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
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Division of Engineering

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## 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.

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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 stake holders 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

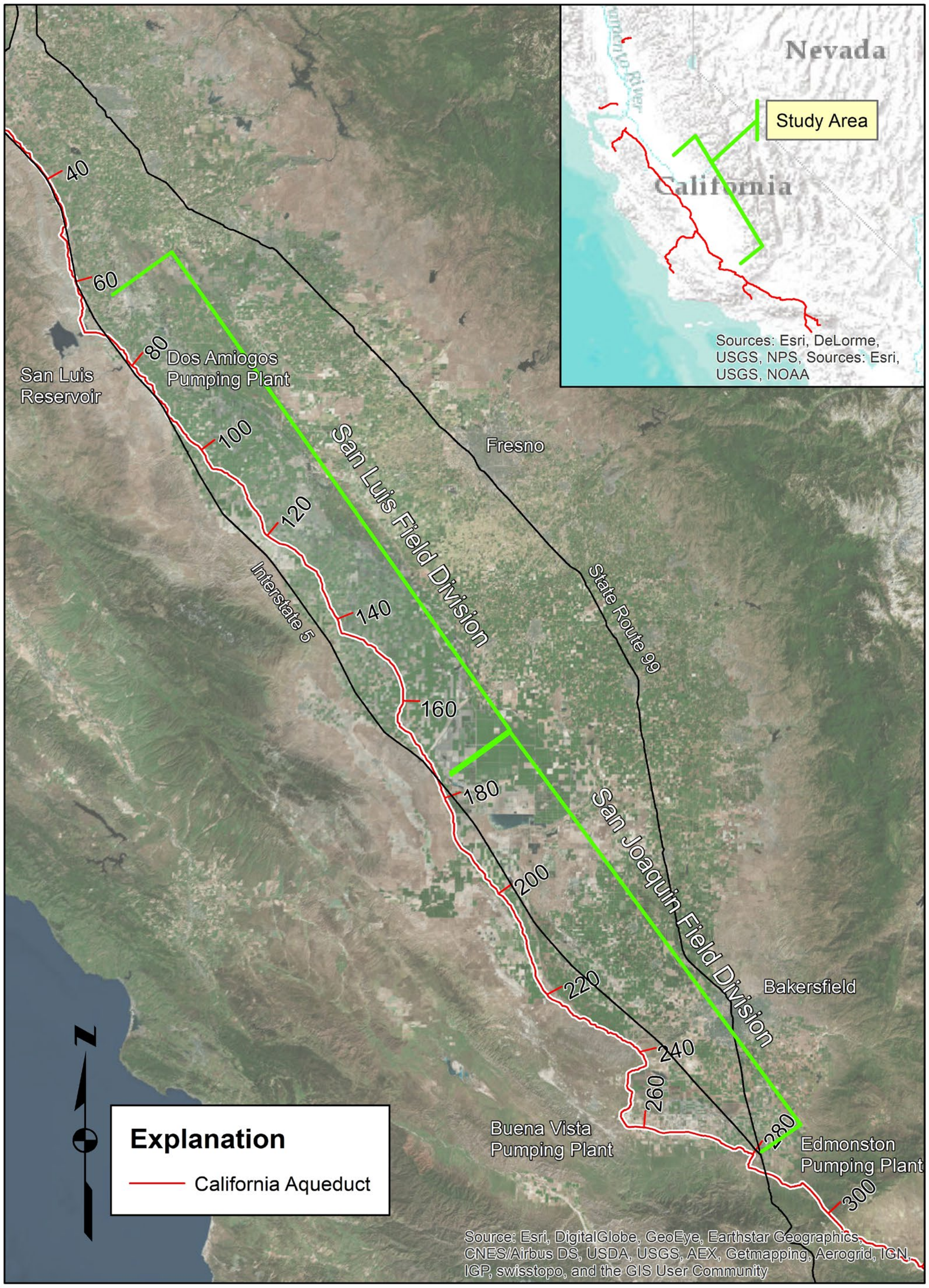
The Valley covers about 10,000 square miles and represents the southern part of the Central Valley, also known as the Great Valley of California. The Valley is bounded by the Sacramento-San Joaquin Delta (Delta) and Sacramento Valley on the north, the Sierra Nevada on the east, the Tehachapi Mountains 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 Klausing 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 in1995 (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.

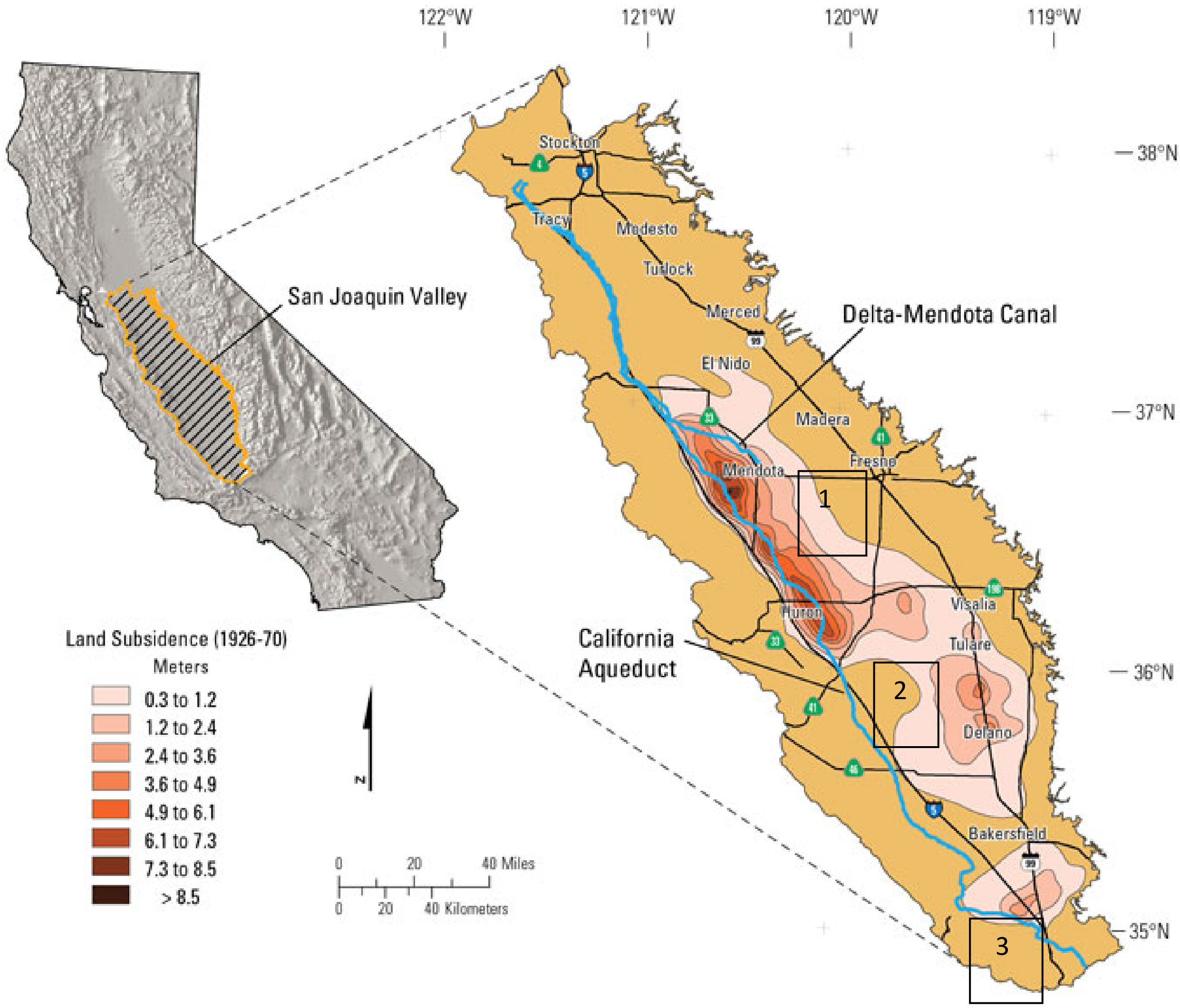
The previous studies documented that aquifer-system compaction brought on by groundwater overdraft causes the largest magnitude and areal extent of land subsidence in the Valley (Poland and others 1975, Ireland and others 1984, Farrar and Bertoldi 1988, Bertoldi and others 1991, Galloway and Riley 1999).

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.

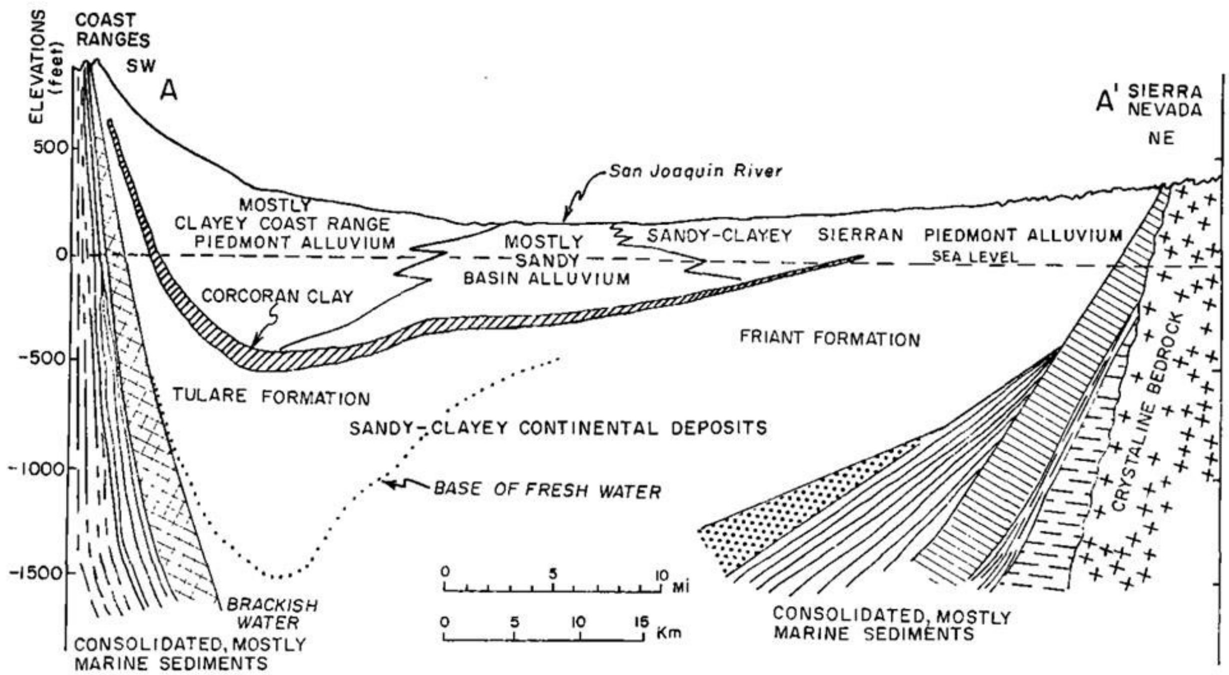
Numerous lenses of fine-grained sediments are distributed throughout the Valley and generally constitute more than 50 percent of the total thickness of the valley fill. These lenses are highly compressible and account for nearly all aquifer-system compaction and subsequent land subsidence. In general, 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 happed 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 1foot 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
[a]Figured for maximum water surface elevation.
[b]Invert grade raise due to subsidence.
[c]1969 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:
[a]Figured for maximum water surface elevation.
[b]Canal lining raised in 1989.
[c]Canal lining raised in 1996.

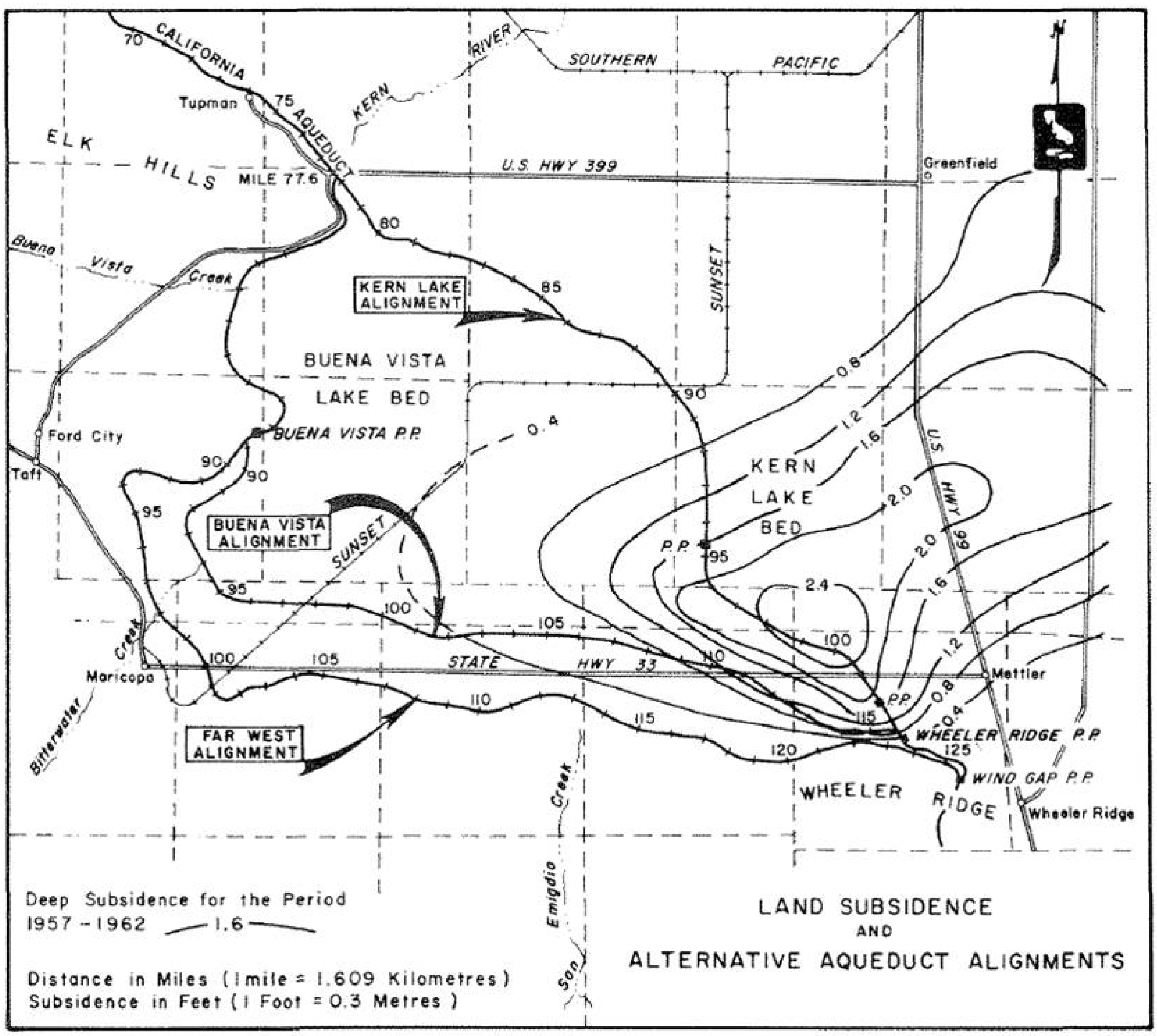
in the area where a 60-inch diameter culvert crossed under the road. There were some lining cracks, but everything appeared stable after the lining was patched and the sunken area was refilled. Another area of settlement was observed just downstream of the Buena Vista Pumping Plant at a 72-inch diameter 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 to18 months. A series of gravel packed infiltration wells 75-, 100-, and 125-feet 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
[a]Maximum 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 Milepost174.8 (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.

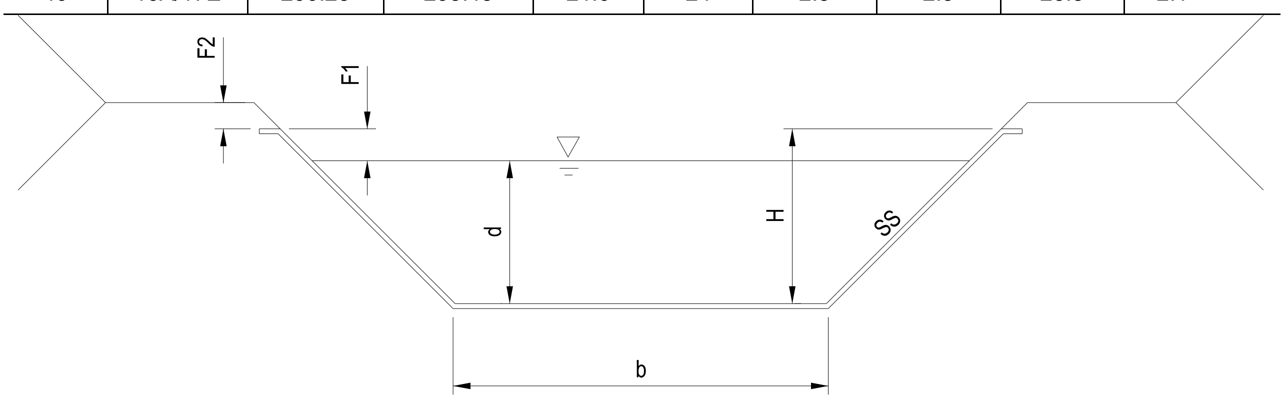
### Table 4-4 Construction Contracts for the San Luis and San Joaquin Field Divisions

| 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 |

| 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 Mileposts104.29 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 OrderNo.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<br>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
[a]Design pending. Work to be done under future contract.
Grouting was done at Milepost 90.30 and Milepost131.14 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 where 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 Station1042+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 in1982 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 |

[a]Estimate 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.

