Ball Enterprises and Granite Construction Company. Construction began February 17, 1966, and was completed on December 12, 1967. In addition to the lined channel, this contract included the construction of three county road crossings, two oil-company road crossings, one operating road crossing, one siphon (Avenal Gap), five culverts, nine overchutes, and six turnouts.

Reaches 9, 10A, 11B and 12D were built under Specification No. 65-28, *Canal – STA. 845+30 to STA. 2580+90 Avenal Gap to 7th Standard Road – Mile 187.1 to Mile 220.1*. It was awarded to Gordon H. Ball Enterprises and Granite Construction Company. Construction began July 24, 1965, and was completed on May 29, 1968. In addition to the lined channel, this contract included the construction of four check structures, one canal drain (7th Standard Road), four county road bridges, three operational bridges, three culverts, 25 overchutes, and 11 irrigation turnouts.

Reaches 12D, 12E and 13B were built under Specification No. 67-06, *Canal – STA. 2580+90 to STA. 3575+00 Standard Road to Tupman Road - Mile 220.1 to Mile 239.0*. It was awarded to Peter Kiewit Sons' Company. Construction began April 13, 1967, and was completed on June 12, 1969. In addition to the lined channel, this contract included the construction of six bridges (two operation bridges and four county road overcrossings), two check structures, 15 overchutes, one siphon (Temblor Creek), two turnouts, and several pipe crossings.

Reaches 13B and 14A were built under Specification No. 67-37, *Canal – STA. 3575+00 to STA. 4250+00 Tupman Road to Buena Vista Pumping Plant Intake Channel - Mile 239.0 to Mile 251.8*. It was awarded to Western Contracting Corporation. Construction began September 2, 1967, and was completed September 23, 1969. In addition to the lined channel, this contract included the construction of six farm bridges, one operational bridge, two utility pipeline crossing bridges, two check structures, nine overchutes, and one intake structure for a turnout.

Reaches 14A, 14B, 15A and 16A were built under Specification No. 67-69, *Canal – STA. “N” 4250+00 to STA. “B” 1507+53 Buena Vista Pumping Plant to Wheeler Ridge Pumping Plant - Mile 251.8 to Mile 280.0*. It was awarded to Griffith Company. Construction began January 22, 1968, and was completed on August 31, 1970. In addition to the lined channel, this contract included the construction of two under drain systems, siphon undercrossings (three with upstream check structures, one road siphon undercrossing, and one combined road and railroad siphon undercrossing), one separate check structure, one spill basin (upstream of Buena Vista Pumping Plant), 11 bridges (four county road overcrossings, one farm road overcrossing, and six operational bridges), two sediment traps, eight overchutes, several culvert drainage undercrossings, and nine turnouts.

Reaches 16A and 17E were built under Specification No. 68-07, *Canal – STA. “W” 5+57 to STA. “T” 670+61 Wheeler Ridge Pumping Plant to Tehachapi Pumping Plant - Mile 280.8 to Mile 295.8*. It was awarded to Griffith Company. Construction began May 10, 1968, and was completed April 15, 1971. In addition to the lined channel, this contract included the construction of three siphons (two with upstream check structures and all with operating road bridges), one bridge to Wind Gap Power Plant, one county road bridge crossing and an oil-pipeline access bridge, one check structure, 14 overchutes, six pipe culverts, one sediment trap, and one canal drain (Pastoria Creek).

4.4 Post-Construction

In the SLFD, Pools 14 to 21 (Milepost 67.1 to Milepost 174.8) were constructed from 1963 to 1968 (Section 4.3), but initial filling and operation of the canal did not start until 1967. Table 4-6 has more details.

In the SJFD, Pools 22 to 40 (Milepost 174.8 to Milepost 295.8) were constructed from 1965 to 1971 (Section 4.3), but initial filling and operation of the canal did not start until 1968. Table 4-6 has more details.

Table 4-6 Post-Construction Initial Water Deliveries in the San Luis and San Joaquin Field Divisions

Division	Milepost		Miles	Pools	Reach	Completion Date	First Water In
	From	To					
San Luis	67.1	86.6	19.5	13	1	Dec. 6, 1965	April 1967
	86.6	108.8	22.2	14-16	2	Dec. 27, 1966	Oct. 1967
	108.8	143.6	34.8	16-19	3	Dec. 29, 1967	Dec. 1967
	143.6	155.7	12.1	19-20	4	July 28, 1967	Dec. 1967
	155.7	174.8	19.1	20-21	5	Feb. 21, 1968	Dec. 1967
San Joaquin	174.8	187.1	12.3	22-23	7, 8D, 9	Dec. 12, 1967	Jan. 13, 1968
	187.1	220.1	33	23-26	9, 10A, 11B, 12D	May 29, 1968	
	220.1	239	18.9	26-29	12D, 12E, 13B	June 12, 1969	Note ^a
	239	251.8	12.8	29-31	13B, 14A	Sept. 23, 1969	
	251.8	280	28.2	31-36	14A, 14B, 15A, 16A	Aug. 31, 1970	Sept. 17, 1970
	280	295.8	15.8	36-40	16A, 17E	April 15, 1971	

Note:

^a “Water was admitted into each reach between checks as soon as construction permitted.” (Bulletin 200, pg. 181)

4.4.1 Canal Raises

After the San Luis Canal was built and became operational in December 1967, it became apparent that the canal had subsided in certain areas. Initially, the benchmarks between the 75-mile subsidence-prone areas were frozen to prevent discrepancies caused by subsidence during the construction period. Once the benchmarks were unfrozen in December 1967, with the release of water downstream of Check 14 (Milepost 95.06), as much as 3 feet of subsidence was recorded in some areas, and a rate of up to 1 foot of subsidence per year was documented.

During the first year of operation from January 1968 to January 1969, subsidence continued at a rate of 1 foot per year. As a result, the canal was raised 2 feet between Mileposts 132.19 and 132.95 (Stations 3315+00 and 3355+90), 4,000 feet upstream of Check 17 (Contract No. 200C-752). That raise was done to compensate for the 4 feet, of the original 6 feet, that had subsided, and to prevent water from overtopping the canal. In 1970, Reclamation proposed a two-stage 7-foot lining raise between Mileposts 128.76 and 137.02 (Stations 3134+00 and 3570+00). In the first stage, 3 feet of lining was raised between Mileposts 128.76 and 132.19, and between Mileposts 132.95 and 137.02. Two feet were raised between Mileposts 132.19 and 132.95 (Contract No. DC-6859).

In addition to the canal repairs that were done in 1969 and 1970, several other repairs were done, from 1968 through 1973, to bridges, turnouts, check structures, and other structures, to compensate for subsidence. In 1973, several of the more notable raises were to the three bridges on San Mateo Avenue, Cerini Avenue, and Mt. Whitney Avenue, between Mileposts 130 and 135. The bridges were raised 4.5 feet to 5.5 feet to compensate for the continuous subsidence in this area. Another notable raise was done between Mileposts 152.76 and 158.46. A protective dike was raised 4 feet between Dorris Avenue and Gale Avenue in 1972, to protect the canal. Table 4-7 has a summary of subsidence work that was done.

In 1980, another detailed study on subsidence was done along the canal (Section 4.4.5). Subsequently, in 1982, the canal and parts of the road embankment were raised. Eight locations, totaling 27 miles of canal, were raised because of subsidence, and to accommodate higher operating levels. The canal was raised between Mileposts 87 and 172, and the amount of canal that was raised varied from pool to pool, as much as 56 inches in some critical areas. Table 4-8 has more details.

Two canal freeboard raises were completed in the SJFD. The first raise was done in 1989 under Specification No. 89-26, *Aqueduct Modification Mile 182.4 to Mile 184.8 and Mile 194.9 to Mile 197.0*. The lining was raised 30 inches between Mileposts 182.39 and 184.82 (Pool 22), and as much as 39 inches of canal were raised (to a set elevation of 312.75 feet) at Pool 23, between Mileposts 194.94 and 197.05. A total of 4.5 miles of canal were raised under that specification.

Another raise was done seven years later in 1996, under Specification No. 96-19, *Aqueduct Modification Mile 206.10 to Mile 207.94*. A 1.84-mile section of the canal, from Milepost 206.1 to Milepost 207.94, was raised 30 inches (Pool 24). Table 4-9 has more details.

Table 4-7 Initial Canal Lining and Structure Raises in the San Luis and San Joaquin Field Divisions from 1968 to 1973

Year	Milepost		Description of Raise	Amount of Raise (inches)
	From	To		
1968	89.68		Left side of abutment raised at Eagle Field Bridge.	13
1968	89.68		Right side of abutment raised at Eagle Field Bridge.	18
1969	89.68		Right and Left side of abutments raised at Eagle Field Bridge.	3
1969	117.47		Left side of abutment raised at Manning Avenue Bridge.	4.5
1969	117.47		Right side of abutment raised at Manning Avenue Bridge.	16
1969	117.47		Right side of abutment raised at Manning Avenue Bridge.	3
1969	132.19	132.95	Canal lining raised.	24
1970	93.67		Left pier raised at Nees Avenue Bridge.	3
1970	128.76	132.19	Canal lining raised.	36
1970	132.19	132.95	Canal lining raised.	24
1970	132.95	137.02	Canal lining raised.	36
1970	151.73		Installed invert and slope protection, by means of sheet piling and sacked concrete, at Lassen Avenue Bridge. The work was done because the east-west training dike was severely undercut.	
1971	89.68		Left abutment raised at Eagle Field Road Bridge.	3
1971	100.55		Left abutment raised at Shields Avenue Bridge.	3
1971	117.47		Right abutment raised at Manning Avenue Bridge.	3
1971	132.95		Gate sills at Check No. 17 raised.	36
1972	152.76	158.45	Canal protective dike and the east-west training dike raised between Dorris Avenue and Gale Avenue.	48
1973	108.50	143.23	Turnouts modified to accommodate settlement in Reach 3.	
1973	130.00	135.00	San Mateo Avenue, Cerini Avenue, and Mt. Whitney Avenue bridges raised.	54-66
	152.70	172.40	12 gated drain inlets installed at the lower end of reach 5 to prevent overtopping of the low canal embankment by flood runoff.	

Table 4-8 1982 Canal Liner Raises in the San Luis Field Division

Pool	Milepost		Station		Length (miles)	Description of Raise	Maximum Raise (inches)
	From	To	From	To			
14	87.02	91.11	910+10	1126+00	4.09	Canal lining raised.	56
15	99.40	103.19	1570+00	1770+00	3.79	Canal lining raised.	24
15	104.29	105.25	1828+00	1878+17	0.96	Canal lining raised.	40
17	124.65	130.02	2916+00	3200+00	5.37	Canal lining raised.	58
17-18	130.02	137.00	3200+00	3569+00	6.98	Road embankment raised.	36
18	137.00	138.67	3569+00	3660+00	1.67	Canal lining raised.	33
21	164.74	166.76	5041+28	5148+00	2.02	Canal lining raised.	36
21	170.09	172.19	5324+00	5435+00	2.10	Canal lining raised.	27

Table 4-9 1989 and 1996 Canal Raises in the San Joaquin Field Division

Year	Pool	Spec.	Milepost		Station		Length (miles)	Maximum Raise (inches)
			From	To	From	To		
1989	22	89-26	182.39	184.82	718+00	846+00	2.43	30
1989	23	89-26	194.94	197.05	1373+41.59	1485+00	2.11	39
1996	24	96-19	206.10	207.94	1962+84	2059+99.2	1.84	30

Note:

Spec. = specification number

4.4.2 Canal Repairs

Canal lining repairs along the San Luis Canal were done as early as 1996. Three lining repair contracts were awarded from 1996 to 2007. The repairs included small areas between Mileposts 55 and 164.9 (Pools 10 to 21). Table 4-10 has a summary of repairs that have been done over the years. This section of the report will only focus on lining repairs that have been done for Contract Nos. 96-30 and 07-20 (Milepost 56.4 to Milepost 164.9), specifically major lining repairs done at Mileposts 87.45, 89.56, 90.12 (Contract No. 07-20), 134.98 and 157.4 (Contract No. 96-30).

Table 4-10 Canal Repairs in the San Luis Field Division

Spec.	Pools	Description of Work
96-30	18 and 20	Canal Lining Repair - MP 134.98 and 157.4
98-06	10	Emergency Canal Repair - MP 55
07-20	10-21	Canal Lining Repair - MP 56.4-64.90

Notes:

MP = milepost, Spec. = specification number

For Contract No. 96-30, two Type I repairs were done. *Type I repairs* were defined as those conducted in areas where lining damage extended below the water surface. That type of repair consisted of removing damaged concrete panels above and below the water, filling voids with gravel, placing geo-textile fabric over the gravel, anchoring the bottom and sides of the fabric with angle battens, injecting concrete slurry into the fabric form, and placing compacted backfill material along the trench.

Type II repairs were defined as those conducted in areas where lining damage was only present above the water surface. That type of repair consisted of removing damaged concrete panels above water, filling voids with compacted backfill, placing new concrete, and cutting contracting joints and installing joint sealant. For Contract No. 07-20, 67 lining repairs were done along the canal; 32 of those repairs were Type I repairs, and 35 were Type II repairs. Table 4-11 has a summary of Type I and Type II lining repairs that were done at each pool under that contract.

Most of the repairs were done in Pools 14 and 15, which are two of the pools where parts of the canal were raised in 1982 to compensate for subsidence (Mileposts 87.02 to 91.11, Mileposts 99.4 to 103.19,

Table 4-11 Specification 07-20 Lining Repairs in the San Luis Canal

Pool	Type I Repair	Type II Repair	Total
10	1	0	1
13	2	1	3
14	12	15	27
15	7	14	21
16	2	0	2
17	0	1	1
18	4	2	6
19	1	1	2
20	2	0	2
21	1	1	2
Total	32	35	67

and Mileposts 104.29 to 105.25). Between Pools 14 and 15, 48 locations, which account for approximately 72 percent of the lining repairs, were done.

4.4.2.1 Milepost 134.98

Lining repairs at Milepost 134.98 R (Pool 18) were done under Specification No. 96-30 in 1996. That part of the canal was raised in 1970 (Milepost 128.76 to 137.02) and in 1982 (Milepost 130.02 to 137) because of subsidence. The Type I repairs consisted of removing 70 concrete panels (top three rows) and replacing them with fabric form liner. The damage was approximately 435 feet long by 45 feet wide. The damaged panels that were replaced covered 15,750 square feet. In addition, 315 feet of eroded embankment was backfilled and compacted.

4.4.2.2 Milepost 157.4

Lining repairs at Milepost 157.4 R (Pool 20) were done under Specification No. 96-30 in 1996 because of flood damage. The Type I repair consisted of removing 15 concrete panels (top two rows) and replacing them with fabric form liner. The damage was approximately 195 feet long by 25 feet wide. A total of 2,730 square feet of damaged panels was repaired, and 320 linear feet of eroded embankment were backfilled and compacted.

4.4.2.3 Milepost 87.45

Lining repairs at Milepost 87.45 R (Pool 14), under Specification No. 07-20, were Type I repairs that consisted of removing 54 (buckled) concrete panels and replacing them with a fabric form liner (nine columns by six rows). The area of repair was 7,811 square feet (107 feet long by 73 feet wide). The panels were severely buckled from the top of the liner to the invert of the canal. Work was done in March 2008 under Construction Order No. 46. That part of the canal was raised in 1982 to compensate for subsidence between Mileposts 87.02 and 91.11.

4.4.2.4 Milepost 89.56

Lining repairs at Milepost 89.56 R (Pool 14), under Specification No. 07-20, were Type I repairs that consisted of removing 24 (buckled) concrete panels and replacing them with fabric form liner. The area of

repair was 5,757 square feet. The extent of the damage in the area was from the top of the liner to the invert of the canal. Work at this location commenced in December 2007, under Construction Order No. 3. The site is within Milepost 87.02 to 91.11, which was raised in 1982 to compensate for subsidence.

4.4.2.5 Milepost 90.12

Lining repairs at Milepost 90.12 L (Pool 14), under Specification No. 07-20, were Type I repairs that consisted of removing 25 concrete panels and replacing them with fabric form liner (five columns by five rows). The area of repair was 5,480 square feet (78 feet long by 70.25 feet wide). This area had a five-panel wide 2-inch gap on the invert that was observed to be draining water into the embankment. Coincidentally, a 70-foot long (1-foot thick) berm was also placed at the location, on the land side toe of the embankment, because of seepage problems. The lining was repaired in January 2010 under Construction Order No. 75. The berm was installed in November 2009 under Construction Order No. 68. That part of the canal was raised in 1982 to compensate for subsidence between Mileposts 87.02 and 91.11.

Canal lining repairs along the SJFD were done as early as 1979. Three lining repair contracts were awarded from 1979 to 1998. The repairs included small areas between Mileposts 260.42 and 330.82 (Pools 32 to 48). Table 4-12 has a summary of repairs that have been done over the years. This section of the report focuses on lining repairs that were done under Specification Nos. 79-28 and 87-42 (Milepost 260.42 to Milepost 287.15), specifically major lining repairs done at Mileposts 273.48 (79-28) and 287.15 (87-42).

Table 4-12 Canal Lining Repairs in the San Joaquin Field Division

Specification	Pools	Description of Work
79-28	32 and 35	Lining Repair - MP 260.42, 260.45, and 273.48
87-42	39	Lining Repair - MP 287.15
98-01	40 and 48	Emergency Canal Lining and Forebay Liner Repairs MP 326.77-330.82 and Edmonston Power Plant

Notes:

MP = milepost

4.4.2.6 Milepost 273.48

Lining repairs at Milepost 273.48 R were done under Specification No. 79-28 in 1979. The repair consisted of removing 21 concrete panels (eight above water and 13 below water) and replacing them with precast concrete panels. A 1-foot thick bedding material was placed on the slope, behind the precast panels, to fill in the voids. The damage started at the top of the liner and continued to the invert of the canal (eight panels long by six panels wide). The damaged panels that were replaced covered an area of 3,024 square feet.

4.4.2.7 Milepost 287.15

Lining repairs at Milepost 287.15 R were done under Specification No. 87-42 in 1987. The repair consisted of removing the top 1.5 rows (seven panels wide) of concrete panels and replacing them with precast concrete panels. Bedding material was placed on the slope, behind the precast panels, to fill in the voids as necessary. The replaced concrete panels covered an area 87.5 feet long by 18 feet wide

(1,575 square feet). Approximately 1,200 square feet (an area 112.5 feet by 10.67 feet) of sacked concrete slope protection was placed near the top of the lining to protect the slope.