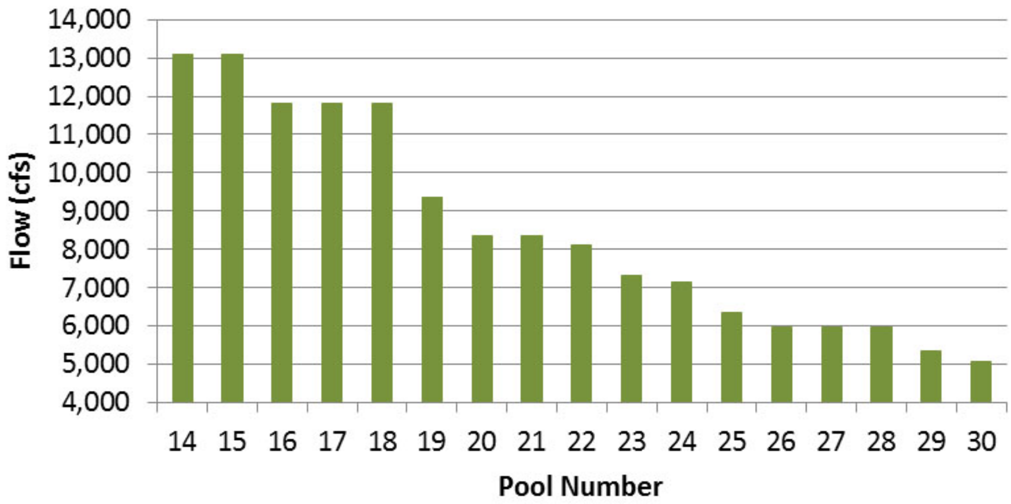
**Table 5-1 Maximum Conveyance Capacity of California Aqueduct between Dos Amigos Pumping Plant and Buena Vista Pumping Plant**

| 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) | • Contractors submit monthly schedule for coming year in December. Contractor is obliged to take 90 percent of the request.<br>• DWR reviews the request with regard to water availability.<br>• DWR determines the final delivery amounts. |
| Monthly | • Schedules may be revised until 25th day of the month.<br>• Deliveries may vary 10 percent from scheduled amount. |
| Daily (DWR, 1967, Page 47) | • Request for daily delivery made 48 hours in advance.<br>• 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<br>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<br>0.5 foot/first hour |
| CK 16 | 326.1<br>(Top of Gate) | 322.1 | 325.4 | 323.4 | 2 feet/24 hours<br>0.5 foot/first hour |
| CK 17 | 324.5 | 320.5 | 323.5 | 321.5 | 2 feet/24 hours<br>0.5 foot/first hour |
| CK 18 | 321.8<br>(Top of Gate) | 319.0<br>(Pleasant Valley Pumping Plant) | 322.5 | 320.5 | 2 feet/24 hours<br>0.5 foot/first hour |
| CK 19 | 319.0 | 315.5 | 318.8 | 316.8 | 2 feet/24 hours<br>0.5 foot/first hour |
| CK 20 | 317.8 | 313.4 | 316.9 | 314.9 | 2 feet/24 hours<br>0.5 foot/first hour |
| CK 21 | 314.5<br>(Top of Gate) | 311.0 | 313.9 | 311.3 | 2 feet/24 hours<br>0.5 foot/first hour |
| CK 22 | 312.2<br>(Lined Freeboard Encroachment) | 310.5 (To Prevent Las Perillas Pump Trip at 309.5) | 311.8 | 310.8 | 1.5 feet/24 hours<br>0.5 feet/first hour |
| CK 23 | 309.6<br>(Low Overchute) | 307.1 | 309.2 | 307.6 | 1.5 feet/24 hours<br>0.5 foot/first hour |
| CK 24 | 306.6<br>(Low Overchute) | 304.9 | 306.4 | 305.1 | 1.5 feet/24 hours<br>0.5 foot/first hour |
| CK 25 | 306.0 (Lined Freeboard encroachment) | 304.9 | 306.4 | 305.1 | 1.5 feet/24 hours<br>0.5 foot/first hour |
| CK 26 | 304.2 (Lined Freeboard encroachment) | 302.0 | 303.7 | 302.2 | 1.5 feet/24 hours<br>0.5 foot/first hour |
| CK 27 | 302.4 (Lined Freeboard encroachment) | 300.3 | 302.2 | 300.7 | 1.5 feet/24 hours<br>0.5 foot/first hour |
| CK 28 | 299.0<br>(Low Overchute) | 297.7 | 298.8 | 297.9 | 1.5 feet/24 hours<br>0.5 foot/first hour |
| CK 29 | 298.0<br>(Pipeline Bridge) | 296.6 | 297.8 | 296.8 | 1.5 feet/24 hours<br>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<br>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)**

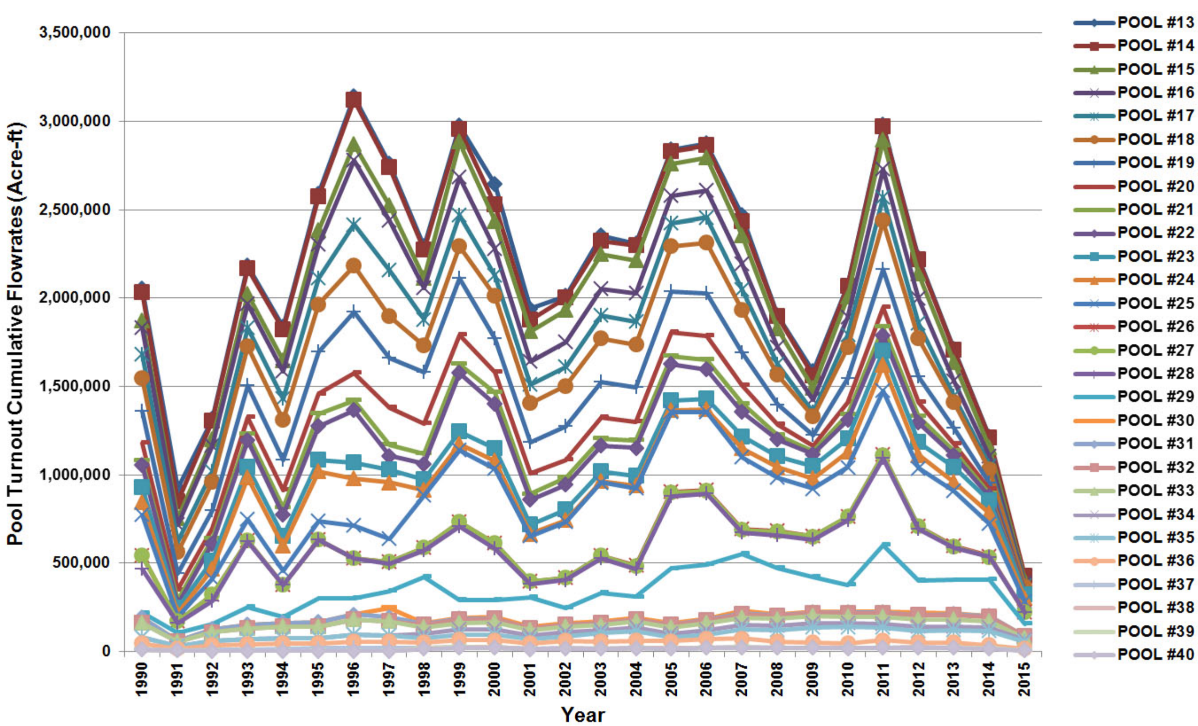


**Table 5-4 Turnout Count per Pool for 2000, 2005, 2010, and 2015**

| 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 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 to1986. 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**

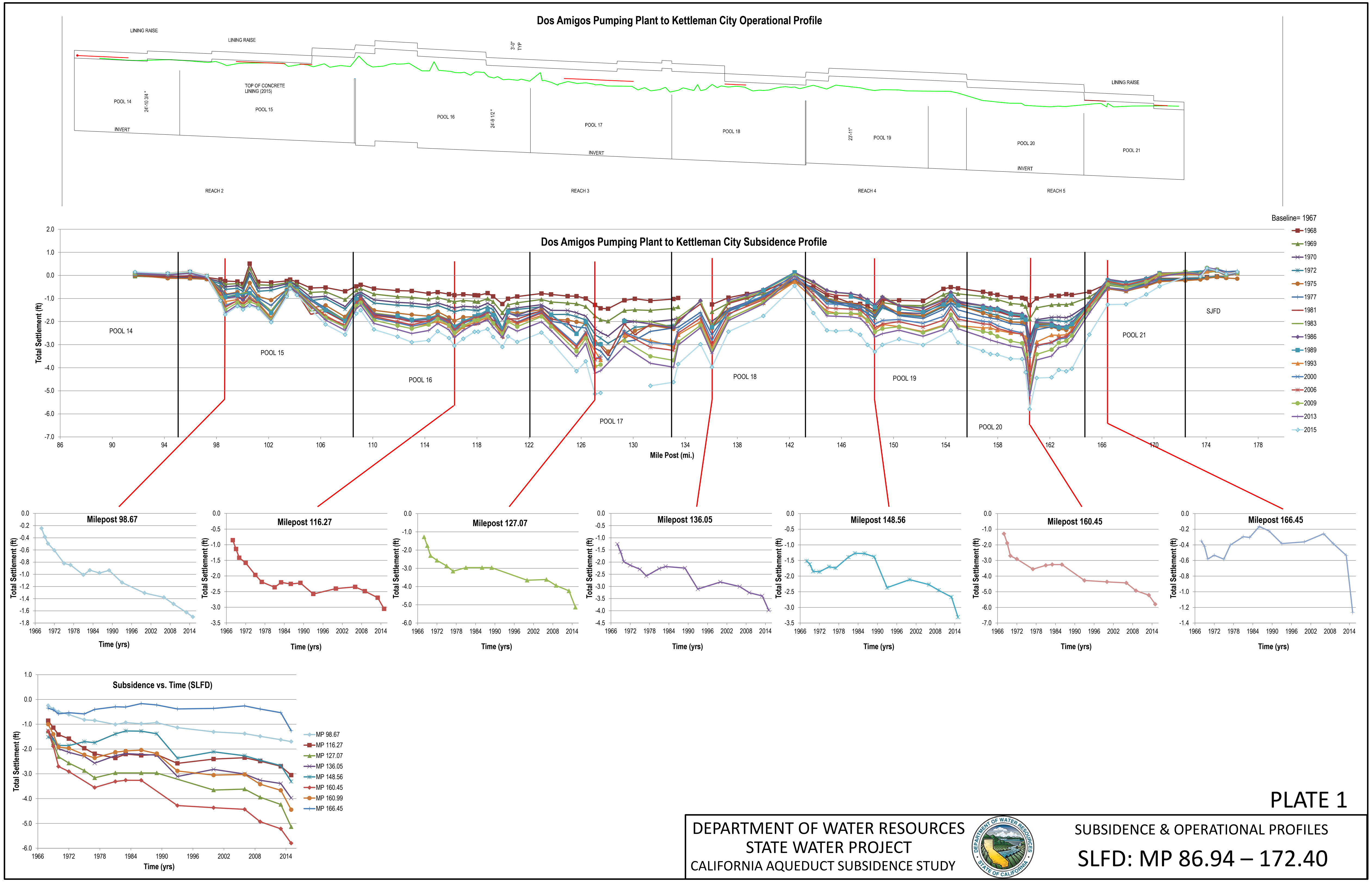
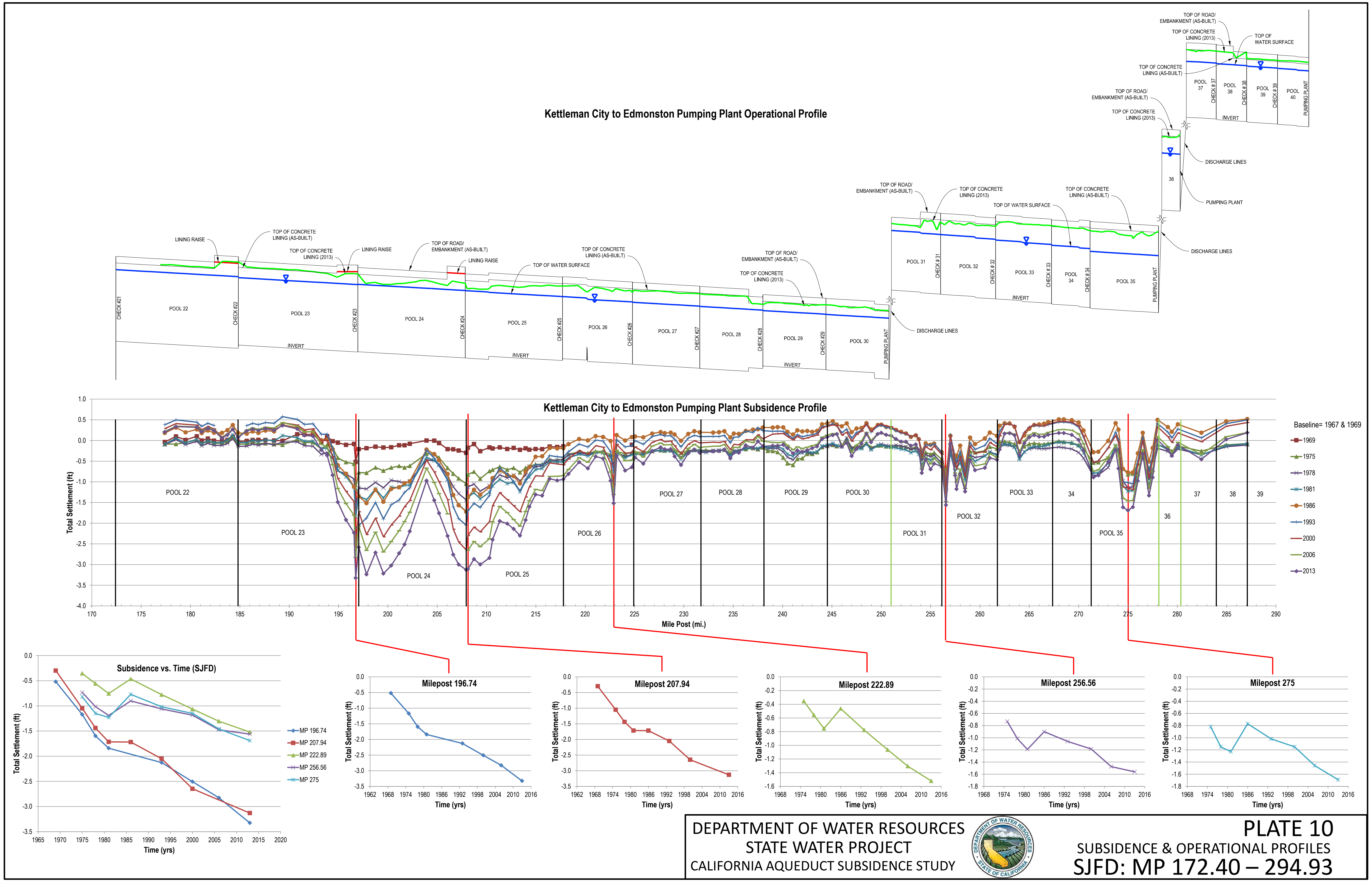


**Figure 6-2 Subsidence and Operational Profiles - San Joaquin Field Division - Plate 10**



## 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.

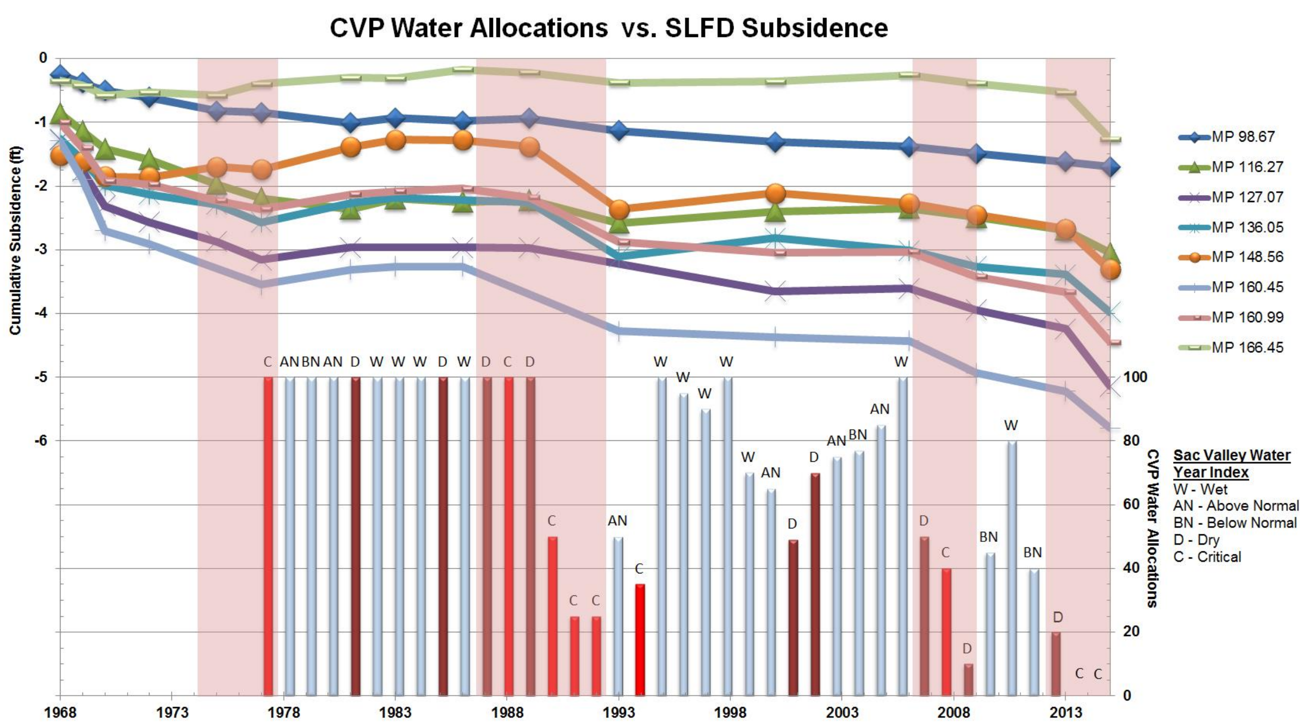
Looking at subsidence versus time at Mileposts 196.74, 207.94, 222.89, 256.56, and 275 (areas of maximum liner subsidence in five different pools), there appears to be no correlation between allocation and the amount of subsidence. Instead, the data suggests constant rates of subsidence over time (Figure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