4.4.3 Leaks and Seeps

There have been several problem spots along the canal that have had leaks and seeps over the years. A few of the problem spots originated not long after the canal was constructed. A few of them have been temporarily repaired by grouting bentonite or a cement-bentonite mix into the embankment (a brief explanation is in Section 4.4.4). Other sites have had far worse issues where grouting only seemed to slow down the leak and not fix the problem. Most of the leaks and seeps have occurred between Pools 14 and 17 (Milepost 88 to Milepost 133), which are the pools where grouting through the embankment has been done over the years, and where extensive damage exists, particularly in Pool 17. Table 4-13 has a summary of leaks along the canal, and the different types of repairs that have been done. This section of the report will focus on repairs done at Mileposts 88.30, 88.96, 89.50, and 121.98 in the San Luis Canal.

Table 4-13 San Luis Division Leak and Seep Repairs

MP	Pool	Type of Repair									
		Berm	Station	L (ft.)	H (ft.)	Date	Sheetpile	Station	L (ft.)	D (ft.)	Date
88.30	14	x	973+00-980+20	720	15	Dec. 2008	x	972+00-981+98	998	60	Apr. 2010
88.96	14	x	1008+66.08-1014+21.08	555	10	May 2012	x	1008+87-1013+87	500	50	July 2012
89.50	14	x	1040+50-1043+00	250	6.5	July 2009	-	-	-	-	-
90.12	14	x	1075+70-1076+40	70	1	Nov. 2009	-	-	-	-	-
90.30	14	-	-	-	-	-	-	-	-	-	-
106.60 ^a	15	-	-	-	-	-	-	-	-	-	-
121.98	16	x	N/A	100	11.3	Oct. 2012	-	-	-	-	-
125.48 ^a	17	-	-	-	-	-	-	-	-	-	-
125.60 ^a	17	-	-	-	-	-	-	-	-	-	-
126.02 ^a	17	-	-	-	-	-	-	-	-	-	-
131.14	17	-	-	-	-	-	-	-	-	-	-
132.77 ^a	17	-	-	-	-	-	-	-	-	-	-

Notes:

D = depth of sheetpile wall, ft. = feet, H = height of berm, L = length of berm or sheetpile wall, MP = milepost, N/A = not applicable

^aDesign pending. Work to be done under future contract.

Grouting was done at Milepost 90.30 and Milepost 131.14 to repair the leaks.

4.4.3.1 Milepost 88.30

Based on documentation at the SLFD headquarters, seepage problems at Milepost 88.30 date back to mid-1970. Documents from SLFD files indicate that boils, seeps, and ponded water have been observed repeatedly over the years. Seepage problems have usually occurred on the left embankment toe road,

approximately between Mileposts 88.28 and 88.32. Over the years, the field division has pumped bentonite slurry into a group of 2-inch perforated PVC pipes, located on the water-side shoulder of the embankment crest road, to control the seepage. Generally, this procedure would slow down, or temporarily stop the seepage, but the problem would eventually recur.

In November 2008, following Reclamation's recommendation, an interim seepage berm (Specification No. 07-20) was built to control seepage through the embankment while a more permanent fix was done. That decision came after SLFD personnel started continuously monitoring and measuring flow from a cluster of boils that were located on the left toe of the embankment. In October 2008, the recorded cumulative flow from these boils was between 50 gallons per minute (gpm) and 60 gpm. A peak flow of 71 gpm was recorded during November 2008. The thought was, the interim seepage berm would protect the embankment from internal erosion and would decrease the risk of failure of the embankment. A 720-foot (Station 973+00 to Station 980+20, Milepost 88.21 to Milepost 88.35) seepage berm was built in December 2008. The berm was placed up the slope to a height of 15 feet and a thickness of 5 feet. The 5-foot thick berm consisted of a 1-foot layer of well-graded sand placed on the ground, a 3-foot layer of 3/8-inch gravel placed on top of the sand and covered with geofabric (Station 976+80 to Station 978+20, Milepost 88.28 to Milepost 88.31), and miscellaneous fill material placed over those layers.

The seepage berm did nothing to slow down the flow at Milepost 88.3. Soon after the berm was built, seepage flow rates in this area ranged from 42.5gpm to 55.3 gpm, which was similar to the rates recorded before the berm was built. In late 2009, a PZC-18 sheetpile wall was installed through the crest of the embankment to permanently cut off the seepage path and form an impervious barrier (Specification No. 09-07). A 998-foot long sheetpile wall was placed on the edge of the embankment road (near the water side) from Station 972+00 to Station 981+98 (Milepost 88.19 to Milepost 88.38), 60 feet deep into the embankment. The first seepage zone from Station 973+20 to Station 975+00 (Milepost 88.21 to Milepost 88.25) was completed in January 2010, and seepage was intersected at Station 973+25, Milepost 88.21. The second seepage zone from Station 977+00 to Station 978+20 (Milepost 88.28 to Milepost 88.31) was completed in March 2010, and seepage through the berm ceased as piles were installed at Station 979+03 (Milepost 88.32).

4.4.3.2 Milepost 88.96

In April 2009, a suspicious pond of standing water on the left side of the embankment toe road (near an existing v-ditch) was found at Milepost 88.96 (Station 1011+96.06). In May 2009, the SLFD installed a sandbag ring around the boil and placed a pipe in the ring to discharge any seepage from the Aqueduct into a nearby v-ditch. The flow was monitored over the course of 2.5 years and seepage increased from 2 gpm to 33 gpm from May 2009 to April 2012. In March 2012, cracks and lobate bulges were also observed on the crest of the road and on the toe of the embankment. Subsequently, another sandbag ring and outlet pipe was constructed in April 2012, and a geological investigation was done during March and April 2012.

Following Reclamation's recommendation, a 555-foot long seepage berm was constructed in May 2012 (Specification No. 09-21), between Stations 1008+66.08 and 1014+21.08, Mileposts 88.88 and 88.99, because of safety concerns and while a more permanent fix was done to address the problem. But before the stability berm was installed, the top 4 feet of the embankment crest and portions of the land side slope were removed along 300 feet between Stations 1009+60 and 1012+60, Mileposts 88.90 and 88.96, to increase the stability of the Aqueduct. After parts of the embankment were rebuilt to design elevations

and slopes, the stability berm was built. The berm consisted of placing a 2-foot layer of sand on the toe of the embankment and up the slope to an elevation of 330 feet, placing a 3-inch diameter perforated PVC pipe along the toe of the embankment and attaching it to four lateral pipes (spaced 100 feet apart to drain into the v-ditch) and to four 16-inch relief wells (40 feet deep), and placing native material over the sand (and pipes) to a total thickness of 8 feet. After the berm was built, the rest of the embankment was re-built to original design with the crest widened to 20 feet.

The stability berm did little to stop, or slow, the seepage. The flow increased to 41 gpm during the construction of the berm. As a result, a sheetpile cutoff wall (Specification No. 09-21), similar to the one installed at Milepost 88.3, was installed. A 500-foot long (Station 1008+87 to Station 1013+87, Milepost 88.89 to Milepost 88.98), 50-foot deep AZ 26-700 sheetpile wall was installed in July 2012. A total of 109 sheetpile pairs were pushed through a 4-foot deep trench that was excavated on the crest of the embankment. The head elevation of the wall was 336 feet and the tip elevation 286 feet. The seepage path was successfully intersected at Station 1012+04, Milepost 88.95 (sheetpile pair number 69 at elevation 311 feet) on July 21, 2012. Even though the sheetpile wall successfully cutoff any flow of water to the boil, the seepage pathway behind the piles remained. As a result, permeation grouting (behind the piles, on the land side of the wall) was done during October 2012 to close the seepage pathway.

4.4.3.3 Milepost 89.50

The left side of the embankment at Milepost 89.5 has had seepage issues since the early 1990s. Over the years, the problem was dealt with by pumping bentonite slurry into the embankment, but that procedure only temporarily stopped the seepage. In 2009, a boil was observed a third of the way up the embankment slope, 11 feet below the crest (to an approximate elevation of 325 feet) at Station 1042+00 (Milepost 89.52). The seepage was flowing at 12 gpm when Pool 14 was at (or near) maximum water surface elevation. Seepage was collected by a pipe and drained into an existing nearby agricultural v-ditch across the toe road. As part of Specification No. 07-20, *Canal Lining Repair, Milepost 56.40 to 164.90*, a change order was issued in July 2009 to build a drainage berm to reduce and control the seepage.

A 250-foot seepage berm was built between Stations 1040+50 and 1043+00, Mileposts 89.49 and 89.54, in July 2009. A 1-foot thick layer of sand was placed on the toe of the embankment and up the slope to an elevation of 330 feet, like other similar designs, to collect and control seepage and prevent the migration of fine soil particles from the embankment. A 2-foot layer of gravel (10 feet by 10 feet) was placed over the boil and a 3-inch diameter PVC pipe was placed in the gravel to drain any seepage into the concrete v-ditch across the toe road. Filter fabric was placed on all sides of the gravel and native material was placed over the drainage system to a thickness of 6.5 feet (up to an elevation of 332 feet). The berm has successfully controlled seepage from the Aqueduct.