**Table 6-1** **Total Subsidence in the San Luis Field Division, by Pool, up to 2015**

| Pool | Milepost | | Total Subsidence (feet) | | Benchmarks on Hard Structures | Benchmarks on Top of Liner | Total Benchmarks |
|---|---|---|---|---|---|---|---|
| | From | To | Minimum | Maximum | | | |
| 14 | 86.94 | 95.06 | +0.15 | +0.09 | 0 | 2 | 2 |
| 15 | 95.06 | 105.50 | +0.17 | -2.57 | 7 | 8 | 15 |
| 16 | 108.50 | 122.07 | -1.51 | -3.10 | 5 | 12 | 17 |
| 17 | 122.07 | 132.95 | -2.48 | -5.14 | 2 | 10 | 12 |
| 18 | 132.95 | 143.23 | -0.46 | -4.63 | 4 | 4 | 8 |
| 19 | 143.23 | 155.64 | -1.63 | -3.31 | 0 | 13 | 13 |
| 20 | 155.64 | 164.69 | -3.28 | -5.80 | 2 | 10 | 12 |
| 21 | 164.69 | 172.40 | -0.06 | -2.57 | 5 | 1 | 6 |
| 22 | 172.4 | 176.39 | 0.31 | -0.06 | 0 | 5 | 5 |
| | | | | **TOTAL** | **25** | **65** | **90** |

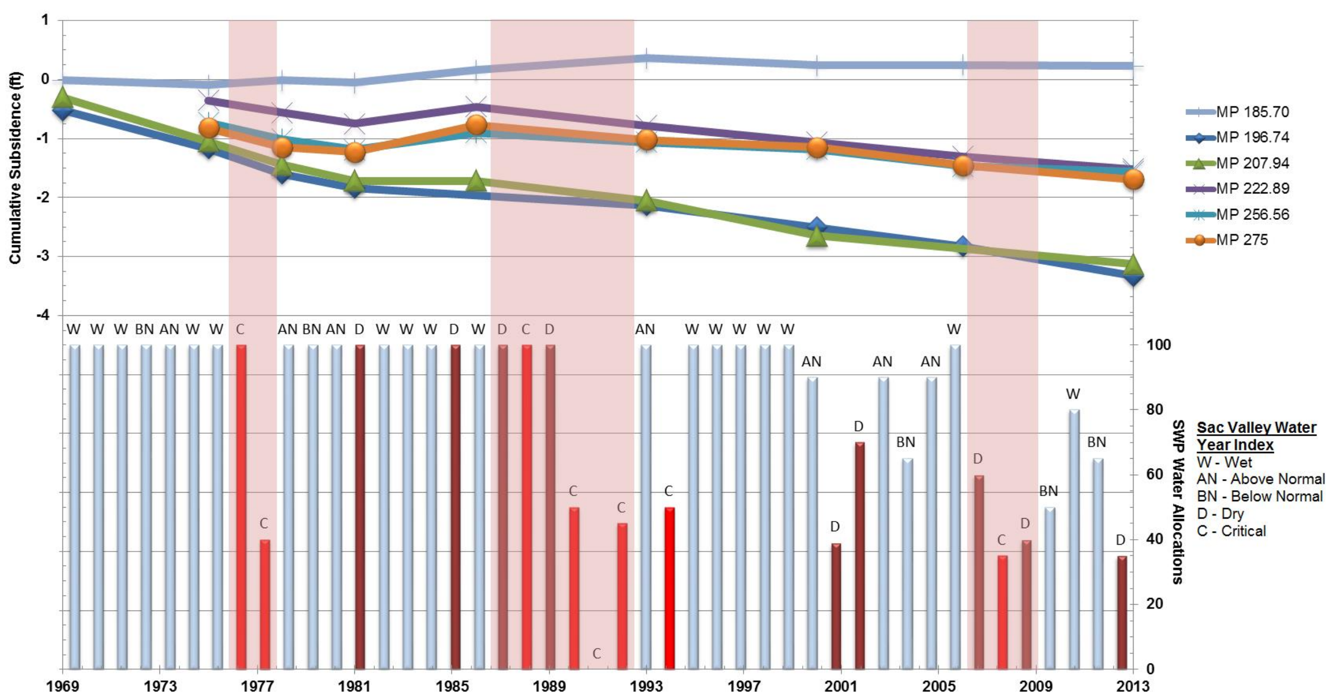
**Figure 6-3** **Central Valley Project Water Allocations versus San Luis Field Division Subsidence**



## Table 6-2 Total Subsidence in the San Joaquin Field Division, by Pool, up to 2013

| Pool | Milepost | | Total Subsidence (feet) | | Benchmarks on Hard Structures | Benchmarks on Top of Liner | Total Benchmarks |
|---|---|---|---|---|---|---|---|
| | From | To | Minimum | Maximum | | | |
| 22 | 172.40 | 184.82 | +0.33 | +0.05 | 0 | 11 | 11 |
| 23 | 184.82 | 197.05 | +0.36 | -3.33 | 0 | 17 | 17 |
| 24 | 197.05 | 207.94 | -0.97 | -3.24 | 0 | 14 | 14 |
| 25 | 207.94 | 217.79 | -0.93 | -3.11 | 1 | 15 | 16 |
| 26 | 217.79 | 224.92 | -0.37 | -1.52 | 1 | 9 | 10 |
| 27 | 224.92 | 231.73 | -0.17 | -0.59 | 0 | 11 | 11 |
| 28 | 231.73 | 238.11 | -0.02 | -0.43 | 0 | 11 | 11 |
| 29 | 238.11 | 244.54 | +0.08 | -0.28 | 0 | 13 | 13 |
| 30 | 244.54 | 250.99 | +0.19 | -0.38 | 0 | 13 | 13 |
| 31 | 250.99 | 256.14 | +0.04 | -0.79 | 0 | 11 | 11 |
| 32 | 256.14 | 261.72 | -0.08 | -1.56 | 0 | 11 | 11 |
| 33 | 261.72 | 267.36 | +0.18 | -0.35 | 0 | 11 | 11 |
| 34 | 267.36 | 271.27 | +0.18 | -0.24 | 0 | 5 | 5 |
| 35 | 271.27 | 278.13 | +0.01 | -1.69 | 0 | 13 | 13 |
| 36 | 278.13 | 280.36 | -0.11 | -0.37 | 1 | 2 | 3 |
| 37 | 280.36 | 283.95 | -0.46 | | 1 | 0 | 1 |
| 38 | 283.95 | 287.09 | +0.19 | +0.03 | 2 | 0 | 2 |
| | | | | TOTAL | **6** | **167** | **173** |

## Figure 6-4 State Water Project Water Allocations versus San Joaquin Field Division Subsidence



### 6.3.1 2013 to 2015 Data

Subsidence along the San Luis Canal has varied from year to year. Generally, subsidence has not been consistent. Such is the case from 2013 to March 2015 when some areas of the Aqueduct subsided more than in all previous years combined. Plate 29 shows a profile of subsidence along the San Luis Canal (Los Banos to the Kettleman City area) from Milepost 91.75 to 176.39, from 2013 to March 2015. This profile was created using leveling data taken from 22 benchmarks located on hard structures and from 60 benchmarks located on the top of the liner.

Based on the profile and on Table 6-3, most of the subsidence occurred in Pools 17 through 21. The greatest amount of subsidence was at Milepost 163.69 (Pool 20) with -1.26 feet from 2013 to March 2015. Pools 17, 18, 19, and 21 subsided as much as -0.98 foot at Milepost 131.33, -0.82 foot at Milepost 133.46, -0.84 foot at Milepost 148, and -0.94 foot at Milepost 165.03, respectively. Table 6-3 has a summary of subsidence within each pool.

**Table 6-3 San Luis Field Division Subsidence from 2013 to March 2015**

| Pool | Milepost | | Subsidence 2013-March 2015 (feet) | |
|---|---|---|---|---|
| | From | To | Lowest | Highest |
| 14 | 86.94 | 95.06 | +0.07 | +0.05 |
| 15 | 95.06 | 105.50 | +0.07 | -0.12 |
| 16 | 108.50 | 122.07 | -0.19 | -0.47 |
| 17 | 122.07 | 132.95 | -0.47 | -0.98 |
| 18 | 132.95 | 143.23 | -0.43 | -0.82 |
| 19 | 143.23 | 155.64 | -0.38 | -0.84 |
| 20 | 155.64 | 164.69 | -0.48 | -1.26 |
| 21 | 164.69 | 172.40 | -0.19 | -0.94 |
| 22 | 172.4 | 176.39 | +0.03 | -0.10 |

### 6.3.2 Pool 20 2015 Data

In August 2015, a survey was done to compare WSE System Control and Data Acquisition (SCADA) readings with actual WSEs (Section 6.5). Additional points were surveyed to evaluate the amount of subsidence at select locations that may have occurred since March 2015. Twelve points were surveyed in Pool 20; 10 of the points were on top of the liner and two of them were on turnouts. The August 2015 survey data showed the canal subsided from -0.38 foot at Milepost 162.69, to -0.74 foot at Milepost 156.87, since March 2015. Table 6-4 and Figures 6-5 and 6-6 provide a comparison of subsidence in Pool 20 from 2013 to March 2015, and from March 2015 to August 2015.

**Table 6-4 Pool 20 Subsidence from 2013 to March 2015, and from March 2015 to August 2015**

| Milepost | Subsidence 2013-March 2015 (feet) | Subsidence March 2015-August 2015 (feet) |
|---|---|---|
| 156.87 | -0.60 | -0.74 |
| 157.44 | -0.61 | -0.73 |
| 157.97 | -0.54 | -0.66 |
| 158.99 | -0.56 | -0.63 |
| 159.87 | -0.48 | -0.65 |
| 160.14 | -0.54 | -0.61 |
| 160.45 | -0.58 | -0.62 |
| 160.99 | -0.78 | -0.39 |
| 162.13 | -0.93 | -0.45 |
| 162.69 | -0.97 | -0.38 |
| 163.26 | -1.11 | -0.51 |
| 163.69 | -1.26 | -0.56 |

**Figure 6-5 Pool 20 Subsidence from 2013 to March 2015**

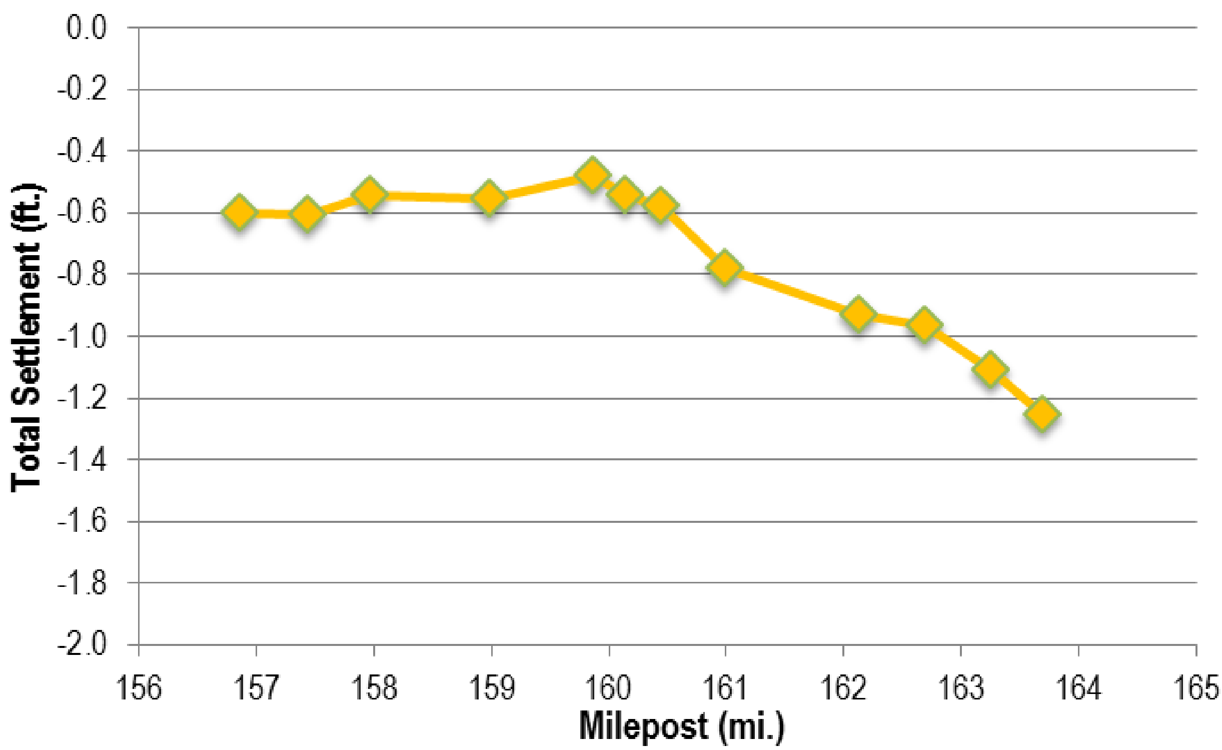
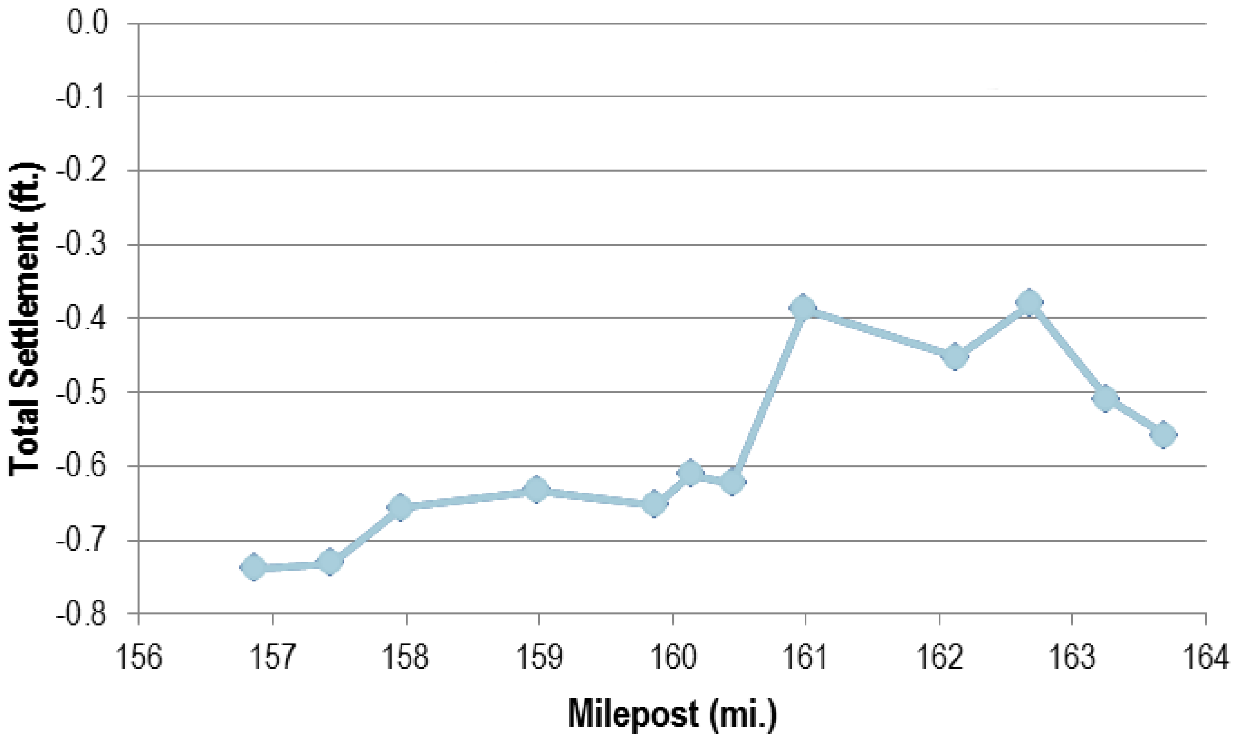


**Figure 6-6 Pool 20 Subsidence from March 2015 to August 2015**




## 6.4 Subsidence Rates

Subsidence rates for Pools 14 to 38 (Milepost 91.75 to 287.09) were calculated from 1967 to 2015 for the SLFD, and from 1967 to 2013 for the SJFD. The rates were determined using historical subsidence values in feet (the same values that were used to plot the subsidence profiles), divided by the number of years between each data set, and then multiplied by 12 to get a subsidence rate in inches per year (Tables 6-5, 6-6, and 6-7). As detailed in Section 6.1.3, subsidence rates between 1981 and 1986 are not going to be used for calculation or comparison for this study.

For the SLFD (Pools 14 to 21), the subsidence rates were the greatest between March 2015 and August 2015 (a critical dry year) with subsidence rates between -4.6 inches per 6 months at Milepost 162.69 and -8.9 inches per 6 months at Milepost 156.87. Other critical dry years (in red) were 1975 to 1977, 1989 to 1993, 2006 to 2009, and 2013 to 2015. Subsidence rates during these years ranged from an uplift of +2.1 inches per year at Milepost 174.83 to the most subsidence of -7.5 inches per year at Milepost 163.69. Other years with significant subsidence rates occurred from 1967 through 1970 (in green), a few years after the canal was built and in operation. The rates between these years ranged from +0.3 inches per year at Milepost 98.31 to -18.2 inches per year at Milepost 148.56, with the most subsidence occurring between 1967 and 1968, the canal's first year of operation. Rates from 1970 to 1972, 1972 to 1975, 1977 to 1981, 1986 to 1989, 1993 to 2000, 2000 to 2006, and 2009 to 2013 (in gray), ranged from a fairly small uplift of +1.9 inches per year at Milepost 118.82 to -1.9 inches per year at Milepost 128.07. Those years were wet years, according to the water index.

For the SJFD (Pools 22 to 38), the subsidence rates were significantly lower when compared to the SLFD rates. The highest rates for the SJFD were from 1975 to 1978 (critical dry years) with rates ranging between an uplift of +0.7 inch per year at Milepost 242 and -1.7 inches per year at Milepost 196.74. Other critical dry years (in red) were from 1986 to 1993, and from 2006 to 2013. The subsidence rates ranged during these years from an uplift of +0.3 inch per year to -1.0 inch per year. Other years where subsidence rates were significant were from 1967 to 1969, and from 1969 to 1975 (in green), a few years after the canal was built and in operation. The rates among these years ranged from +0.9 inch per year and -3.1 inches per year at Milepost 196.74. Rates from 1977 to 1981, 1993 to 2000, and 2000 to 2006 (in gray), ranged from an uplift of +0.6 inches per year at Milepost 189.33 to -1.1 inches per year. Those years were wet years according to the water index.

When comparing rates between field divisions, the overall subsidence rates were extremely higher in the SLFD than in the SJFD. The highest rate in the SLFD was -8.9 inches per 6 months. In the SJFD it was 3.1 inches per year.