4.4.3.4 Milepost 121.98

The SLFD started monitoring seepage through the left side of the embankment at Milepost 121.98 in August 2011. At first, seepage rates remained constant (less than 1 gpm). Then in January 2012, seepage rates started steadily increasing. They reached 2 gpm by May 2012, 2.5 gpm by August 2012, and 6 gpm by September, 2012. As a temporary measure, Civil Maintenance staff from SLFD placed a sandbag ring around the boil on the land side toe of the embankment. To fix the problem, as part of Specification No. 09-21, *Temporary Rock Barriers – 2010, 2011 and 2012 Middle River, Old River and Grant Line Canal*, a construction order was issued in October 2012 to construct a seepage berm.

In October 2012, a 100-foot long berm was constructed on the landside toe of the embankment. A 2-foot layer of sand was placed on the toe of the embankment and up the slope, to an elevation of 327.7 feet, to prevent the migration of fine soil particles. A 2-foot layer of gravel was placed on top of the sand layer and wrapped with geofabric. A filter trench, 10 feet deep by 2 feet wide, was constructed 4 feet away from the toe of the embankment to intersect any seepage. To complete the berm, a 2-foot layer of native material was placed on top of the sand and gravel layer, to an elevation of 329.7 feet (11.33 feet total height). Like the work done at Milepost 89.5, this berm has been successful at controlling seepage from the Aqueduct.

4.4.4 Grout Tubes

The SLFD has dealt with boils, seeps, and/or ponded water (on the land side toe) by using grout tubes to control seepage problems. Generally, 2-inch PVC pipes (approximately 40 feet deep) have been installed on the water-side shoulder of the embankment crest road, and a bentonite-water slurry has been pumped into the pipes to stop seepage. That procedure has been done repeatedly over the years, but in general, the injection of bentonite into the embankment has slowed seepage, and not permanently solved seepage problems along the canal. Table 4-14 has a summary of grout tubes that have been installed along the canal. GPS readings were taken in 2013 during a Civil Condition Assessment Program field investigation.

Based on Table 4-14, Pools 14 to 17 have experienced many seepage problems over the years. Particularly Pool 17, which is consistent with subsidence issues that part of the canal had pre- and post-construction. Several miles of that pool had linings raised in 1969, 1970, and 1982. Similarly, Pools 14 and 15 had several miles of lining raised in 1982 to compensate for subsidence. Section 4.4.1 has more details.

Table 4-14 Grout Tubes in the San Luis Field Division

Side of Embankment	Start GPS (North)	Start GPS (West)	End GPS (North)	End GPS (West)	Number of Tubes	Milepost ^a	Pool
Primary	36.91	120.81	36.91	120.81	7	88.58	14
Primary	36.90	120.81	36.90	120.81	21	88.60	14
Primary	36.90	120.81	36.90	120.81	31	88.68	14
Primary	36.90	120.81	36.90	120.80	30	88.76	14
Primary	36.90	120.80	36.90	120.80	18	88.83	14
Primary	36.90	120.80	36.90	120.80	21	88.90	14
Primary	36.90	120.79	36.89	120.79	14	89.62	14
Primary	36.89	120.79	36.89	120.79	6	90.10	14
Primary	36.73	120.60	36.73	120.60	3	106.60	15
Primary	36.73	120.60	36.73	120.60	6	106.73	15
Primary	36.71	120.58	36.71	120.58	9	108.26	15
Primary	36.56	120.46	36.56	120.46	16	121.10	16
Primary	36.56	120.44	36.56	120.44	12	121.98	16
Primary	36.55	120.43	36.55	120.43	13	122.78	17
Primary	36.54	120.39	36.54	120.39	33	125.35	17

Side of Embankment	Start GPS (North)	Start GPS (West)	End GPS (North)	End GPS (West)	Number of Tubes	Milepost ^a	Pool
Secondary	36.53	120.38	36.53	120.38	18	126.02	17
Primary	36.52	120.37	36.52	120.37	6	126.87	17
Primary	36.51	120.36	36.51	120.36	23	128.09	17
Secondary	36.48	120.32	36.48	120.32	8	130.92	17
Primary	36.48	120.32	36.48	120.32	15	131.02	17
Secondary	36.48	120.32	36.48	120.32	12	131.14	17
Secondary	36.47	120.31	36.47	120.31	12	132.19	17

^aEstimate of milepost.

4.4.5 1980–1981 Study

A study was done, during 1980 and 1981, along the San Luis Canal to estimate future subsidence, and subsequently, determine how much to raise the lining on the canal. The study was based on several data sources, which included:

1. Seven surveys (Table 4-15) that were done from November 1967 through April 1978. The surveys included numerous benchmarks that were installed along the alignment of the canal after it was constructed.
2. Subsidence information that was obtained from six deep-compaction recorders that were installed in the vicinity of the canal.
3. A 1980-1981 top of lining survey that was conducted by the DWR Fresno Construction Office.
4. Previously prepared drawings that plotted out subsidence during construction.
5. Drawings showing piezometric levels, canal deliveries, and the effects of droughts on piezometric levels.
6. Brief visual field observations at major structures that were used to evaluate leveling data.
7. Two sets of published USGS topographic maps (1919-1932 and 1955-1956) that were used to approximate the beginning of subsidence.

The estimates for the study were made using semilogarithmic plots by projecting subsidence rate trends until 1985-1990, which is the time frame in which most of the future subsidence was expected to occur according to the calculated half-life of 29 selected benchmarks. The subsidence estimates were made under the assumption that post-construction subsidence, before and after November 1977-April 1978, would be residual.

Estimates were completed for 76 benchmarks, located along the canal, and for the period after November-December 1967. Appendix A has a description of the methodology that was used to estimate future subsidence. Appendix B (which is included in the supplemental information packet for this report) is a plot showing the top of lining as it was designed, at its condition in 1980-81, and as the lining was estimated to subside in the future (1985-1990).

Table 4-15 Surveys Used in the 1980–1981 Study

Number	Date	Survey By
1	Nov.- Dec.1967	U.S. Coast and Geodetic Survey (Baseline)
2	Dec. 1968-Jan. 1969	U.S. Coast and Geodetic Survey
3	Oct.-Nov. 1969	U.S. Coast and Geodetic Survey
4	Nov. 1970-Jan. 1971	National Geodetic Survey
5	Nov. 1971-Feb. 1972	National Geodetic Survey
6	Jan.-March 1975	National Geodetic Survey
7	Nov. 1977- April 1978	California Department of Water Resources

Note:

Several surveys were ignored, and others regrouped because of inaccuracies created by poor data. The final four surveys used were November-December 1967, October-November 1969, November 1971 to February 1972, and November 1977 to April 1978.

Based on Appendix B, and after the study was completed, a few sections of the canal were raised in 1982 to address the areas where subsidence was expected to affect the required freeboard (specifically in Pools 14, 15, 17, 18, and 21). Section 4.4.1 has more details on the raises that were done in 1982.

Chapter 5. Operational Criteria

5.1 Aqueduct Design Criteria

The Aqueduct was designed as an unreinforced, concrete lined, trapezoidal canal with a side slope of 2:1 (horizontal:vertical). The design bed slope of the canal profile was 0.00004 and the conveyance capacity varied from reach to reach. The Aqueduct area was sized for a friction factor of 0.016 and a minimum lined freeboard (vertical distance between the top of the maximum water surface elevation to the top of lining) of 3 feet. The lined freeboard was reduced to 2.5 feet from Pool 22 on downstream. To increase the embankment's safety factor against failure because of overtopping, unlined freeboard (vertical distance between the top of the lining to the top of the embankment) was also provided. The total freeboard was the summation of lined and unlined freeboard.

5.1.1 Initial Intent of Freeboard

The total freeboard provided along the Aqueduct is given in Tables 4-1 and 4-2. In the original design, the minimum amount of lined and unlined freeboard was fixed either by the canal capacity or by normal wave conditions. In many locations, the provided freeboard was more than minimum freeboard (Tables 4-1 and 4-2). The additional freeboard was provided to account for:

- Future subsidence.
- Surges during sudden gate closures or openings.
- Waves during extreme winds.
- Possible water surface undulations where the design velocity is expected to be greater than 0.8 times the critical velocity (additional freeboard was provided depending upon possible water surface undulations).
- Higher velocities, such as on the outside of a horizontal curve or through transition structures.
- Emergency shutdown conditions.

5.1.2 Original Manning's N-Value

The major hydraulic loss came from the canal friction loss and Manning's friction factor was used to characterize it. Minor hydraulic losses were contributed by the bridges, siphons, bends, and check structures along the Aqueduct. In the original design, all conduits and conveyance structures were designed with a contingency factor of 5 percent to assure deliveries could be made even if an adverse change in Manning's n-value (increase in n) occurred because of aquatic growth, sediment deposits, or other features. The 5 percent contingency was used in the n-value for friction loss computations and in other coefficients used to compute minor losses (California Department of Water Resources 1965).

5.1.3 Check Structures

A number of gated check structures are provided along the Aqueduct alignment. Aqueduct pools are separated by check structures and pumping plants throughout the Aqueduct. The gates at check structures regulate the Aqueduct's water flow rate and water surface elevation. Primary functions of the check structures are:

- To maintain minimum water surface elevation to prevent damage to the lining because of high groundwater.