## Table 6-5 Historic Subsidence Rates in San Luis Field Division, Inches per Year

| Pool | Mile | 1967-1968 | 1968-1969 | 1969-1970 | 1970-1972 | 1972-1975 | 1975-1977 | 1977-1981 | 1981-1983 | 1983-1986 | 1986-1989 | 1989-1993 | 1993-2000 | 2000-2006 | 2006-2009 | 2009-2013 | 2013-2015 | Mar 2015 - Aug 2015 (1) |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| P 14 | 91.75 | 0.0 | -0.5 | 0.5 | 0.0 | 0.0 | 0.2 | -0.1 | 0.2 | 0.1 | 0.0 | 0.1 | 0.0 | 0.0 | 0.0 | 0.0 | 0.3 | |
| | 94.26 | -0.3 | -0.7 | 0.2 | -0.2 | -0.1 | 0.3 | 0.0 | 0.5 | 0.0 | 0.2 | 0.1 | 0.0 | 0.0 | 0.0 | -0.1 | 0.4 | |
| Pool 15 | 95.97 | -0.8 | -0.6 | 0.2 | -0.2 | 0.0 | 0.8 | 0.0 | 0.7 | 0.0 | 0.2 | 0.1 | 0.0 | 0.0 | -0.1 | -0.1 | 0.4 | |
| | 97.26 | -1.0 | -0.7 | 0.3 | -0.3 | 0.0 | 0.5 | -0.2 | 0.6 | -0.1 | | | 0.0 | 0.1 | -0.1 | -0.1 | 0.3 | |
| | 98.31 | -2.1 | -1.0 | -0.3 | -0.5 | -0.6 | -0.1 | -0.5 | 0.4 | -0.2 | | | -0.2 | 0.0 | -0.3 | -0.3 | -0.2 | |
| | 98.67 | -3.0 | -1.7 | -1.3 | -0.7 | -0.9 | -0.2 | -0.5 | 0.5 | -0.2 | 0.2 | -0.6 | -0.3 | -0.1 | -0.4 | -0.4 | -0.4 | |
| | 99.61 | -3.2 | -1.1 | -1.0 | -0.7 | -0.7 | -0.2 | -0.4 | | | 0.3 | -0.6 | -0.1 | 0.0 | -0.4 | -0.3 | -0.3 | |
| | 100.04 | -4.4 | -1.3 | -1.2 | -0.7 | -0.8 | -0.2 | -0.3 | | | | | -0.1 | 0.0 | -0.3 | -0.3 | -0.3 | |
| | 100.55 | | -2.5 | -2.0 | -0.8 | -1.3 | -1.1 | -0.7 | 0.2 | -0.3 | 0.0 | -0.5 | -0.1 | -0.1 | -0.4 | -0.4 | -1.3 | |
| | 101.20 | -3.3 | -1.6 | -1.7 | -0.8 | -0.9 | -0.4 | -0.5 | 0.6 | -0.2 | 0.2 | -0.4 | -0.1 | 0.0 | -0.2 | -0.3 | -0.3 | |
| | 102.20 | -3.7 | -1.5 | -1.1 | -0.8 | -1.7 | -1.4 | -0.8 | 0.3 | -0.4 | 0.0 | -0.6 | -0.1 | 0.0 | -0.1 | -0.2 | -0.2 | |
| | 103.40 | -2.9 | -1.0 | -0.6 | -0.5 | -0.7 | | | 0.5 | -0.2 | 0.2 | -0.5 | 0.0 | 0.2 | -0.1 | -0.2 | -0.4 | |
| | 103.66 | -2.2 | -0.9 | -0.4 | -0.5 | -0.6 | 0.2 | -0.3 | 0.7 | -0.1 | 0.3 | -0.3 | 0.0 | 0.2 | -0.1 | -0.2 | -0.6 | |
| | 104.18 | -3.4 | -1.3 | -0.7 | -0.5 | -0.6 | | | 0.7 | -0.1 | 0.2 | -0.4 | 0.0 | 0.2 | -0.1 | -0.2 | -0.7 | |
| | 105.23 | -6.5 | -2.3 | -2.6 | -0.8 | -1.3 | | | 0.8 | | | | | | | | | |
| | 106.38 | -6.3 | -2.2 | -2.4 | -0.6 | -1.2 | | | 0.5 | -0.2 | 0.1 | -0.7 | -0.2 | 0.1 | -0.2 | -0.3 | -1.2 | |
| | 107.88 | -8.3 | -4.3 | -3.6 | -0.7 | -1.3 | -0.8 | -0.2 | 0.6 | -0.2 | 0.0 | -0.7 | -0.1 | 0.2 | -0.3 | -0.4 | -1.1 | |
| Pool 16 | 108.74 | -5.7 | -2.0 | -2.0 | -0.5 | -0.8 | | | 0.9 | -0.1 | 0.2 | -0.5 | 0.0 | 0.1 | -0.6 | -0.4 | -1.3 | |
| | 109.08 | -4.9 | -2.0 | -1.9 | -0.5 | -0.7 | -0.3 | 0.0 | 0.9 | 0.0 | | | -0.1 | 0.2 | -0.6 | -0.3 | -1.3 | |
| | 110.10 | -6.9 | -2.8 | -3.0 | -0.8 | -1.3 | -1.1 | -0.1 | 0.6 | -0.2 | | | 0.0 | 0.1 | -0.7 | -0.4 | -1.6 | |
| | 111.91 | -7.9 | -3.1 | -3.0 | -0.8 | -1.4 | | | 0.7 | -0.1 | 0.1 | -0.8 | -0.1 | 0.1 | -0.7 | -0.5 | -1.9 | |
| | 113.00 | -8.1 | -3.3 | -3.0 | -0.7 | -1.5 | -1.2 | -0.3 | 0.4 | -0.1 | 0.2 | -0.8 | 0.0 | 0.1 | -0.7 | -0.6 | -2.3 | |
| | 114.29 | -9.4 | -3.0 | -3.0 | -0.7 | -1.5 | -0.9 | -0.2 | 0.7 | -0.1 | 0.3 | -0.8 | 0.1 | 0.1 | -0.6 | -0.8 | -2.6 | |
| | 114.99 | -8.7 | -2.8 | -2.8 | -0.6 | -1.1 | -1.1 | -0.2 | 0.7 | -0.2 | 0.1 | -0.8 | 0.3 | 0.1 | -0.6 | -0.6 | -2.1 | |
| | 115.86 | -9.8 | -3.3 | -3.1 | -0.8 | -1.4 | -1.0 | -0.3 | 1.0 | -0.1 | | | 0.2 | 0.1 | -0.6 | -0.6 | -2.1 | |
| | 116.27 | -10.3 | -3.4 | -3.3 | -1.0 | -1.5 | -1.3 | -0.5 | 1.0 | -0.2 | 0.1 | -1.1 | 0.3 | 0.1 | -0.5 | -0.6 | | |
| | 116.96 | -9.9 | -3.1 | -3.0 | -0.8 | -1.3 | -1.2 | -0.6 | 1.3 | -0.2 | 0.3 | -0.8 | 0.3 | 0.1 | -0.6 | -0.6 | -2.2 | |
| | 117.82 | -10.3 | -3.2 | -2.9 | -0.9 | -1.1 | -1.0 | -0.6 | 1.7 | 0.0 | 0.4 | -0.7 | 0.3 | 0.1 | -0.5 | -0.5 | -2.0 | |
| | 118.10 | -10.4 | -3.3 | -2.8 | -0.9 | -1.1 | -0.8 | -0.5 | 1.9 | 0.0 | 0.3 | -0.8 | 0.2 | 0.1 | -0.4 | -0.5 | -2.1 | |
| | 118.82 | -9.2 | -2.9 | -2.4 | -0.9 | -0.9 | -0.8 | -0.5 | 1.9 | 0.0 | 0.2 | -0.9 | 0.2 | 0.1 | -0.4 | -0.5 | -2.4 | |
| | 119.25 | -10.9 | -2.8 | -2.4 | -0.9 | -0.7 | -0.7 | -0.6 | 1.7 | -0.2 | 0.1 | -1.1 | 0.2 | 0.0 | -0.4 | -0.5 | -2.5 | |
| | 119.96 | -14.9 | -4.6 | -3.5 | -2.3 | -0.4 | -1.1 | -0.4 | 1.9 | 0.2 | 0.3 | -1.0 | 0.3 | -0.1 | -0.2 | -0.5 | -2.5 | |
| | 120.39 | -12.1 | -3.2 | -2.3 | -0.3 | -0.2 | -0.4 | -0.8 | 1.4 | 0.1 | 0.2 | -1.1 | 0.2 | 0.0 | -0.6 | -0.6 | -2.6 | |
| | 121.10 | -11.0 | -3.3 | -2.6 | -0.4 | -0.6 | -0.8 | -0.3 | 1.2 | -0.1 | | | 0.1 | -0.1 | -0.7 | -0.6 | -2.8 | |
| Pool 17 | 122.96 | -9.4 | -3.1 | -2.7 | -0.5 | -0.5 | -0.4 | 0.1 | 1.0 | 0.0 | | | 0.3 | -0.1 | -0.6 | -0.7 | -2.8 | |
| | 123.70 | -9.9 | -3.9 | -4.3 | -1.1 | -0.8 | -0.5 | 0.3 | 0.8 | 0.1 | 0.1 | -1.4 | 0.2 | -0.1 | -0.5 | -0.5 | -3.0 | |
| | 124.97 | -10.7 | -3.6 | -3.9 | -1.0 | -1.0 | -1.1 | 0.2 | | | | | | | | | | |
| | 125.65 | -10.8 | -4.0 | -4.6 | -1.0 | -0.9 | -1.3 | -0.8 | | | -0.1 | -1.7 | 0.2 | -0.2 | -0.8 | -0.7 | -3.8 | |
| | 126.36 | -12.0 | -4.3 | -4.9 | -1.0 | -0.7 | -1.1 | 0.7 | | | | -1.9 | 0.3 | -0.1 | -0.9 | -0.7 | -4.6 | |
| | 127.07 | -15.4 | -5.8 | -6.6 | -1.5 | -1.2 | -1.7 | 0.6 | | | 0.0 | | | 0.1 | -1.3 | -0.9 | -5.4 | |
| | 127.50 | -17.1 | -5.9 | -6.6 | -1.5 | -1.0 | -1.8 | 0.7 | | | 0.0 | -2.0 | 0.3 | -0.2 | -1.2 | -0.8 | -5.8 | |
| | 128.07 | -17.3 | -6.5 | -7.8 | -1.9 | -1.4 | -2.2 | 0.6 | | | | | | | | | | |
| | 129.32 | -13.0 | -5.3 | -6.3 | | | | 0.8 | 0.9 | 0.1 | -0.2 | -1.9 | 0.2 | -0.1 | -1.3 | -0.7 | | |
| | 129.34 | -13.0 | -5.3 | -6.3 | | | | | | | | | | | | | | |
| | 130.13 | -12.2 | -5.7 | -6.1 | -1.4 | -0.6 | -2.4 | 0.7 | | | | | | | | | | |
| | 131.33 | | | -6.0 | | | -1.5 | 0.9 | 0.7 | | | | -0.2 | -0.4 | -1.6 | -0.9 | -5.9 | |
| Pool 18 | 133.10 | -12.3 | -4.9 | -5.8 | -1.4 | -0.7 | | | 0.2 | 0.0 | | | 0.2 | -0.5 | -1.7 | -0.9 | -3.9 | |
| | 133.43 | | | | | -0.6 | -1.4 | 1.0 | 0.6 | 0.1 | | | 0.3 | -0.3 | -1.4 | -1.7 | -5.2 | |
| | 133.46 | -11.7 | -4.7 | -5.8 | | | -1.5 | 0.9 | 0.7 | | | | 0.3 | -0.2 | -1.5 | -0.4 | -4.9 | |
| | 135.17 | | | | | | | 1.1 | 0.6 | 0.2 | | | 0.5 | -0.3 | -1.2 | -0.3 | -4.3 | |
| | 136.05 | -15.1 | -3.9 | -4.9 | -0.8 | -0.7 | -1.7 | 0.9 | 0.5 | | | -2.6 | 0.5 | -0.4 | -1.0 | -0.4 | -3.5 | |
| | 137.32 | -11.6 | -2.8 | -3.1 | -0.1 | -0.1 | -0.3 | 1.0 | 0.4 | 0.3 | | | 0.6 | -0.3 | -0.8 | -0.3 | -2.9 | |
| | 138.74 | -9.6 | -2.0 | -2.3 | -0.4 | -0.2 | 0.2 | 0.7 | 0.4 | 0.3 | | | | | | | | |
| | 140.00 | -7.7 | -2.0 | -1.9 | -0.2 | 0.0 | 0.5 | 0.6 | 0.4 | 0.2 | -0.1 | -1.6 | 0.5 | -0.3 | -0.4 | -0.2 | -3.1 | |
| | 142.40 | -3.0 | -0.2 | -0.2 | 0.0 | 0.1 | 0.7 | 0.3 | 0.7 | 0.2 | 0.0 | -1.2 | 0.6 | -0.1 | 0.0 | 0.0 | -2.6 | |
| Pool 19 | 143.84 | -6.2 | -0.9 | -0.6 | -0.2 | 0.1 | 0.2 | 0.5 | 0.5 | 0.2 | | | 0.6 | -0.3 | -0.6 | -0.2 | -3.6 | |
| | 144.88 | -10.2 | -1.7 | -1.1 | -0.2 | 0.3 | 0.1 | 0.8 | 0.6 | 0.1 | | | 0.5 | -0.3 | -1.0 | -0.3 | -3.8 | |
| | 145.58 | -11.6 | -1.5 | -1.0 | -0.3 | 0.4 | -0.2 | 0.9 | 0.7 | 0.0 | | | 0.7 | -0.4 | -0.9 | -0.4 | -3.8 | |
| | 146.72 | -14.4 | -0.9 | -0.9 | 0.0 | 0.6 | -0.2 | 1.0 | 0.6 | 0.1 | -0.5 | -2.2 | 0.6 | -0.3 | -0.7 | -0.5 | -3.5 | |
| | 147.43 | -14.8 | -1.1 | -1.7 | -0.1 | 0.7 | -0.1 | 1.0 | 0.6 | 0.0 | | | 0.5 | -0.2 | -0.6 | -0.6 | -4.3 | |
| | 148.00 | -16.0 | -1.3 | -2.3 | -0.1 | 0.7 | -0.1 | 1.0 | 0.7 | -0.1 | -0.4 | -2.4 | 0.5 | -0.2 | -0.6 | -0.7 | -5.1 | |
| | 148.56 | -18.2 | -1.2 | -2.8 | -0.1 | 0.6 | -0.2 | 1.1 | 0.7 | 0.0 | -0.4 | -3.0 | 0.4 | -0.3 | -0.7 | -0.6 | -3.9 | |
| | 149.14 | -13.0 | -1.2 | -2.6 | 0.0 | 0.6 | -0.2 | 0.9 | 0.5 | -0.1 | | | 0.3 | -0.3 | -0.8 | -0.7 | -2.9 | |
| | 150.42 | -13.0 | -2.5 | -3.9 | -0.6 | 0.3 | -0.7 | 1.3 | 0.3 | -0.1 | -0.6 | -2.3 | 0.5 | -0.2 | -0.9 | -0.5 | -2.3 | |
| | 152.26 | -13.3 | -2.4 | -4.1 | -0.9 | 0.2 | -0.8 | 1.0 | 0.9 | -0.1 | -0.7 | -2.5 | 0.6 | -0.3 | -1.0 | -0.5 | -2.3 | |
| | 153.83 | -7.0 | -3.0 | -4.4 | -1.0 | 0.2 | -0.9 | 1.0 | 0.8 | -0.3 | -0.8 | -2.7 | 0.7 | -0.3 | -1.0 | -0.5 | | |
| | 154.39 | -6.1 | -2.6 | -3.9 | -0.7 | 0.4 | -0.8 | 1.0 | 0.8 | 0.0 | -0.6 | -2.5 | 0.7 | -0.3 | -1.0 | -0.4 | -3.2 | |
| | 154.95 | -6.7 | -2.8 | -4.4 | -1.0 | 0.1 | -1.2 | 0.8 | 0.6 | -0.1 | -0.7 | -2.6 | 0.7 | -0.2 | -1.1 | -0.4 | -3.6 | |
| Pool 20 | 156.87 | -8.5 | -2.4 | -5.1 | -1.4 | -0.3 | -0.9 | 1.0 | 0.7 | 0.1 | -0.4 | -2.6 | 0.5 | -0.2 | -1.3 | -0.7 | -3.6 | -8.9 |
| | 157.44 | -9.5 | -2.4 | -5.3 | -1.4 | -0.4 | -0.6 | 1.0 | 0.7 | 0.1 | -0.3 | -2.4 | 0.4 | -0.2 | -1.5 | -0.9 | -3.6 | -8.8 |
| | 157.97 | -10.1 | -2.5 | -5.7 | -1.2 | -0.4 | -0.5 | 1.0 | 0.8 | 0.1 | -0.2 | -2.7 | 0.3 | -0.2 | -1.4 | -0.7 | -3.3 | -7.9 |
| | 158.99 | -11.5 | -3.1 | -6.3 | -0.9 | -0.5 | -0.5 | 1.0 | 0.8 | 0.1 | -0.3 | -2.4 | 0.2 | -0.1 | -1.6 | -0.6 | -3.3 | -7.6 |
| | 159.87 | -11.7 | -3.4 | -6.3 | -0.6 | -0.7 | -0.5 | 1.0 | 0.7 | 0.1 | -0.3 | -2.3 | 0.1 | -0.1 | -1.6 | -0.6 | -2.9 | -7.8 |
| | 160.14 | -12.2 | -4.1 | -6.1 | -0.1 | -0.8 | -0.7 | 0.9 | 0.4 | 0.1 | -0.5 | -2.3 | -0.5 | -0.1 | -1.8 | -0.6 | -3.2 | -7.3 |
| | 160.45 | -15.5 | -7.2 | -9.8 | -1.2 | | | 0.7 | 0.3 | 0.0 | | | -0.2 | -0.1 | -2.0 | -0.9 | -3.5 | -7.5 |
| | 160.99 | -12.0 | -4.8 | -6.2 | -0.3 | -1.1 | -0.8 | 0.7 | 0.3 | 0.1 | -0.6 | -2.1 | -0.3 | 0.1 | -1.6 | -0.7 | -4.7 | -4.6 |
| | 162.13 | -10.6 | -4.8 | -6.4 | -0.7 | -1.2 | 0.1 | 0.3 | 0.1 | 0.2 | -0.4 | -1.3 | -0.6 | 0.0 | -1.3 | -0.8 | -5.6 | -5.4 |
| | 162.69 | -10.6 | -4.5 | -6.5 | -1.0 | -1.5 | 0.3 | 0.0 | 0.1 | 0.3 | -0.2 | -1.0 | -0.3 | 0.1 | -0.9 | -0.6 | -5.8 | -4.6 |
| | 163.26 | -9.8 | -5.5 | -6.8 | -1.1 | -1.4 | 0.5 | 0.0 | 0.0 | 0.4 | -0.2 | -0.9 | -0.2 | 0.1 | -0.8 | -0.6 | -6.7 | -6.1 |
| | 163.69 | -10.1 | -4.5 | -6.1 | -1.0 | -1.3 | 0.5 | 0.0 | -0.1 | 0.5 | -0.2 | -0.8 | -0.2 | 0.2 | -0.8 | -0.6 | -7.5 | -6.7 |
| Pool 21 | 165.03 | -8.6 | -2.7 | -3.5 | -0.2 | -0.6 | 1.1 | 0.1 | | | | | -0.2 | 0.3 | -0.8 | -0.4 | -5.6 | |
| | 166.45 | -4.2 | -0.8 | -1.9 | 0.3 | -0.2 | 1.1 | 0.3 | -0.1 | 0.6 | -0.2 | -0.5 | 0.0 | 0.2 | -0.5 | -0.4 | -4.4 | |
| | 167.85 | -5.2 | -1.2 | -1.9 | 0.4 | -0.2 | 1.1 | 0.3 | -0.1 | 0.6 | -0.2 | -0.5 | 0.0 | 0.2 | -0.5 | -0.4 | -3.6 | |
| | 169.40 | -3.0 | -0.9 | -1.3 | 0.4 | -0.5 | 1.3 | 0.1 | -0.1 | 0.5 | -0.1 | -0.4 | 0.1 | 0.1 | -0.3 | -0.4 | -2.5 | |
| | 170.42 | -1.5 | -0.3 | -1.0 | 0.4 | -0.4 | 1.6 | 0.1 | -0.2 | 0.4 | -0.1 | -0.3 | 0.1 | 0.0 | -0.1 | -0.3 | -2.4 | |
| | 172.40 | -1.5 | -0.3 | -0.4 | 0.2 | -0.3 | 2.0 | -0.1 | -0.3 | 0.4 | 0.1 | -0.2 | 0.1 | 0.0 | 0.1 | -0.1 | -1.1 | |
| Pool 22 | 173.56 | -1.1 | -0.1 | -0.4 | 0.1 | -0.3 | 1.9 | -0.2 | -0.4 | 0.2 | | | 0.1 | 0.0 | 0.1 | -0.1 | -0.6 | |
| | 174.07 | -0.9 | 0.0 | -0.4 | 0.2 | -0.2 | 2.1 | -0.1 | -0.4 | 0.2 | | | 0.2 | 0.1 | 0.1 | 0.0 | -0.2 | |
| | 174.83 | -0.6 | 0.1 | -0.4 | 0.2 | -0.1 | 2.1 | -0.1 | -0.4 | 0.2 | 0.1 | -0.1 | 0.0 | 0.1 | 0.0 | 0.0 | -0.3 | |
| | 175.54 | -0.7 | 0.1 | -0.5 | 0.1 | -0.2 | 1.7 | -0.2 | -0.6 | 0.1 | 0.0 | -0.1 | 0.0 | 0.0 | 0.0 | 0.0 | 0.2 | |
| | 176.39 | | | | | | 2.0 | -0.1 | -0.4 | 0.1 | -0.1 | 0.0 | 0.0 | 0.0 | 0.0 | 0.0 | 0.2 | |

-7 Subsidence Rate reported per location, per period. Data bar depicts relative magnitude of subsidence rates. Red is negative (bad).
Period During Aqueduct and Water Distribution Construction in San Luis Field Division.
Normal to Wet Hydrologic Years (Water Index).
Dry to Critical Hydrologic Years (Water Index).
**(1)** Results for Mar 2015 - Aug 2015 are recorded in Inch per 6 month period.
- Columns for 1967-1968 are larger than the other columns to correctly scale data bars.

## Table 6-6 Historic Subsidence Rates in San Joaquin Field Division, Inches per Year (1 of 2)

-7 Subsidence Rate reported per location, per period. Data bar depicts relative magnitude of subsidence rates. Red is negative (bad).
Period During Aqueduct and Water Distribution Construction in San Joaquin Field Division.
Normal to Wet Hydrologic Years (Water Index).
Dry to Critical Hydrologic Years (Water Index).

**Pool 22**

| Year \ Mile | 177.40 | 178.53 | 179.50 | 180.64 | 181.14 | 181.76 | 182.39 | 183.19 | 183.74 | 184.29 | 184.82 |
|---|---|---|---|---|---|---|---|---|---|---|---|
| 1967-1969 | -0.2 | 0.6 | 0.1 | 0.6 | 0.0 | 0.3 | 0.0 | -0.1 | 0.1 | 0.6 | -0.2 |
| 1969-1975 | -0.1 | -0.4 | -0.1 | -0.2 | -0.2 | -0.1 | -0.1 | -0.1 | -0.1 | 0.0 | -0.1 |
| 1975-1978 | -0.1 | 0.4 | -0.2 | -0.1 | -0.2 | -0.2 | -0.2 | -0.2 | -0.2 | -0.1 | -0.2 |
| 1978-1981 | 0.1 | 0.1 | 0.2 | 0.2 | 0.2 | 0.2 | 0.3 | 0.3 | 0.3 | 0.4 | 0.4 |
| 1981-1986 | 0.7 | 0.6 | 0.6 | 0.6 | 0.6 | 0.6 | 0.6 | 0.6 | 0.6 | 0.6 | 0.6 |
| 1986-1993 | 0.3 | 0.3 | | 0.3 | 0.3 | 0.3 | 0.3 | | | | |
| 1993-2000 | -0.2 | -0.2 | | -0.1 | -0.1 | -0.1 | -0.1 | | | | |
| 2000-2006 | -0.1 | -0.1 | | -0.1 | -0.1 | -0.1 | 0.0 | 0.0 | 0.0 | 0.0 | 0.0 |
| 2006-2013 | 0.0 | 0.0 | | 0.0 | 0.0 | 0.0 | 0.0 | 0.0 | 0.0 | 0.0 | 0.0 |

**Pool 23**

| Year \ Mile | 185.70 | 186.30 | 186.88 | 187.60 | 188.47 | 189.33 | 190.82 | 191.55 | 192.41 | 193.26 | 193.85 | 194.41 | 194.93 | 195.76 | 196.57 | 196.74 | 197.05 |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| 1967-1969 | -0.1 | 0.1 | 0.1 | 0.1 | 0.0 | 0.0 | 0.9 | 0.9 | 0.9 | -0.2 | 0.6 | -0.1 | -0.3 | -0.6 | -0.5 | -3.1 | -1.3 |
| 1969-1975 | -0.2 | -0.1 | -0.2 | -0.2 | -0.2 | -0.2 | -0.3 | -0.4 | -0.4 | -0.3 | -0.4 | -0.3 | -0.7 | -0.8 | -1.0 | -1.3 | -1.1 |
| 1975-1978 | -0.2 | -0.1 | -0.2 | 0.0 | -0.1 | 0.3 | -0.2 | -0.3 | -0.4 | -0.5 | -0.7 | -1.0 | -1.3 | -1.1 | -1.5 | -1.7 | -1.5 |
| 1978-1981 | 0.3 | 0.4 | 0.3 | 0.3 | 0.4 | 0.6 | 0.4 | 0.4 | 0.5 | 0.6 | 0.4 | 0.3 | 0.1 | -0.2 | -0.8 | -1.0 | -0.8 |
| 1981-1986 | 0.5 | 0.5 | 0.5 | 0.5 | 0.6 | 0.6 | 0.6 | 0.5 | 0.5 | 0.5 | 0.5 | 0.3 | 0.3 | 0.2 | 0.0 | | 0.0 |
| 1986-1993 | 0.3 | 0.3 | 0.3 | 0.3 | 0.3 | 0.3 | 0.3 | 0.3 | 0.3 | 0.2 | 0.2 | 0.0 | -0.1 | | | | |
| 1993-2000 | -0.2 | -0.2 | -0.2 | -0.2 | -0.2 | -0.2 | -0.2 | -0.2 | -0.2 | -0.3 | -0.3 | -0.4 | -0.5 | | | -0.7 | |
| 2000-2006 | 0.0 | 0.0 | 0.1 | 0.0 | 0.0 | 0.0 | 0.0 | -0.1 | -0.1 | -0.2 | -0.3 | -0.4 | -0.5 | -0.6 | -0.7 | -0.6 | -0.7 |
| 2006-2013 | 0.0 | -0.1 | -0.1 | -0.1 | -0.1 | -0.1 | -0.1 | -0.1 | -0.2 | -0.2 | -0.4 | -0.4 | -0.6 | -0.7 | -0.7 | -0.9 | -0.9 |

**Pool 24**

| Year \ Mile | 197.84 | 198.75 | 199.55 | 200.34 | 201.14 | 201.69 | 202.22 | 203.92 | 204.70 | 205.19 | 206.06 | 206.63 | 207.18 | 207.94 |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| 1967-1969 | -1.1 | -0.9 | -1.0 | -1.0 | -0.7 | -0.7 | -0.5 | 0.0 | 0.0 | -0.4 | -1.3 | -1.3 | -1.5 | -1.8 |
| 1969-1975 | -1.2 | -1.0 | -1.1 | -1.0 | -1.0 | -1.1 | -1.1 | -0.6 | -0.6 | -0.7 | -0.9 | -1.2 | -1.4 | -1.5 |
| 1975-1978 | -1.6 | -1.4 | -1.6 | -1.2 | -1.5 | -1.5 | -1.5 | -0.6 | -0.6 | -0.6 | -0.8 | -1.3 | -1.5 | -1.6 |
| 1978-1981 | -1.0 | -0.6 | -1.0 | -0.7 | -0.6 | -0.2 | -0.4 | 0.2 | 0.0 | -0.1 | -0.3 | -0.9 | -1.1 | -1.1 |
| 1981-1986 | -0.2 | -0.1 | -0.2 | -0.1 | 0.0 | 0.1 | 0.2 | 0.5 | 0.4 | 0.3 | 0.3 | 0.0 | 0.0 | 0.0 |
| 1986-1993 | -0.6 | -0.5 | -0.7 | -0.7 | -0.5 | -0.4 | -0.3 | 0.0 | -0.2 | -0.2 | -0.3 | -0.5 | -0.6 | -0.6 |
| 1993-2000 | -0.7 | -0.6 | -0.7 | -0.8 | -0.7 | -0.6 | -0.5 | -0.4 | -0.6 | -0.7 | -0.8 | -0.9 | -1.0 | -1.0 |
| 2000-2006 | -0.8 | -0.7 | -0.7 | -0.8 | -0.7 | -0.7 | -0.6 | -0.4 | -0.5 | -0.6 | -0.7 | | | |
| 2006-2013 | -1.0 | -0.8 | -0.9 | -1.0 | -0.9 | -0.9 | -0.8 | -0.5 | -0.7 | -0.8 | -0.9 | | | |

**Pool 25**

| Year \ Mile | 208.11 | 208.73 | 209.36 | 210.30 | 210.61 | 211.34 | 212.10 | 212.75 | 213.39 | 213.96 | 214.34 | 214.97 | 215.64 | 216.40 | 217.16 | 217.78 |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| 1967-1969 | -1.1 | -0.5 | -1.6 | -1.0 | -1.2 | -1.0 | -1.2 | -1.1 | -1.4 | -1.1 | -1.3 | -1.2 | -1.3 | -0.9 | -0.9 | -0.8 |
| 1969-1975 | -1.3 | -1.3 | -1.3 | -1.1 | -1.0 | -0.9 | -1.0 | -1.0 | -1.1 | -0.9 | -0.8 | -0.7 | -0.7 | -0.6 | -0.6 | -0.6 |
| 1975-1978 | -1.2 | -1.2 | -1.2 | -1.4 | -1.2 | -0.8 | -0.8 | -0.7 | -0.8 | -0.7 | 0.0 | -0.2 | -0.2 | 0.0 | 0.0 | 0.0 |
| 1978-1981 | -1.0 | -0.8 | -0.7 | -0.6 | -0.4 | -0.4 | -0.5 | -0.7 | -1.0 | -0.6 | -1.0 | -0.2 | -0.3 | -0.1 | -0.2 | -0.2 |
| 1981-1986 | 0.0 | 0.2 | 0.2 | 0.3 | 0.5 | 0.5 | 0.5 | 0.4 | 0.5 | 0.6 | 0.6 | 0.7 | 0.7 | 0.8 | 0.8 | 0.8 |
| 1986-1993 | -0.6 | -0.6 | -0.5 | -0.3 | -0.1 | 0.1 | 0.0 | -0.2 | -0.3 | -0.3 | -0.3 | -0.3 | -0.4 | -0.3 | -0.3 | -0.4 |
| 1993-2000 | -1.0 | -1.0 | -1.0 | -1.1 | -1.0 | -1.0 | -1.1 | -1.0 | -0.9 | -0.7 | -0.5 | -0.5 | -0.5 | -0.3 | -0.3 | -0.3 |
| 2000-2006 | -0.7 | -0.7 | -0.7 | -0.8 | -0.8 | -0.7 | -0.6 | -0.7 | -0.7 | -0.7 | -0.7 | -0.7 | -0.7 | -0.7 | -0.6 | -0.5 |
| 2006-2013 | -0.8 | -0.7 | -0.7 | -0.8 | -0.7 | -0.6 | -0.5 | -0.4 | -0.4 | -0.4 | -0.2 | -0.2 | -0.2 | -0.1 | -0.2 | -0.2 |

**Pool 26**

| Year \ Mile | 218.31 | 219.38 | 220.27 | 221.06 | 221.81 | 222.58 | 222.89 | 223.19 | 224.05 | 224.73 |
|---|---|---|---|---|---|---|---|---|---|---|
| 1967-1969 | | | | | | | | | | -0.6 |
| 1969-1975 | | | | | | | | | | -0.4 |
| 1975-1978 | -0.1 | -0.1 | -0.1 | -0.1 | -0.1 | -0.2 | -0.8 | -0.1 | -0.3 | -0.2 |
| 1978-1981 | -0.3 | -0.1 | -0.2 | 0.0 | 0.0 | -0.1 | -0.8 | 0.0 | -0.1 | 0.0 |
| 1981-1986 | 0.8 | 0.9 | 0.9 | 1.0 | 0.9 | 0.9 | 0.7 | 1.0 | 1.0 | 1.0 |
| 1986-1993 | -0.3 | -0.2 | -0.3 | -0.2 | -0.2 | -0.4 | -0.5 | -0.2 | -0.3 | -0.3 |
| 1993-2000 | -0.3 | -0.3 | -0.3 | -0.2 | -0.3 | -0.5 | -0.5 | -0.2 | -0.3 | -0.3 |
| 2000-2006 | -0.5 | -0.4 | -0.4 | -0.3 | -0.3 | -0.4 | -0.5 | -0.3 | -0.4 | -0.4 |
| 2006-2013 | -0.2 | -0.2 | -0.3 | -0.2 | -0.2 | -0.4 | -0.4 | -0.2 | -0.4 | -0.3 |

**Pool 27**

| Year \ Mile | 225.05 | 225.89 | 226.79 | 227.64 | 228.06 | 228.67 | 229.06 | 229.71 | 230.44 | 231.06 | 231.60 |
|---|---|---|---|---|---|---|---|---|---|---|---|
| 1967-1969 | | | | | | | | | | | |
| 1969-1975 | | | | | | | | | | | |
| 1975-1978 | -0.1 | -0.2 | 0.0 | 0.0 | 0.0 | 0.0 | -0.1 | -0.4 | -0.1 | -0.1 | -0.1 |
| 1978-1981 | 0.0 | -0.1 | 0.0 | 0.0 | 0.0 | 0.0 | -0.1 | -0.2 | -0.1 | 0.0 | 0.0 |
| 1981-1986 | 1.0 | 1.0 | 1.0 | 1.1 | 1.0 | 1.1 | 1.0 | 1.1 | 1.1 | 1.1 | 1.1 |
| 1986-1993 | -0.2 | -0.3 | -0.1 | -0.1 | -0.1 | -0.1 | -0.2 | -0.3 | -0.2 | -0.2 | -0.2 |
| 1993-2000 | -0.3 | -0.3 | -0.2 | -0.2 | -0.2 | -0.2 | -0.3 | -0.3 | -0.3 | -0.2 | -0.3 |
| 2000-2006 | -0.2 | -0.3 | -0.2 | -0.2 | -0.2 | -0.2 | -0.2 | -0.2 | -0.2 | -0.2 | -0.2 |
| 2006-2013 | -0.2 | -0.3 | -0.2 | -0.2 | -0.2 | -0.2 | -0.3 | -0.3 | -0.2 | -0.2 | -0.2 |

**Pool 28**

| Year \ Mile | 232.24 | 232.96 | 233.56 | 234.10 | 234.71 | 235.06 | 235.56 | 236.27 | 237.00 | 237.50 | 238.00 |
|---|---|---|---|---|---|---|---|---|---|---|---|
| 1967-1969 | | | | | | | | | | | |
| 1969-1975 | | | | | | | | | | | |
| 1975-1978 | -0.1 | 0.0 | 0.0 | 0.0 | -0.1 | 0.0 | 0.0 | 0.1 | 0.1 | 0.0 | 0.1 |
| 1978-1981 | -0.1 | 0.0 | 0.0 | 0.0 | -0.1 | -0.1 | -0.1 | -0.1 | 0.0 | 0.1 | 0.0 |
| 1981-1986 | | 1.1 | 1.1 | 1.1 | 1.1 | 1.1 | 1.0 | 1.0 | 1.1 | 1.1 | 1.1 |
| 1986-1993 | | -0.2 | -0.2 | -0.2 | -0.2 | -0.2 | -0.1 | 0.0 | -0.1 | 0.1 | -0.1 |
| 1993-2000 | | -0.2 | -0.2 | -0.3 | -0.3 | -0.3 | -0.3 | -0.4 | -0.3 | -0.3 | -0.3 |
| 2000-2006 | | -0.2 | -0.2 | -0.2 | -0.1 | -0.1 | -0.1 | -0.1 | -0.1 | -0.1 | -0.1 |
| 2006-2013 | | -0.2 | -0.2 | -0.2 | -0.2 | -0.2 | -0.2 | -0.2 | -0.2 | -0.2 | -0.2 |