- To maintain minimum water surface elevation for turnouts.
- To be able to isolate certain reaches of the Aqueduct should emergency repairs be required.
- To contain the flow in transit in specific reaches when a sudden shutdown of the system occurs.
- To provide storage for local delivery.
- To facilitate the optimal operation of pumps and gates.

Locations of the check structures were selected so that the primary functions were met in an economic way. The height of the lined freeboard was coupled with the spacing of the check structures. The water surface elevation in the lined canal must be kept within specific limits to prevent failures in the lining and embankments because of hydrostatic back pressure caused by water trapped behind the panels. The check structures were spaced such that a maximum of 75 percent of the lined freeboard would be encroached by ponding of water and a minimum freeboard of 0.3 foot is maintained at all check structures.

5.1.4 Design Capacity

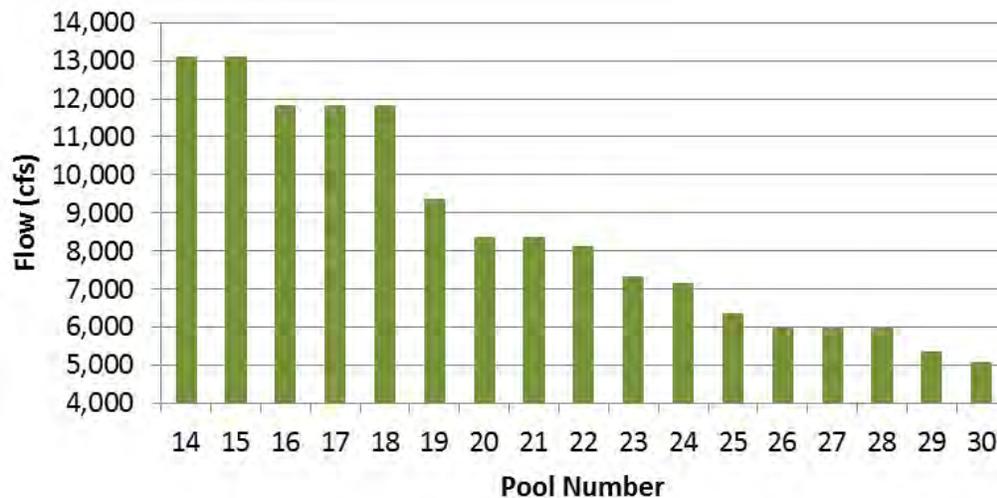
As originally designed, a number of turnouts are used to divert water from the San Luis Canal and the Aqueduct as water is transported south. As a result, the Aqueduct size has been reduced in the downstream direction. Table 5-1 gives the design flow capacity and approximate size of the Aqueduct section for pools downstream of Dos Amigos Pumping Plant (DAPP). The change in designed flow capacity of the Aqueduct is shown in Figure 5-1. The Aqueduct was sized to convey 8,350 cfs to the end of the San Luis Canal. From the end of the San Luis Canal, the flow of Aqueduct gradually tapers from 8,100 cfs to 5,050 cfs at Buena Vista Pumping Plant (BVPP). The Aqueduct was sized such that the maximum monthly water demand could be delivered through a reach by maintaining a constant head and flow.

Table 5-1 Maximum Conveyance Capacity of California Aqueduct between Dos Amigos Pumping Plant and Buena Vista Pumping Plant

Aqueduct Mileage (Milepost)	Pool	Aqueduct Conveyance Capacity (cfs)	Bottom Width/Depth of Canal (feet)
Up to 108.5	14, 15	13,100	85/29.9 to 32.6
108.5 to 133.0	16, 17, 18	11,800	75/30.8 to 33.8
133.0 to 145.05	19	9,350	60/31.9 to 33.9
145.05 to 170.11	20, 21	8,350	50/28.9 to 31.7
170.11 to 184.63	22	8,100	32/31.9
184.63 to 194.94	23, 24	7,300	32/29.1
194.94 to 207.96	25	7,150	32/28.3
207.96 to 236.67	26, 27, 29	5,950	32/24.3 to 28.3
236.67 to 250.99	30	5,050	24/24.3 to 25.5

Note:

cfs = cubic feet per second

Figure 5-1 Variation of Aqueduct Pool Design Capacity

5.2 Water Deliveries

The scheduling of Aqueduct deliveries is planned on an annual, monthly, and daily basis as shown in Table 5-2. The State Water Contractors submit monthly schedules for the coming year in December. DWR evaluates the requests and determines the annual water allocation based on water availability, system physical constraints, and environmental requirements. The annual water allocation process typically starts in December and ends in May. Because of uncertainty in weather and water consumption rate, the monthly water delivery amounts are revised throughout the year. The amounts of monthly deliveries are controlled by multiple factors including water year type, operations of the upstream reservoirs, and environmental commitments. The monthly delivery demands must be revised by the 25th day of the prior month. The revisions to the requests for the daily deliveries are made 48 hours in advance and the deliveries are adjusted according to system capacity and real-time conditions.

Table 5-2 Ordering and Scheduling Criteria

Operation Time Frame	Criteria
Annually (DWR, 1967, page 46)	<ul style="list-style-type: none"> Contractors submit monthly schedule for coming year in December. Contractor is obliged to take 90 percent of the request. DWR reviews the request with regard to water availability. DWR determines the final delivery amounts.
Monthly	<ul style="list-style-type: none"> Schedules may be revised until 25th day of the month. Deliveries may vary 10 percent from scheduled amount.
Daily (DWR, 1967, Page 47)	<ul style="list-style-type: none"> Request for daily delivery made 48 hours in advance. Delivery schedules adjusted according to capability of system to meet demand.

Note:

DWR = California Department of Water Resources

The State Water Project Operations Control Office and the field divisions have established normal and absolute elevations for all Aqueduct pools. Those two limits are referred as normal (minimum and maximum) and absolute (minimum and maximum) elevation. Normal pool elevation criteria (maximum and minimum elevations) was established to provide operators with a guideline for operating Aqueduct pools within a range that provides design flow with a safe margin for turnout submergence on the lower end, and adequate lined freeboard at the top end. Normal pool elevation generally allows for a 2-foot operating range, 1 foot above and 1 foot below design depth of the pool as defined in the *Aqueduct Conveyance Facilities Data Handbook*, which was issued by the State Water Project in 2009.

The absolute elevation criterion (maximum and minimum elevation) was set to provide protection against extreme ranges where water surface elevation would exceed a physical limit or cause loss of turnout delivery.

Absolute maximum elevation is limited by:

- Top of lining.
- Top of closed gate(s).
- Spillway elevation and capacity.
- Obstructions (over-crossings).
- Low turnout overflow.
- Plant gallery drains.

Absolute minimum elevation is limited by:

- Turnout capacity.
- Pumping plant minimum operational forebay elevation.

5.3 Current Operational Restrictions

The Aqueduct storage volume between the normal minimum and normal maximum operating water surface elevation is used as a temporary reservoir, storing water during low-cost high-pumping period (nighttime) and drafting water for downstream delivery during high-cost low-pumping period (daytime). The pool storage volume helps to reduce the overall pumping cost by optimizing the operations of pumps and check structure gates.

For each pool, two drawdown criteria are specified, during the first hour of operation, and for a period of 24 hours of operation. The operation of turnouts and intermediate pumping plants are coordinated such that the drawdown for each pool is within the specified limits. The operating elevations of the pools and drawdown limits have been revised through standing operating orders (Standing Operating Order 600.22, Appendix C). Current restrictions in the operating water surface elevations at each check structure are summarized in Table 5-3.

Table 5-3 Maximum, Minimum, and Drawdown Limits (Standing Operating Orders 600.22, Appendix C)

Check Number	Absolute maximum (feet)	Absolute minimum (feet)	Normal maximum (feet)	Normal minimum (feet)	Drawdown limits
CK 14	333.0 (SLWD TO Lat #1 MP 89.67L Overflows)	328.0	331.0	329.0	2 feet/24 hours 0.5 foot/first hour
CK 15	330.4 (WWD T.O. MP 105.22, Lat #1 Pad Overflows)	326.0	329.0	327.0	2 feet/24 hours 0.5 foot/first hour
CK 16	326.1 (Top of Gate)	322.1	325.4	323.4	2 feet/24 hours 0.5 foot/first hour
CK 17	324.5	320.5	323.5	321.5	2 feet/24 hours 0.5 foot/first hour
CK 18	321.8 (Top of Gate)	319.0 (Pleasant Valley Pumping Plant)	322.5	320.5	2 feet/24 hours 0.5 foot/first hour
CK 19	319.0	315.5	318.8	316.8	2 feet/24 hours 0.5 foot/first hour
CK 20	317.8	313.4	316.9	314.9	2 feet/24 hours 0.5 foot/first hour
CK 21	314.5 (Top of Gate)	311.0	313.9	311.3	2 feet/24 hours 0.5 foot/first hour
CK 22	312.2 (Lined Freeboard Encroachment)	310.5 (To Prevent Las Perillas Pump Trip at 309.5)	311.8	310.8	1.5 feet/24 hours 0.5 feet/first hour
CK 23	309.6 (Low Overchute)	307.1	309.2	307.6	1.5 feet/24 hours 0.5 foot/first hour
CK 24	306.6 (Low Overchute)	304.9	306.4	305.1	1.5 feet/24 hours 0.5 foot/first hour
CK 25	306.0 (Lined Freeboard encroachment)	304.9	306.4	305.1	1.5 feet/24 hours 0.5 foot/first hour
CK 26	304.2 (Lined Freeboard encroachment)	302.0	303.7	302.2	1.5 feet/24 hours 0.5 foot/first hour
CK 27	302.4 (Lined Freeboard encroachment)	300.3	302.2	300.7	1.5 feet/24 hours 0.5 foot/first hour
CK 28	299.0 (Low Overchute)	297.7	298.8	297.9	1.5 feet/24 hours 0.5 foot/first hour
CK 29	298.0 (Pipeline Bridge)	296.6	297.8	296.8	1.5 feet/24 hours 0.5 foot/first hour
CK 30	296.6 (0.3 feet below spill basin crest)	294.6	296.3		1.5 feet/24 hours 0.5 foot/first hour

Notes:

Lat. = lateral , MP = milepost, SLWD = San Luis Water District , TO = turnout, WWD = Westlands Water District

5.4 Historical Turnout Flow Capacity and Delivery

Within each Aqueduct pool, there are numerous turnouts that deliver water to the federal and State Water Contractors. The number of turnouts has increased over the years, as shown in Table 5-4, which lists the number of turnouts for each pool for the years 2000, 2005, 2010, and 2015.

For this study, an analysis was conducted on turnout deliveries to identify if there have been changes in turnout flows (quantity or distribution) that have affected the flow capacity over time.

Historical turnout delivery data was analyzed to see if there were any noticeable trends. The flowrate for each turnout was recorded monthly for Pools 13 to 40 from 1990 to 2015.

Because there are currently 245 turnouts in Pools 13 through 40, this analysis combined the individual deliveries for each turnout into an overall delivery total per pool. In order to identify delivery trends, the overall total delivery per pool was graphed from 1990 through 2015. Figure 5-2 displays the cumulative turnout deliveries between 1990 and 2015. The cumulative per pool delivery represents the amount of water delivered for all of the turnouts that are in a particular pool. The plot of cumulative delivery per pool did not show any increasing trends with time (that may correspond with the increase in turnouts). The increases and decreases seem to come from the varying allocations throughout the years.

From 1990 to 2000, the turnout flowrates were summed into one turnout per pool. The data prior to 2000 were not reliable for establishing a turnout count for each year. While evaluating the data on a 5-year interval, (Figure 5-2) it was noted that there was an average increase of one turnout per pool from 2000 to 2005, from 2005 to 2010, and from 2010 to 2015. By evaluating the cumulative flowrate in each pool and the amount of turnouts per year, no significant trends were observed.

Figure 5-2 Total Flowrate from Turnouts for Pools 13 to 40 (Cumulative Flowrate, per Pool Over Time)

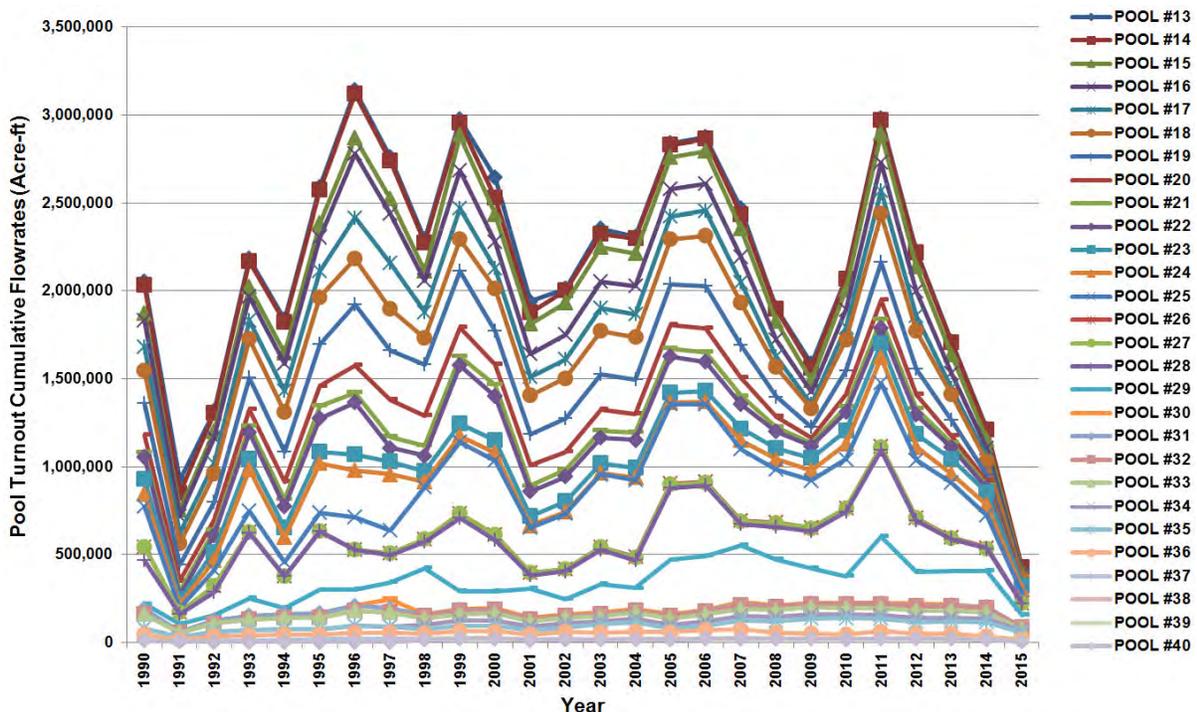


Table 5-4 Turnout Count per Pool for 2000, 2005, 2010, and 2015

Pool Number	2000	2005	2010	2015
13	20	24	24	25
14	15	16	16	16
15	23	23	23	23
16	25	26	26	26
17	15	16	16	16
18	19	20	22	23
19	12	12	13	13
20	13	13	13	14
21	9	9	9	9
22	9	9	9	9
23	4	4	5	5
24	3	3	5	5
25	6	7	7	7
26	1	1	1	1
27	1	1	1	1
28	3	4	4	7
29	5	7	7	8
30	1	1	1	1
31	2	2	2	2
32	2	2	2	2
33	2	2	2	2
34	1	2	5	6
35	2	6	10	12
36	2	2	3	3
37	1	1	1	1
38	2	2	3	3
39	1	1	1	1
40	4	4	4	4
Total (13-40)	203	220	235	245

Chapter 6. Subsidence Data and Analysis

6.1 Precise Survey Data

For the purposes of this chapter, the following sign convention will be used for subsidence data: negative numeric values represent a decrease in ground surface elevation; positive numbers represent an increase in ground surface elevation; values without a sign convention shall be absolute magnitudes defined within the context of discussion.

6.1.1 San Luis Field Division Survey Data

Numerous survey benchmarks, based on the National Geodetic Vertical Datum (NGVD) of 1929, were established along the Aqueduct (and its associated structures) after it was constructed. Benchmark leveling has been used since then as one of the main tools for providing elevation data for subsidence studies. The first comprehensive leveling of these benchmarks was conducted in October 1967. For the San Luis Canal, 940 benchmarks have been monitored at 1-year to 7-year intervals. Those benchmarks are located on structures, top of the lining, canal embankments, and near right-of-way boundaries. The benchmarks were surveyed using precise surveying techniques in 1967 (baseline), 1968, 1969, 1970, 1972, 1975, 1977, 1981, 1983, 1986, 1989, 1993, 2000, 2006, 2009, 2013, and using GPS in 2015. In 1975, 1994, and 2009, the survey data were adjusted in order to bring the observed data sets in better agreement with the NGVD of 1929. Appendix D has a brief explanation on how and why data were adjusted for the SLFD and the SJFD. In addition to surveying adjustments, some data points were corrected over the years because their locations were disturbed during modifications to the canal liner and other structures.

For this study, two types of data sets were extracted from the 940 benchmarks that were surveyed along the San Luis Canal:

1. A data set of 90 benchmarks, located on structures and top of the lining, was selected based on historic regional data benchmarks that were used in past subsidence studies. The yearly data of these benchmarks, identified by their milepost location, were plotted, milepost versus total subsidence in a given year, to show the progression of subsidence from 1967 to 2015. That change was calculated by subtracting a recorded elevation for a particular benchmark, at each year, from the baseline value recorded in 1967. The process was repeated for all of the benchmarks and the difference in elevation was plotted out on the y-axis, for a given milepost (x-axis). The leveling data from these benchmarks are summarized in Section 6.3 and are profiled in the subsidence profile graphs on Plates 1 through 9. Plate 1 is a plot of the SLFD. Plates 2 through 9 are plots of Aqueduct Pools 14 through 21.
2. A data set of 136 benchmarks, located on the top of the lining, was also extracted from the 940 benchmarks that were surveyed along the San Luis Canal in 2015. Their elevations were plotted on the operational profile graphs on Plates 1 through 9. They show the total subsidence that has occurred on the lining up to 2015.

6.1.2 San Joaquin Field Division Survey Data

For the SJFD, 1,009 benchmarks have been monitored at 3-year to 7-year intervals. Those benchmarks are located on structures, top of the lining, canal embankments, and near right-of-way boundaries. The benchmarks were surveyed using precise surveying techniques in 1967 (baseline), 1969, 1975, 1978,

1981, 1986, 1993, 2000, 2006, and 2013. In 1975, 1994, 2006, and 2013, adjustments were done in order to bring the observed data sets in better agreement with the constraints of the NGVD of 1929. In addition to surveying adjustments, some data points were corrected because their locations were disturbed during modifications to the canal liner, and other structures, over the years.