## Table 6-7 Historic Subsidence Rates in San Joaquin Field Division (2 of 2) (Inches per Year)

| Year \ Mile | Pool 29 | | | | | | | | | | | | | Pool 30 | | | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | 238.50 | 239.00 | 239.50 | 240.07 | 240.50 | 241.06 | 241.50 | 242.00 | 242.50 | 243.00 | 243.50 | 244.00 | 244.53 | 245.09 | 245.50 | 246.00 | 246.50 | 247.00 | 247.50 | 248.00 | 248.50 | 248.97 | 249.65 | 250.00 | 250.50 | 250.99 |
| 1967-1969 | | | | | | | | | | | | | | | | | | | | | | | | | | |
| 1969-1975 | | | | | | | | | | | | | | | | | | | | | | | | | | |
| 1975-1978 | 0.2 | 0.3 | 0.4 | 0.6 | 0.6 | 0.5 | 0.4 | 0.7 | 0.1 | 0.1 | 0.1 | 0.0 | -0.1 | 0.0 | 0.0 | 0.0 | 0.0 | -0.1 | -0.1 | -0.1 | -0.1 | -0.1 | 0.0 | 0.1 | 0.1 | 0.0 |
| 1978-1981 | 0.1 | 0.1 | 0.2 | 0.3 | 0.3 | 0.3 | 0.3 | -0.2 | 0.1 | 0.1 | 0.0 | 0.1 | 0.1 | 0.1 | 0.0 | -0.1 | -0.2 | -0.2 | -0.2 | -0.1 | 0.0 | -0.1 | 0.0 | -0.1 | -0.1 | -0.1 |
| 1981-1986 | | 1.1 | 1.1 | 1.3 | 1.3 | 1.3 | 1.3 | 1.2 | 1.2 | 1.2 | 1.2 | 1.3 | 1.3 | 1.3 | 1.2 | 1.2 | 1.0 | 1.1 | 1.2 | 1.2 | 1.2 | 1.2 | 1.2 | 1.2 | 1.1 | 1.1 |
| 1986-1993 | | -0.2 | -0.2 | -0.2 | -0.1 | -0.2 | -0.2 | -0.2 | -0.2 | -0.1 | -0.1 | -0.1 | -0.1 | -0.1 | 0.0 | -0.3 | -0.2 | -0.2 | -0.2 | -0.1 | -0.1 | -0.1 | 0.0 | 0.0 | 0.0 | 0.0 |
| 1993-2000 | | -0.1 | -0.1 | -0.1 | -0.1 | -0.1 | -0.1 | -0.1 | -0.1 | -0.1 | -0.1 | 0.0 | 0.0 | -0.1 | 0.0 | -0.1 | -0.1 | -0.1 | -0.1 | 0.0 | 0.0 | 0.0 | 0.0 | 0.0 | 0.1 | 0.1 |
| 2000-2006 | | -0.4 | -0.4 | -0.4 | -0.4 | -0.5 | -0.5 | -0.5 | -0.5 | -0.5 | -0.5 | -0.5 | -0.4 | -0.4 | -0.4 | -0.5 | -0.4 | -0.5 | -0.5 | -0.5 | -0.4 | -0.4 | -0.4 | -0.4 | -0.4 | -0.5 |
| 2006-2013 | | -0.1 | -0.1 | -0.1 | -0.1 | -0.1 | -0.1 | -0.1 | -0.1 | -0.1 | -0.1 | -0.1 | -0.1 | -0.1 | 0.0 | -0.1 | -0.1 | -0.2 | -0.1 | -0.1 | -0.1 | -0.1 | 0.0 | 0.0 | 0.0 | 0.0 |

| Year \ Mile | Pool 31 | | | | | | | | | | | Pool 32 | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | 251.50 | 252.01 | 252.50 | 253.00 | 253.50 | 253.80 | 254.11 | 254.50 | 255.00 | 255.36 | 256.12 | 256.56 | 257.00 | 257.48 | 257.63 | 258.00 | 258.50 | 259.00 | 259.50 | 260.50 | 261.00 | 261.72 |
| 1967-1969 | | | | | | | | | | | | | | | | | | | | | | |
| 1969-1975 | | | | | | | | | | | | | | | | | | | | | | |
| 1975-1978 | 0.0 | 0.0 | 0.0 | -0.1 | -0.1 | -0.2 | -0.3 | -0.2 | -0.3 | -0.2 | -0.4 | -1.1 | -0.2 | -0.5 | -0.5 | -0.4 | -0.8 | -0.4 | -0.5 | -0.5 | -0.5 | -0.6 |
| 1978-1981 | -0.2 | -0.3 | -0.3 | -0.3 | -0.2 | -0.3 | -0.4 | -0.3 | -0.3 | -0.3 | -0.3 | -0.7 | -0.2 | -0.4 | -0.7 | -0.2 | -0.6 | -0.1 | -0.4 | 0.0 | -0.1 | 0.0 |
| 1981-1986 | 1.0 | 1.0 | 0.9 | 0.9 | 0.9 | 0.9 | 0.8 | 0.8 | 0.9 | 0.8 | 0.8 | 0.7 | 1.0 | 0.9 | 0.7 | 1.0 | 0.8 | 1.1 | 1.0 | 1.0 | 1.2 | 1.1 |
| 1986-1993 | 0.0 | 0.0 | 0.0 | 0.0 | 0.0 | 0.0 | -0.1 | -0.1 | 0.0 | 0.0 | 0.0 | -0.3 | 0.0 | -0.1 | -0.3 | -0.2 | -0.2 | -0.2 | -0.2 | -0.3 | -0.2 | -0.3 |
| 1993-2000 | 0.0 | 0.0 | 0.0 | 0.0 | 0.0 | 0.0 | -0.1 | -0.1 | -0.2 | -0.1 | -0.1 | -0.2 | 0.0 | -0.2 | -0.3 | -0.2 | -0.2 | -0.1 | -0.2 | -0.2 | -0.2 | -0.1 |
| 2000-2006 | -0.5 | -0.5 | -0.4 | -0.5 | -0.5 | -0.5 | -0.6 | -0.5 | -0.7 | -0.6 | -0.5 | -0.6 | -0.4 | -0.5 | -0.7 | -0.6 | -0.6 | -0.5 | -0.6 | -0.6 | -0.5 | -0.5 |
| 2006-2013 | 0.0 | 0.0 | 0.0 | 0.0 | 0.0 | 0.0 | -0.2 | -0.1 | -0.2 | -0.2 | -0.1 | -0.1 | 0.0 | -0.1 | -0.3 | -0.2 | -0.1 | -0.1 | -0.2 | -0.2 | -0.2 | -0.1 |

| Year \ Mile | Pool 33 | | | | | | | | | | | Pool 34 | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | 262.00 | 262.50 | 263.00 | 263.50 | 264.00 | 264.50 | 265.00 | 265.50 | 266.00 | 266.50 | 266.91 | 268.00 | 268.50 | 269.30 | 270.00 | 270.50 |
| 1967-1969 | | | | | | | | | | | | | | | | |
| 1969-1975 | | | | | | | | | | | | | | | | |
| 1975-1978 | -0.4 | -0.3 | -0.3 | -0.3 | -0.7 | -0.5 | -0.5 | -0.4 | -0.4 | -0.4 | -0.5 | -0.6 | -0.6 | -0.6 | -0.7 | -0.9 |
| 1978-1981 | 0.0 | 0.1 | 0.1 | 0.0 | -0.1 | 0.0 | 0.1 | 0.1 | 0.2 | 0.3 | 0.4 | 0.5 | 0.5 | 0.6 | 0.6 | 0.5 |
| 1981-1986 | 1.2 | 1.2 | 1.2 | 1.2 | 1.0 | 1.0 | 1.1 | 1.2 | 1.3 | 1.3 | 1.3 | 1.3 | 1.3 | 1.3 | 1.4 | 1.3 |
| 1986-1993 | -0.2 | 0.0 | 0.0 | 0.0 | 0.0 | 0.1 | 0.1 | 0.0 | -0.1 | -0.1 | -0.1 | -0.1 | -0.1 | -0.1 | -0.2 | -0.3 |
| 1993-2000 | -0.2 | 0.0 | 0.0 | 0.0 | 0.0 | 0.0 | 0.0 | 0.0 | 0.0 | 0.0 | 0.0 | 0.0 | 0.0 | 0.0 | 0.0 | 0.0 |
| 2000-2006 | -0.5 | -0.4 | -0.4 | -0.4 | -0.5 | -0.4 | -0.4 | -0.4 | -0.4 | -0.4 | -0.4 | -0.4 | -0.4 | -0.4 | -0.4 | -0.4 |
| 2006-2013 | -0.1 | 0.0 | 0.0 | 0.0 | -0.1 | -0.1 | -0.1 | -0.1 | -0.1 | -0.1 | -0.1 | -0.1 | -0.2 | -0.2 | -0.2 | -0.3 |

| Year \ Mile | Pool 35 | | | | | | | | | | | | | P 36 | | | 37 | P 38 | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | 271.50 | 272.00 | 273.00 | 273.75 | 274.04 | 274.50 | 275.00 | 275.50 | 276.00 | 276.50 | 277.13 | 277.50 | 278.00 | 279.05 | 279.45 | 280.06 | 282.44 | 284.96 | 287.09 |
| 1967-1969 | | | | | | | | | | | | | | | | | | | |
| 1969-1975 | | | | | | | | | | | | | | | | | | | |
| 1975-1978 | -1.3 | -1.2 | -1.0 | | | -1.2 | -1.3 | -1.2 | -0.9 | -0.6 | -1.0 | -0.8 | -0.3 | -0.4 | -0.4 | -0.3 | -0.4 | -0.2 | -0.1 |
| 1978-1981 | 0.3 | 0.2 | 0.4 | 0.4 | 0.1 | -0.5 | -0.3 | -0.4 | -0.1 | 0.2 | -0.6 | -0.1 | 0.4 | 0.2 | 0.1 | 0.2 | 0.0 | 0.1 | 0.1 |
| 1981-1986 | 1.2 | 1.2 | 1.3 | 1.3 | 1.2 | 1.0 | 1.1 | 1.1 | 1.2 | 1.3 | 1.4 | 1.3 | 1.4 | 1.3 | 1.3 | 1.3 | 1.3 | 1.4 | 1.4 |
| 1986-1993 | -0.4 | -0.4 | -0.3 | -0.4 | -0.4 | -0.5 | -0.4 | -0.5 | -0.3 | -0.2 | -0.4 | -0.3 | -0.2 | -0.2 | -0.2 | -0.2 | -0.2 | -0.1 | 0.0 |
| 1993-2000 | 0.0 | 0.0 | 0.0 | -0.1 | -0.1 | -0.2 | -0.2 | -0.2 | -0.2 | -0.1 | -0.2 | -0.2 | -0.1 | -0.2 | -0.2 | -0.2 | -0.2 | -0.1 | -0.1 |
| 2000-2006 | -0.4 | -0.4 | -0.5 | -0.5 | -0.6 | -0.5 | -0.6 | -0.5 | -0.5 | -0.4 | -0.5 | -0.5 | -0.5 | -0.5 | -0.5 | -0.5 | -0.6 | -0.5 | -0.5 |
| 2006-2013 | -0.3 | -0.2 | -0.2 | -0.2 | -0.5 | -0.4 | -0.4 | -0.3 | -0.2 | -0.2 | -0.3 | -0.2 | -0.2 | -0.1 | -0.1 | -0.1 | -0.2 | -0.1 | 0.0 |

-.7 Subsidence Rate reported per location, per period. Data bar depicts relative magnitude of subsidence rates. Red is negative (bad).

Period During Aqueduct and Water Distribution Construction in San Joaquin Field Division.

Normal to Wet Hydrologic Years (Water Index).

Dry to Critical Hydrologic Years (Water Index).

## 6.5 Water Surface Elevation versus System Control and Data Acquisition

WSEs upstream and downstream of Checks 14 to 26 were surveyed by the Precise Surveys Unit (SLFD) in August 2015. Supervisory control and data acquisition (SCADA) elevations from the Operations Control Office were obtained for the same dates and times as the surveys to compare and contrast the SCADA elevations to surveyed elevations. Table 6-8 provides the comparison.

**Table 6-8 System Control and Data Acquisition compared to Surveyed Water Surface Elevations, August 2015**

| Check | Location | Date – Time of Day | SCADA Reading (feet) | Field Survey (feet) | Difference (feet) |
|---|---|---|---|---|---|
| DAPP | Upstream | Aug. 12, 2015 - 1242 | 222.58 | 223.45 | -0.87 |
| | Downstream | Aug. 12, 2015 - 1212 | 330.68 | 330.58 | 0.10 |
| 14 | Upstream | Aug. 10, 2015 - 1042 | 330.13 | 330.06 | 0.07 |
| (MP 95.06) | Downstream | Aug. 10, 2015 - 1107 | 328.23 | 328.18 | 0.05 |
| 15 | Upstream | Aug. 10, 2015 - 1242 | 329.10 | 328.47 | 0.63 |
| (MP 108.50) | Downstream | Aug. 10, 2015 - 1302 | 324.63 | 323.9 | 0.73 |
| 16 | Upstream | Aug. 11, 2015 - 0740 | 325.36 | 324.43 | 0.93 |
| (MP 122.07) | Downstream | Aug. 11, 2015 - 0757 | 321.62 | 320.63 | 0.99 |
| 17 | Upstream | Aug. 11, 2015 - 0857 | 322.13 | 320.55 | 1.58 |
| (MP 132.95) | Downstream | Aug. 11, 2015 - 0916 | 321.47 | 320.27 | 1.20 |
| 18 | Upstream | Aug. 11, 2015 - 1026 | 321.08 | 320.05 | 1.03 |
| (MP 143.23) | Downstream | Aug. 11, 2015 - 1043 | 318.32 | 317.32 | 1.00 |
| 19 | Upstream | Aug. 11, 2015 - 1219 | 318.66 | 317.14 | 1.52 |
| (MP 155.64) | Downstream | Aug. 11, 2015 - 1241 | 316.62 | 315.04 | 1.58 |
| 20 | Upstream | Aug. 12, 2015 - 0925 | 316.65 | 315.06 | 1.59 |
| (MP 164.69) | Downstream | Aug. 12, 2015 - 0952 | 314.84 | 313.35 | 1.49 |
| 21 | Upstream | Aug. 12, 2015 - 0752 | 313.84 | 313.5 | 0.34 |
| (MP 172.40) | Downstream | Aug. 12, 2015 - 0822 | 312.60 | 312.14 | 0.46 |
| 22 | Upstream | Aug. 17, 2015 - 1110 | 312.02 | 312.55 | -0.53 |
| (MP 184.82) | Downstream | Aug. 17, 2015 - 1120 | 306.80 | 307.29 | -0.49 |
| 23 | Upstream | Aug. 17, 2015 - 1209 | 308.43 | 307.30 | 1.13 |
| (MP 197.05) | Downstream | Aug. 17, 2015 - 1220 | 305.68 | 304.47 | 1.21 |
| 24 | Upstream | Aug. 17, 2015 - 1342 | 305.85 | 304.33 | 1.52 |
| (MP 207.94) | Downstream | Aug. 17, 2015 - 1352 | 305.64 | 304.42 | 1.22 |
| 25 | Upstream | Aug. 17, 2015 - 1410 | 304.61 | 304.18 | 0.43 |
| (MP 217.79) | Downstream | Aug. 17, 2015 - 1419 | 302.15 | 301.78 | 0.37 |
| 26 | Upstream | Aug. 18, 2015 - 0855 | 303.67 | 302.46 | 1.21 |
| (MP 224.92) | Downstream | Aug. 18, 2015 - 0904 | 300.97 | 300.90 | 0.07 |

Notes:
DAPP = Dos Amigos Power Plant, MP = Milepost, SCADA = system control and data acquisition

The difference between actual field WSEs and SCADA elevations ranged from 0.05 foot to 1.59 feet. In general, SCADA elevations were higher than survey elevations (except in Pool 22). Very little difference was seen at Pool 14, while Pools 17 to 20, 23, and 24, showed a difference of more than 1 foot. These pools are some of the areas where significant subsidence has been observed over the years.

DOE recommends that OCO review and re-evaluate SCADA values.

## 6.6 Unmanned Aerial Vehicle Synthetic Aperture Radar

DWR has been funding and working with NASA's Jet Propulsion Laboratory (JPL) to monitor subsidence in the Valley since July 2013. It uses interferometric synthetic aperture radar (InSAR) from satellites and aircraft to record the distance between the radar and the ground surface. NASA then compares multiple data sets, from different times, to produce maps (called interferograms) showing the cumulative change in surface elevation. NASA summarized two techniques and findings for two geographical areas in an August 2015 report, *Progress Report: Subsidence in the Central Valley, California.*

The first method and area covers most of the Valley and was collected using satellite data between 2006 and 2010, and a second set between May 2014 and January 2015. The results from those datasets showed two large subsidence bowls, roughly centered on El Nido in Merced County, and on Corcoran in Kings County. The 2014 to 2015 map included in the report (page 7) showed more widespread subsidence between the two towns, extending west to the Aqueduct. Based on the colored map, the subsidence varied between 0 inches and 4 inches from approximately Milepost 80 to Milepost 179, which was the southern boundary of the map. Because of the low resolution of this method, and the geographical focus being out into the Valley, that data was not used in this study.

The second method and area was focused along the Aqueduct. It was collected from an uninhabited aerial vehicle synthetic aperture radar (UAVSAR) flying two flight lines along the alignment of the Aqueduct, one in the SLFD and the other in the SJFD.

There were 11 flights along the SLFD flight line from July 2013 to March 2015 which produced 10 sets of interferograms showing the progression of subsidence. Figure 6-7 shows the estimated subsidence between July 2013 and March 2015, from O'Neill Forebay to Kettleman City, in yellows (-1.5 inches to -3.0 inches), oranges (-3.0 inches to -5.0 inches) and reds (-5.0 inches to -8.0 inches). There were seven flights along the SJFD flight line between April 2014 and January 2015, which produced six sets of interferograms. The SJFD flight line covers the area approximately between Mileposts 215 and 292. Figure 6-8 shows the estimated subsidence between April 2014 and January 2015, which show areas of green (-3 inches to -4 inches) and light blue (-4 inches to -5 inches); but, those areas are in the Buena Vista Lakebed area. All 16 interferograms produced for the NASA study have been transferred to DWR in raster format for use in this study and future studies.