Two types of data sets were extracted from the 1,009 points for the SJFD:

1. A data set of 173 benchmarks located on structures and top of the lining, was selected based on historic regional data benchmarks that were previously used in subsidence studies. The yearly data of those benchmarks, identified by their milepost location, were plotted, milepost versus total subsidence in a given year, to show the progression of subsidence from 1967 to 2013. This change was calculated by subtracting a recorded elevation for a particular benchmark, at each year, from the baseline value recorded in 1967. This process was repeated for all of the benchmarks and the difference in elevation was plotted out on the y-axis, for a given milepost (x-axis). The leveling data from these benchmarks are summarized in Section 6.3 and as profiles on subsidence profile graphs in Plates 10 through 27. Plate 10 is a plot of the SJFD. Plates 11 through 27 are plots of Aqueduct Pools 22 through 38.
2. A data set of 249 benchmarks located on the top of the lining, was also extracted from the 1,009 benchmarks that were surveyed in the SJFD during 2013. Their elevations were plotted on the operational profile graphs on Plates 10 through 27. They show the total subsidence that has occurred on the lining up until 2013.

6.1.3 1981-1986 Survey Data Ground Surface Elevation Increase

Survey data along the SJFD, Milepost 177.40 to Milepost 287.09 (Pools 22 to 38), indicate an increase of the ground surface elevation between 1981 and 1986. This increase is as much as +0.6 feet across most of SJFD, even in the areas where there was only minimal or no signs of subsidence in the survey data collected since Aqueduct construction. In the areas of SJFD where subsidence is significant (Pool 23 through 25) this increase is reduced to almost zero. That pattern does not follow the concept of subsidence rebound, at least in terms of the recorded subsidence that has occurred since Aqueduct construction.

There are a few possible explanations for this increase in ground surface elevations.

1. Three major earthquakes struck this area in the 1980s, New Idria (1982), Coalinga (1983), and Kettleman Hills (1985). Their moment magnitudes were 5.4, 6.5, and 6.1, respectively. These earthquakes struck on a series of west-dipping, blind thrust faults. Each earthquake was accompanied by a strong aftershock sequence and uplift of a Quaternary age anticline atop the fault. It is possible that the survey control point located in Kettleman City also experienced uplift and created the increase in the 1986 survey data. Appendix D includes a brief discussion of the survey adjustments. It notes that the biggest change in adjustments was between the 1975 adjustment and the 1994 adjustment. The two sets of data (1981 and 1986) that show the increase of ground surface elevation are from two different survey data adjustments (1975 and 1994, respectively). Appendix D notes that the change in adjustment took into consideration the elevation change of the hill-tie.
2. In 1986, California experienced a wet year according to the Sacramento Valley Water Year Index, after having a dry year in 1985. It also experienced a rise in groundwater levels near the canal. After analyzing data between 1980 and 1985, records show that there was an average of 50 feet in groundwater rise in the observation wells adjacent to the canal. Uplift of the ground surface could have occurred during this transition from dry to wet year, or during the rise in groundwater levels, if enough water was present at depth to cause a rebound effect. But, this

may account for only a very minor contribution. Swanson, 1998, (Land Subsidence Case Studies Report) reported on land subsidence in the San Joaquin Valley, updated to 1995. His Figure 2 for Well 16/15-34N4, near Cantua Creek shows that an increase in groundwater level of roughly 300 feet in 20 years show a rebound of less than 0.5 feet.

It is uncertain whether the earthquakes in 1982, 1983 and 1985, the wet year in 1986, the rise in groundwater levels, or other phenomena, contributed to the increase seen in the data from 1981 to 1986. To be able to correct the data, an assumption would have to be made based on a definitive cause of the error. The rate of subsidence is a main focus of this study. A correction to the data would also invalidate the subsidence rates before and after this time period; as a result, the data were not corrected in an attempt to smooth it in with other data. DWR's Division of Engineering (DOE) recommends not using the data from 1981 to 1986 as part of any subsidence rate calculation. Recent 2013 survey data are not affected by the data in 1986. Data from 1986 and data from 2013 have different adjustments that were done by precise surveys and are independent of each other.

Figure 6-1 Subsidence and Operational Profiles - San Luis Field Division - Plate 1

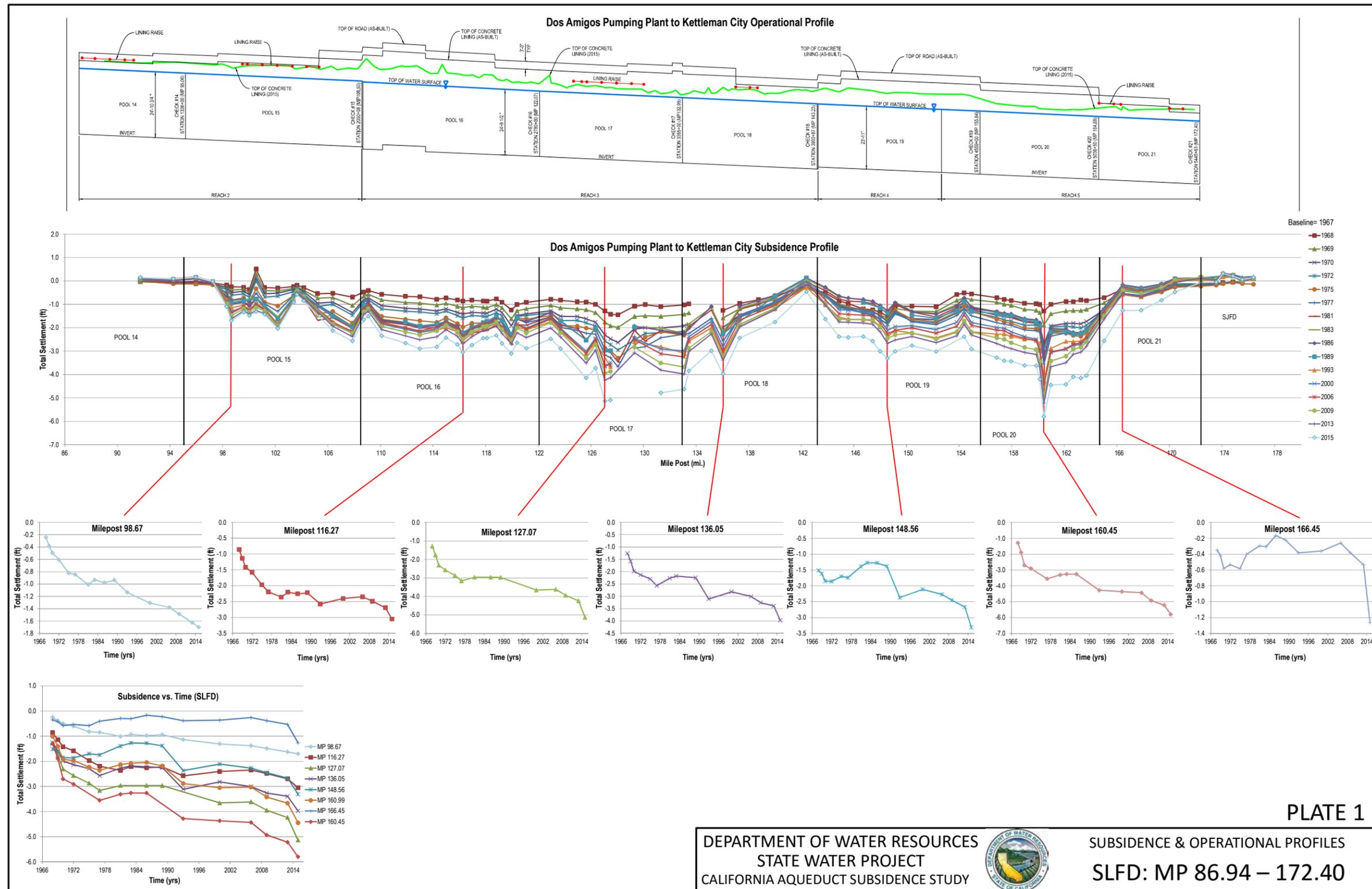
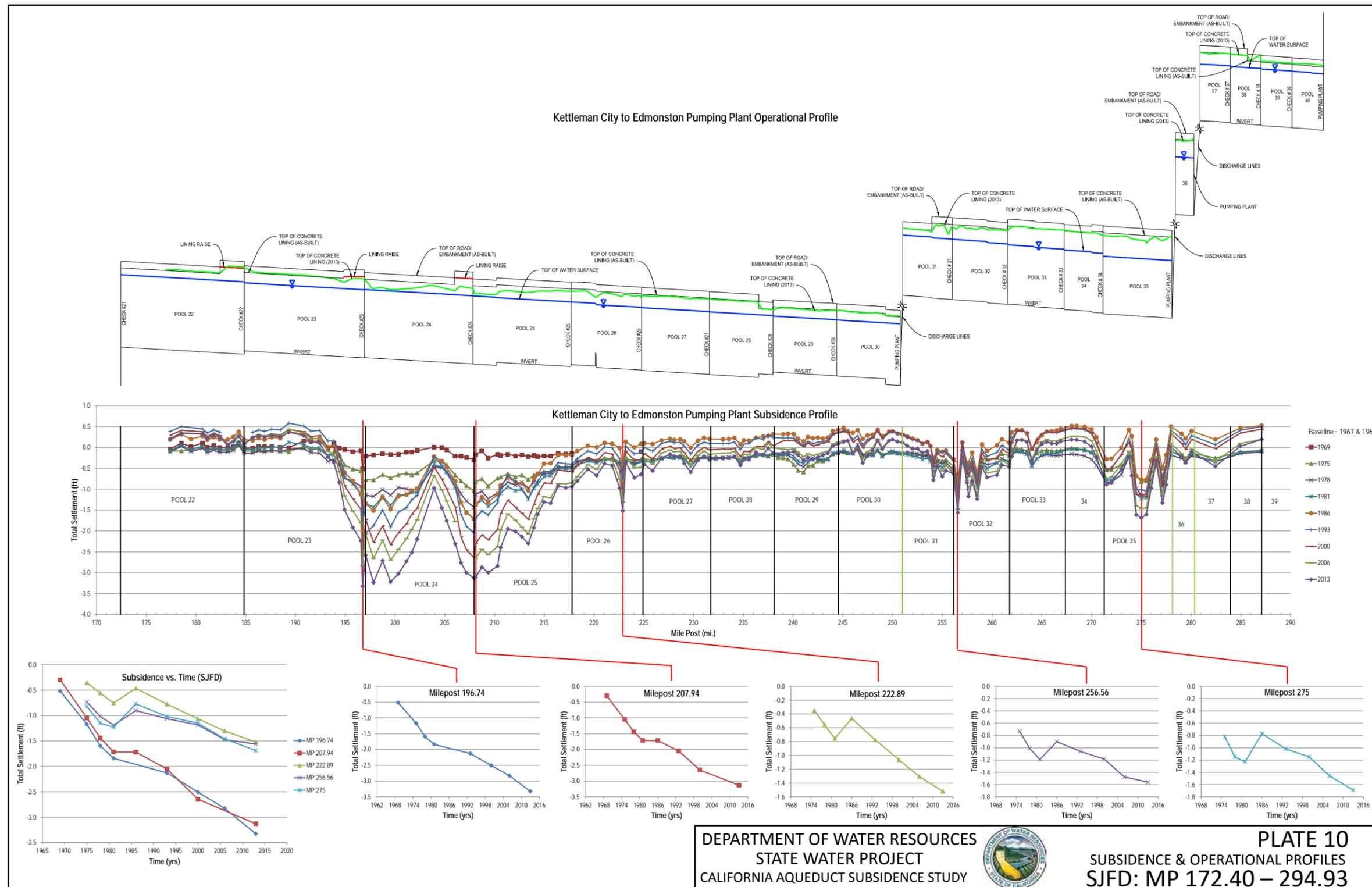