**Figure 6-7 Subsidence in San Luis Field Division between July 2013 and March 2015, from UAVSAR (NASA 2015)**

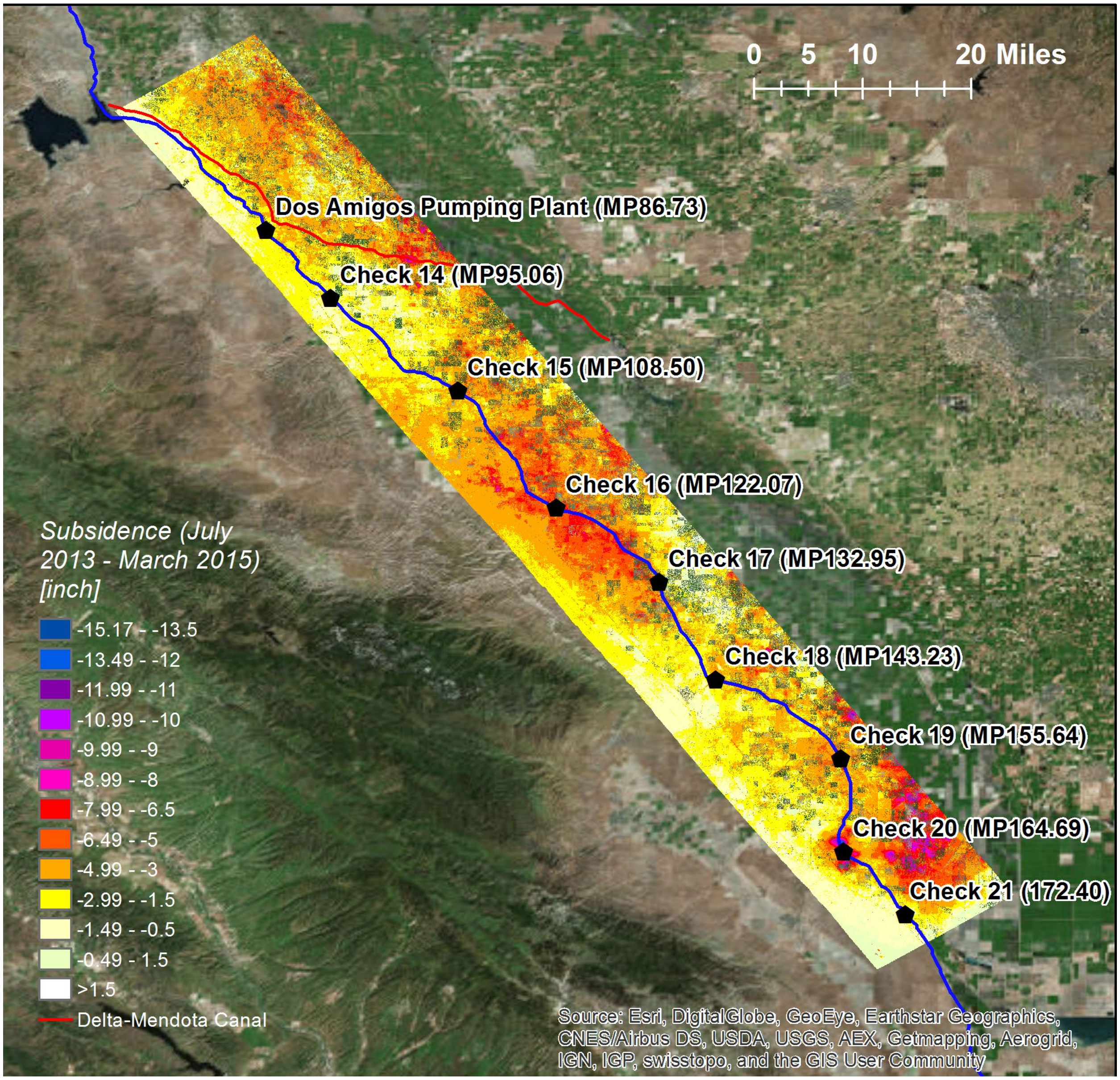
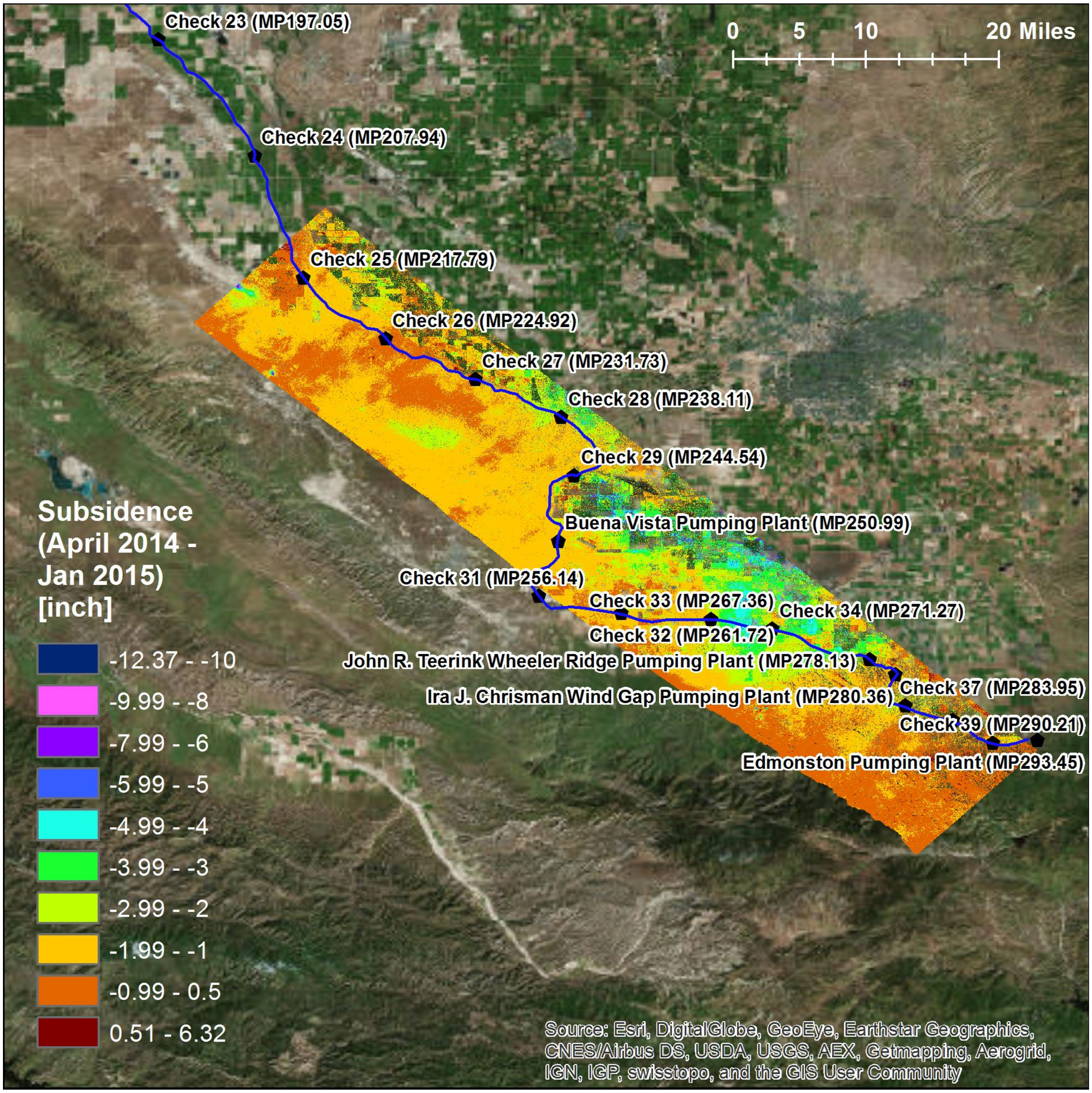


**Figure 6-8** **Subsidence in San Joaquin Field Division between April 2013 and January 2015, from UAVSAR (NASA 2015)**



### 6.6.1 Hotspot near Avenal Cutoff Road

After the creation of the first few sets of UAVSAR interferograms in the SLFD, an area was noticed near Avenal Cutoff Road, Milepost 163.69, which had significantly more subsidence than the surrounding area. It was a distinct “hotspot” bowl of subsidence centered approximately 1,500 feet east of the Aqueduct.

In the first interferogram, which quantified the subsidence between July 2013 and June 2014, the bowl was approximately 3,500 feet across and the subsidence within it ranged from -3.5 inches near the edges to -5.1 inches at the center. During that time frame, the surrounding area displayed between -1 inch and -2.5 inches of subsidence. There was also an increase of subsidence west of the bowl, which crossed the Aqueduct.

In the interferogram between July 2013 and October 2014, the bowl widened to approximately 6,000 feet across and the magnitude increased from -5.1 inches to -14 inches at the center. During the same time frame, the surrounding area experienced -2 inches to -5 inches of subsidence.

The last interferogram that was included in the UAVSAR report was between July 2013 and March 2015. Figure 6-9 shows a detailed plan view of the area of the hotspot bowl. The bowl is approximately 6,200 feet across when measured north to south and about 7,200 feet across when measured east to west. In this interferogram, the magnitude of subsidence of the bowl is -6 inches to -7 inches on its edges and as much as -14.2 inches in the center. The subsidence in the surrounding areas, which is uncolored in Figure 6-9, ranges between -2 inches and -5 inches.

To illustrate the increase in subsidence over time, a profile was created along the eastern side of the Aqueduct for each interferogram. They were combined in Figure 6-10 to show the progression of the cumulative subsidence. When the times between datasets are taken into account, the most subsidence occurred from June 2014 to August 2014 (the space between the dark green line and the light green line).

**Figure 6-9 Avenal Cutoff Subsidence Hotspot – Plan**

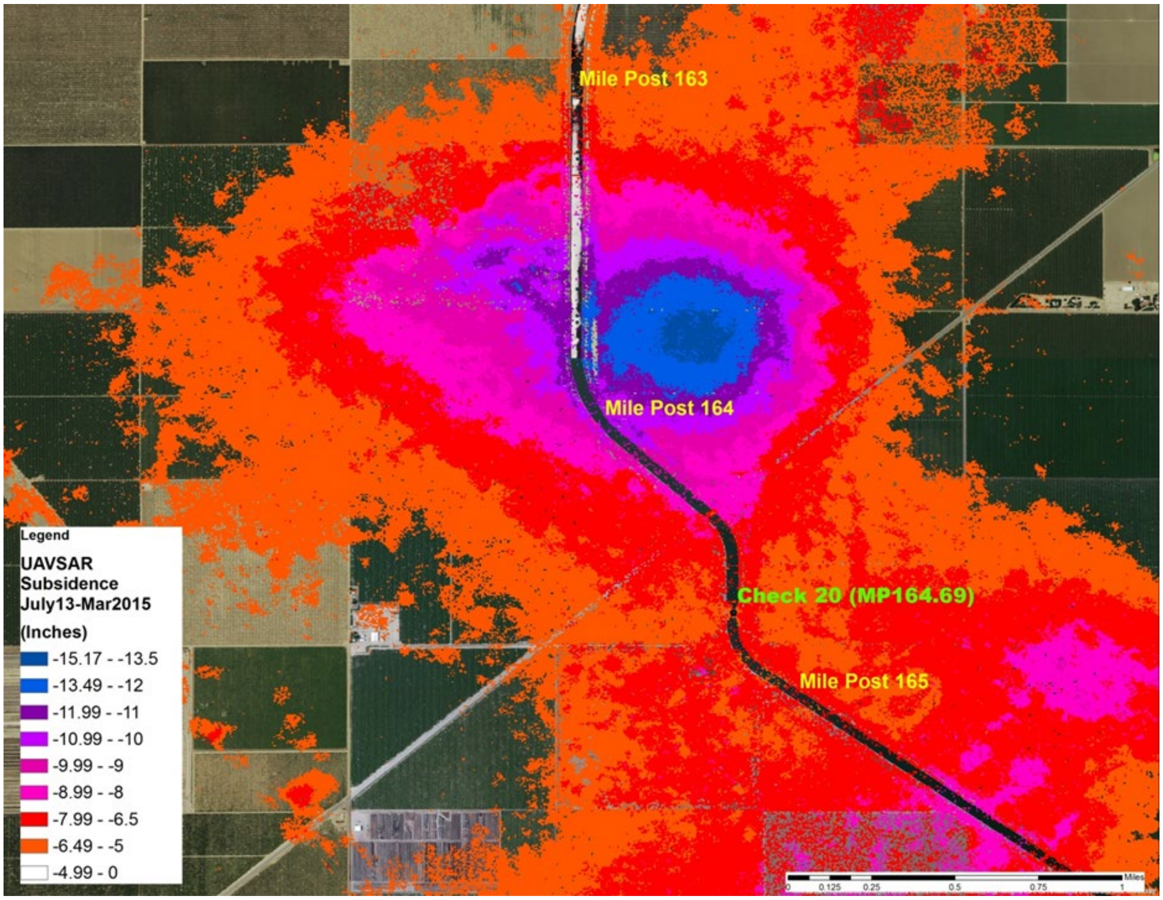
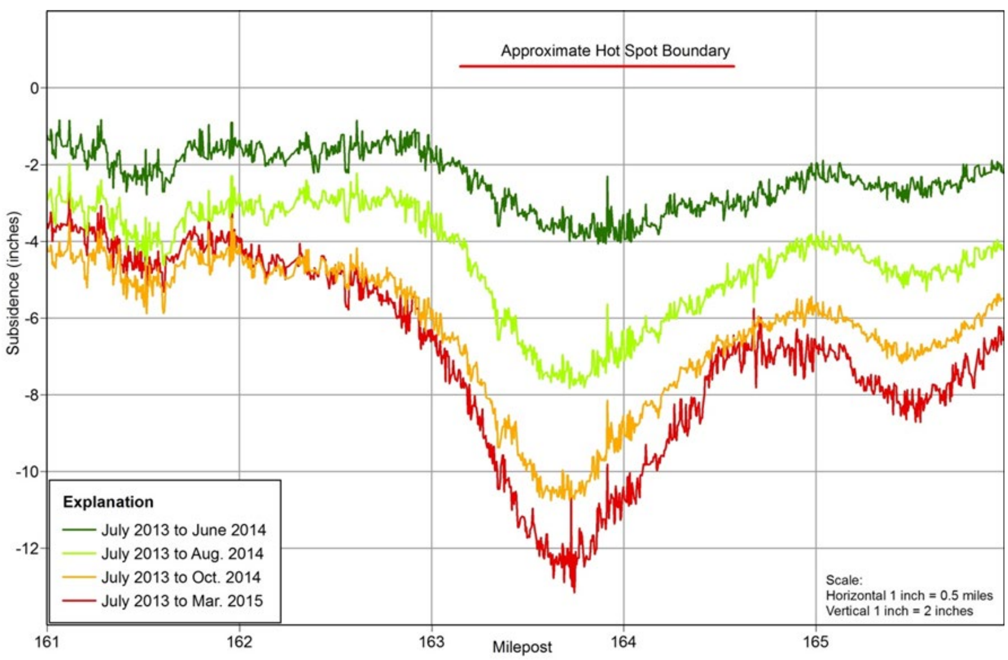



**Figure 6-10 Aqueduct Subsidence Profile at Avenal Cutoff Hotspot**



### 6.6.2 Unmanned Aerial Vehicle Synthetic Aperture Radar versus Precise Survey

Subsidence based on survey data from DWRs Precise Surveys Unit, discussed in Section 6-3, was compared to the UAVSAR data received from NASA. To do that, the raster of the UAVSAR from the SLFD from July 19, 2013, to March 10, 2015 (Figure 6-7), was plotted in a geographic information system. To be able to compare the UAVSAR subsidence "surface" to discrete survey points, a cross section of the UAVSAR data was created. To accomplish that, a section line was created along the Aqueduct alignment following the approximate crest of the Aqueduct and passed through, or close to, each survey point. That required the section line to cross the Aqueduct when survey points were on the opposite side.

Once the cross section of the UAVSAR data was created, the survey point locations and the magnitude of subsidence between the 2013 and April 2015 surveys were plotted on the cross section. Plate 30 shows the subsidence profile for the SLFD. The red continual line is the UAVSAR data and the green dots are the subsidence points from precise survey data. The most recent survey data for the SJFD were done in 2013, so no comparable data were available.

The comparison of the survey data to the UAVSAR data shows a general correspondence between the two data sets. A closer comparison of the survey point data to the UAVSAR data plotted for each survey point for the San Luis Canal allowed the data to be broken into three sections. The first section was between Pools 14 and 16, and indicates the survey data show generally less subsidence than the UAVSAR data by +0.23 inches to +3.0 inches. Figure 6-11 shows the UAVSAR and survey data comparison in Pools 14 and 15, from Plate 30.

The second section is between Pools 17 and 21. It indicates the survey data show more subsidence than the UAVSAR data by about -0.35 inch to -9.5 inches. Figure 6-12 shows the UAVSAR and survey data comparison in Pool 19 through Pool 21, from Plate 30.

The third section is in Pool 22. It indicates the survey data show less subsidence than the UAVSAR data by about +0.33 inch to +0.94 inch. Note the end of the UAVSAR flight only covers part of Pool 22.

A comparison of the survey point data to the UAVSAR data, plotted as a section line for the San Luis Canal, indicates similar results as the individual data comparison. The section line appears as a jagged line because of the density of data and elevation change differences between each cell of the UAVSAR data.

The following general observations can be made based on the UAVSAR data and the comparison with the survey data:

- The differences between the data seem to be a constant percentage between the two sets of data.
- The UAVSAR data between the survey data points show, for the most part, linear trends. There are no spikes or troughs between the data points. That indicates that a linear interpolation between the survey points is a reasonable assumption.