Figure 6-2 Subsidence and Operational Profiles - San Joaquin Field Division - Plate 10



6.2 Surveying Process (Past and Present)

For the *California Aqueduct Subsidence Study*, the DOE's Geodetic Branch met with the DWR Operations and Maintenance Division's Precise Surveys Unit and reviewed how the unit established and maintained a vertical control network for standard monitoring surveys of the Aqueduct and associated structures. The vertical control for the monitoring work was established by the National Geodetic Survey (NGS) around 1967 and continued until 1975. It is based on precise leveling measurements on a series of rock outcrops located on the western and southern edges of the Valley. The datum initially employed was the NGVD of 1929, the same datum used in the Aqueduct design and construction. Adjustments have been made over the years in an attempt to keep project control on datum by compensating for subsidence and crustal uplift.

The Aqueduct monitor points consist of a series of survey marks set on structures and top edges of the liner along the canal. Side surveying of rock outcrops in the hills west of the canal alignment were established as stable non-moving reference monuments to provide accurate measurements of subsidence of the Valley floor. In 1975, analysis of the monitor measurements indicated that the rock outcrop monuments in the hills might be rising ever so slightly because of crustal uplift, thereby giving a false reading that exaggerated the amount of subsidence. Since that time, the Precise Surveys Unit has made multiple adjustments based on detailed analysis of its leveling work in an attempt to maintain a stable, constrained adjustment on the network. Around 2000, it stopped making the ties to the rock outcrop monuments to economize.

Early monitoring was performed using three-wire differential leveling methods. As technology improved, they were replaced with more accurate digital bar-code differential leveling, while maintaining the use of the NGVD of 1929 as project datum. Currently, based on trials and quality control checks, the Precise Surveys Unit is using GPS equipment and techniques for efficiency and cost-effectiveness to monitor the Aqueduct. It is constraining the networks to some accepted "held points" that its analysis indicates are the most stable for this work. Additional follow-up leveling indicates that the GPS and the unit's measurement practices are producing repeatable results within the tolerance necessary to accurately quantify subsidence values. Over time, the practice will be much more cost effective and, if continually tied to the National Spatial Reference System through GPS post-processing, and the NGS Continually Operating Reference Stations network, the repeatability and accuracy of the monitoring should improve. The Precise Surveys Unit's series of adjustments have all been minor in magnitude to maintain the level of accuracy required for this project.

6.3 Subsidence Data/Methods and Measurements/Monitoring

Profiles of subsidence along the SLFD portion of the Aqueduct (Los Banos to the Kettleman City area), Milepost 91.75 to 176.39 (Pools 14 to 21 and part of Pool 22), from 1967 to 2015, are shown on Plate 1. The profiles were created using survey data taken from 25 benchmarks located on hard structures such as bridges, turnout structures, irrigation crossings, siphons, recorder stations, and check structures, and 65 benchmarks located on the top of the liner. Ninety benchmarks are included in the profile.

The survey data show this section of the Aqueduct, the San Luis Canal, has subsided the most over the years. According to the profiles, Pools 17 and 20 have experienced the most subsidence, with as much as -5.14 feet of subsidence in Pool 17 and -5.8 feet of subsidence in Pool 20. Pool 18 has also experienced a significant amount of subsidence with the most being -4.63 feet near Milepost 136. Pool 14 rebounded a

few inches in 2015, while Pools 15, 16, 19, and 21 subsided as much as -3.31 feet. Table 6-1 lists the maximum and minimum values of total subsidence in the SLFD by pool. It also includes the number of points, and their locations, that were used for this study.

Analyzing subsidence versus time in areas of maximum liner subsidence in seven pools, Mileposts 98.67, 116.27, 127.07, 136.05, 148.56, 160.45, 160.99, and 166.45, show that subsidence is correlated to allocation (Figure 6-3). Between 1977 and 1989, allocations were constant and subsidence was fairly minor. When allocations dropped in 1990, subsidence increased, and when allocations picked up again, between 1995 and 2006, subsidence decreased. Other similarities are: (1) sporadic rebounds, and (2) the fastest rate of subsidence occurring between 2013 and 2015 (Section 6.3.1). Figure 6-3 provides a graphical representation of subsidence over time at those particular locations with the Central Valley Project water allocations.

The 2015 survey, when comparing benchmarks located on the canal liner with benchmarks located on hard structures, shows that benchmarks located on hard structures tend to show more subsidence. For example, in Pool 15, benchmark 103.40-R, at a turnout, subsided -0.92 foot (from 1967 to 2015), compared to benchmark C1073 (Milepost 103.66), on the top of the liner, which subsided -0.58 foot. Pool 20, at Milepost 160.45, benchmark E1097, at a turnout, subsided -5.8 feet (from 1967 to 2015), compared to top of liner benchmarks D1097 and F1097, located upstream and downstream of the turnout, at Milepost 160.14 and 160.99, respectively, which subsided -4.2 feet and -4.45 feet.

Profiles of subsidence along the SJFD portion of the Aqueduct south of Kettleman City, Milepost 177.40 to 287.09 (Pools 22 to 38), from 1967 to 2013 are shown on Plate 10. The profiles were created using surveying data taken from six benchmarks located on hard structures, and 167 benchmarks located on the top of the liner. This profile includes 173 benchmarks. (Section 6.1 has a description of dataset one).

This particular section of the Aqueduct, which passes through six areas of hydrocompaction (Section 4.2.3 has more details), has not seen as much subsidence as the San Luis Canal, except in Pools 23, 24, and 25, where the canal has subsided as much as -3.33 feet. Pools 26, 32, and 35 have also subsided more than -1 foot, with as much as -1.69 feet of subsidence near Milepost 275. Pools 22, 27 through 31, 33 through 34, and 36 through 38 have experienced very minimal subsidence with the most being -0.79 foot. Table 6-2 lists the maximum and minimum values of total subsidence in the SJFD by pool. It also includes the number of points, and their locations, that were used for this study.

Looking at subsidence versus time at Mileposts 196.74, 207.94, 222.89, 256.56, and 275 (areas of maximum liner subsidence in five different pools), there appears to be no correlation between allocation and the amount of subsidence. Instead, the data suggests constant rates of subsidence over time (Figure 6-4).

Unlike the San Luis Canal, where the data sets had more benchmarks on hard structures and the settlement differences between subsidence of the canal liner versus hard structures could be compared, the San Joaquin dataset has few benchmarks on hard structures, which were too distant from benchmarks on the liner, to make a comparison. Plate 28 has total subsidence plots of the liner and hard structures from the most current surveys.