**Figure 6-11** **Comparison of UAVSAR and Survey Data Subsidence in Pools 14 and 15 (from Plate 30)**

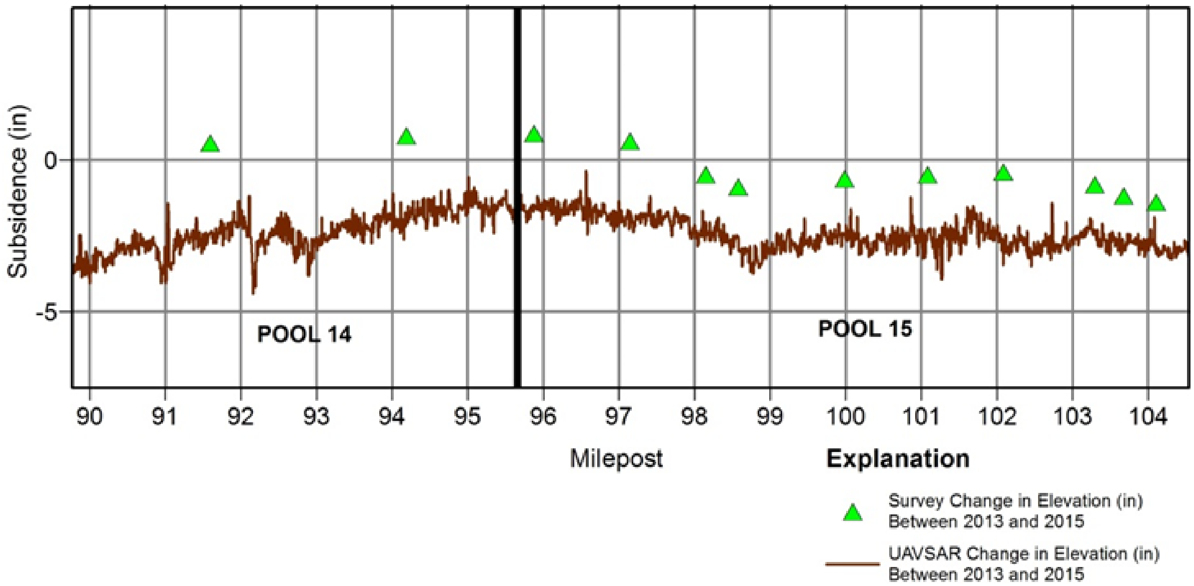
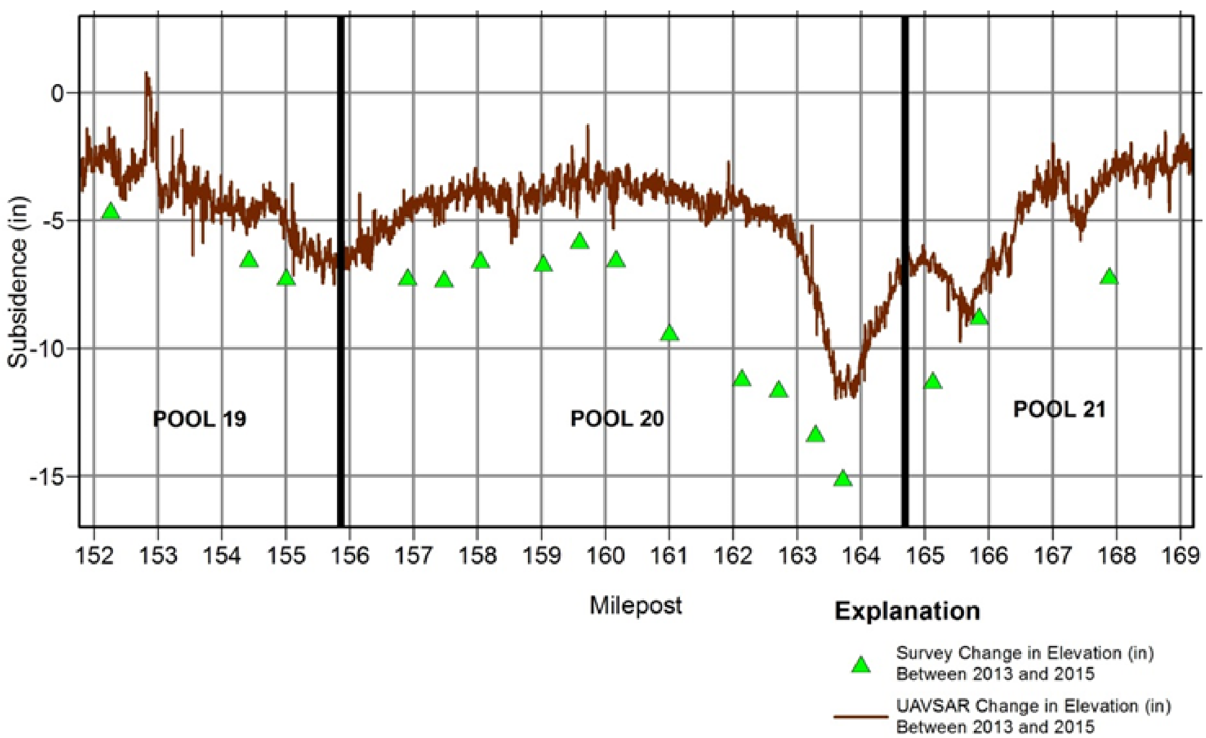



**Figure 6-12** **Comparison of UAVSAR and Survey Data Subsidence in Pools 19 through 21 (from Plate 30)**



The reasons for the differences between the survey data and UAVSAR data may be because of numerous factors:

- The cell size of the UAVSAR data averages all the elevation points in the cell. In that case the Aqueduct embankment, field, and water-elevation data may be averaged.
- The lack of UAVSAR data at some points because of data gaps or no change in the elevation because water surface elevations remained relatively constant.
- The uncertainty of the UAVSAR data ranges between 0.13 inch and 0.80 inch, which is a factor of processing and conditions during acquisition.

# Chapter 7. Operational and Structural Impacts of Subsidence

## 7.1 Current Operational Practice

The operation of gates and pumps are coordinated in a way that gravity flow is maximized and the pool-specific inflow rate is nearly equal to the outflow rate. Gate operation (at check structures) is coordinated to accelerate the water via gravity flow toward the next check structure or the next pumping plant in preparation for pump start-up. Currently, to build the necessary head, gates at Check 14 remain closed for about 45 minutes after the start of the pumps at DAPP. Then, all the gates from Check 14 through Check 29 are opened nearly simultaneously. It then takes about two hours for the water to balance out within the pools to support the pumping at BVPP. The lag time between starting pumps at DAPP and that water being delivered to BVPP is about 2 hours and 45 minutes (according to personnel from Water Operations). The near simultaneous operation of gates allows for quick response of the entire system to changes in pumping rates and minimizes fluctuations in water levels within Aqueduct pools.

The operation of the Checks upstream of the BVPP will impact the operation of downstream Checks. However, the BVPP separates the aqueduct into two hydraulic systems. Therefore changes in the water surface elevation of one system will not impact the water surface elevation of the other system.

## 7.2 Operational Impacts

Field visits and discussions with plant operators at field divisions show that subsidence has had an impact on the conveyance capacity of the Aqueduct. The following is the summary from DWR Operation Control Office's list of major operational issues related to Aqueduct subsidence.

### 7.2.1 Reduction in Conveyance Capacity

It is becoming challenging for water dispatchers and operators to move water through Pools 14 through 21 because of continuing subsidence of Aqueduct. For example, subsidence in the aqueduct has caused a decrease in the flows through Check 21. Although the design capacity of Pool 21 was 8,350 cfs, according to DWR's Operation Control Office, the estimated maximum flow in 2006 for Check 21 was 7,300 cfs. In order to deliver water to the turnouts downstream of Check 21, while still meeting pumping requirements at BVPP, Pool 22 through Pool 26 are operated at the upper elevations to maintain sufficient storage and "head" to move the water.

Recently, new turnouts have been added in Pool 23 through Pool 29. Additionally, the Cross Valley Canal Number 2 in Pool 28 has doubled in capacity from 1,000 cfs to 2,000 cfs. Installation of high capacity turnouts in this stretch further exacerbates water delivery to downstream users. The observed capacity of Pool 21, which was reported to be 7,300 cfs for 2006 level of subsidence and reduced freeboard, limits the project's ability to support the increasing turnout demand within Pools 22 through 29, and the pumping rate of 4,000 cfs at BVPP.

### 7.2.2 Submergence of Overchute

The total subsidence of the overchute at Milepost 208.11 is 2 feet greater than the total subsidence at Check 25. Because of the difference in subsidence, water flowing in Pool 25 touches the lower portion of the overchute within the normal operating range. The submergence of the overchute increases with an increase in flow delivery. Once the water surface touches the bottom of the overchute, additional head is needed for the flow to pass through it. The need for additional head increases the WSE on the upstream end of the overchute, thus raising the water level in the first 800 feet of the pool.

### 7.2.3 Increase in Power Cost

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 canal storage upstream of Check 21 is well suited for this operation. But, Aqueduct subsidence at several pools (for example Pools 14, 17, 18, 20, and 21) has reduced the available freeboard, forcing the Aqueduct to be operated at a lower WSE (Section 5.3), which reduces the overall Aqueduct storage. The reduced storage forces more pumping during more expensive periods to meet downstream demand.

In addition to higher energy costs, pumping during high-demand periods on the California Independent System Operator (CAISO) grid is likely to subject DWR to higher resource adequacy requirements. Resource adequacy is a mandatory CAISO reliability process to ensure adequate electric grid resources to serve all loads in real-time. Resource adequacy requirements have required the State Water Project to be operated at a lower load during the core resource adequacy hours from 6 p.m. to 10 p.m. In addition, more pumping during on-peak hours could obligate DWR to have higher flexible capacity requirements to address generation oversupply and ramps under the CAISO's Flexible Resource Adequacy Criteria and Must Offer Obligation.

## 7.3 Decrease in Freeboard

When comparing design WSEs to 2015 top of lining survey data for the SLFD, from Milepost 86.94 to 172.4 (Pools 14 to 21), 21.3 miles of canal was calculated to have less than 3 feet of freeboard (minimum designed lined freeboard) because of subsidence. Of the 21.3 miles, 13.8 miles have 2 feet to 3 feet of freeboard, 6.8 miles have 1 foot to 2 feet, 0.6 mile has 0 feet to 1 foot, and 0.03 mile has less than 0 feet of freeboard. Table 7-1 has a summary of remaining freeboard.

**Table 7-1 Available Freeboard with Respect to Design Water Surface Elevation**

| | San Luis FD Milepost 86.94-172.4 | | | | | San Joaquin FD Milepost 172.4-293.45 | | | | |
|---|---|---|---|---|---|---|---|---|---|---|
| Available Freeboard (feet) | >3 | 2-3 | 1-2 | 0-1 | <0 | >2.5 | 1.5-2.5 | 0.5-1.5 | 0-0.5 | <0 |
| Aqueduct Length (miles) | 64.3 | 13.8 | 6.8 | 0.6 | 0.03 | 106.2 | 8.0 | 6.8 | 0 | 0 |

Note:
FD = field division
Design water surface elevations were used to calculate remaining freeboard, and were compared against recent top of lining survey data taken in 2013 (San Joaquin Field Division) and 2015 (San Luis Field Division).

When comparing design WSEs to 2013 top of lining survey data for the SJFD, from Milepost 172.4 to 293.45 (Pools 22 to 40), 14.8 miles of canal were calculated to have less than 2.5 feet of freeboard (minimum designed concrete freeboard) because of subsidence. Of the 14.8 miles, 8 miles of canal have

1.5 feet to 2.5 feet of freeboard, and 6.8 miles have 0.5 foot to 1.5 feet. Table 7-1 has a summary of remaining freeboard.

## 7.4 Impacts to Pool Water Surface Elevation

### 7.4.1 Pool Elevation Control

The flow rates and WSEs of the Aqueduct pools are controlled at each pool's downstream check structure. A level sensor upstream of that check structure is used to monitor the pool's water surface level.

Along the Aqueduct, subsidence has caused a reduction of the design freeboard, but the amount of subsidence is not uniform. As shown on Plates 1 and 10 in Chapter 6, the subsidence varies along the Aqueduct alignment. In Pools 18 and 25, the upstream ends of the pools have subsided significantly more than the downstream ends. Past canal raises in the SLFD have left Pool 18 with adequate freeboard to be able to accommodate the variation in subsidence. But on the upstream end of Pool 25, there are approximately 9 inches to12 inches of lined freeboard above the design WSE, for approximately 2 miles, as shown in Figure 7-1.

**Figure 7-1 Pool 25 Operational Profile**

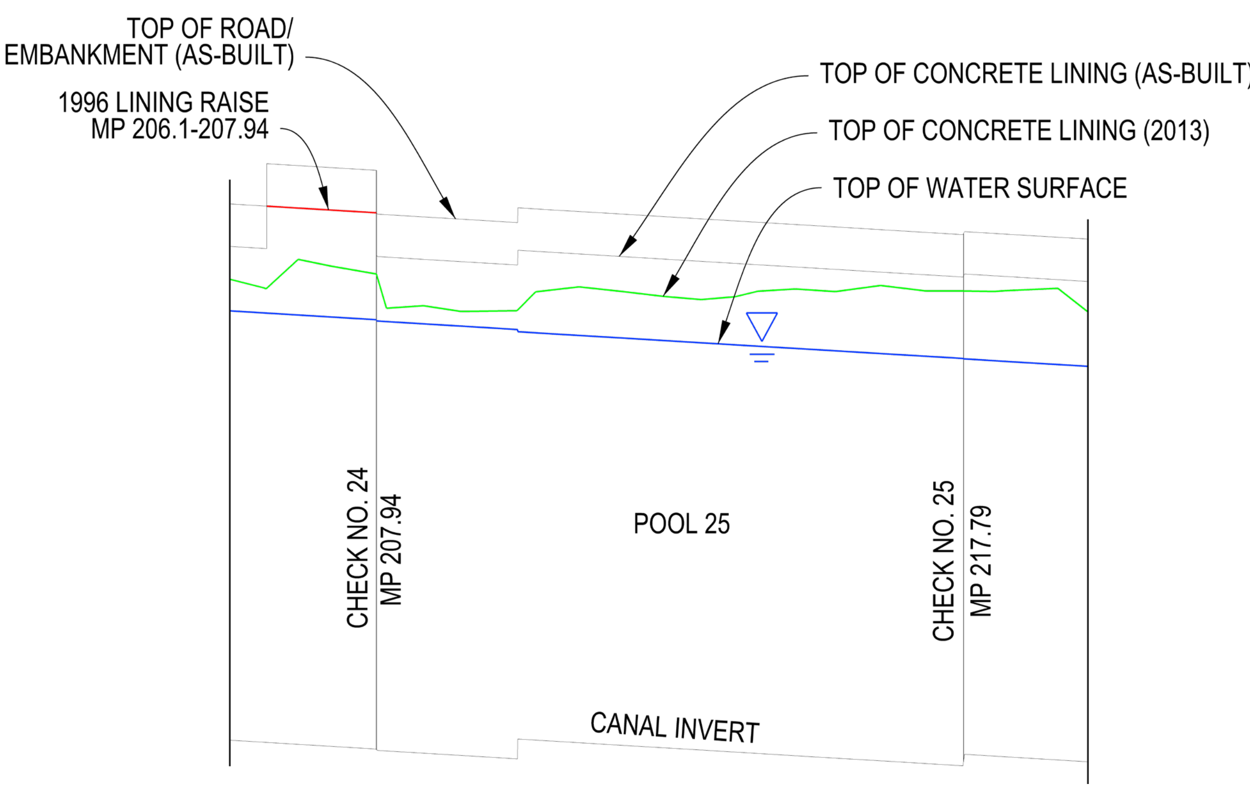
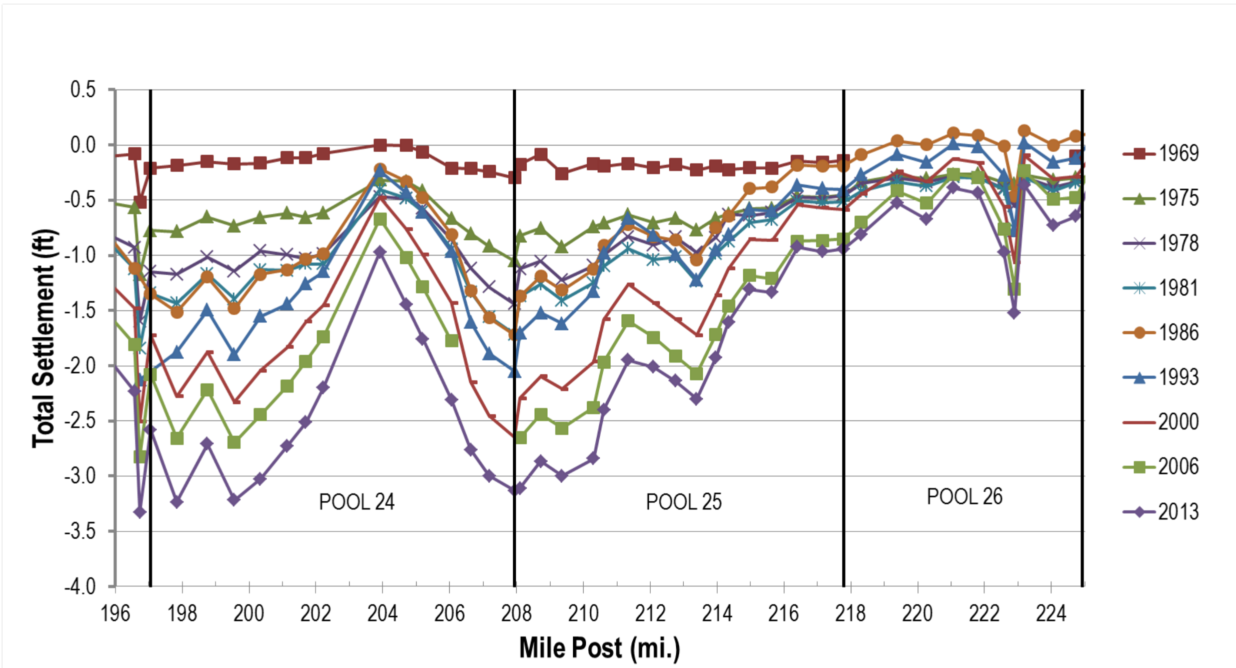


Figure 7-2 shows the magnitude of subsidence in Pools 24 through 26 in the SJFD, by year. Figure 7-2 shows that in 2013, the subsidence at Check 24 (between Pools 24 and 25) was approximately 3.1 feet, while Check 25 subsided 0.94 foot. As of 2013, the upstream side of Pool 25 subsided 2.2 feet more than the downstream end, where the controlling level sensors are located.

**Figure 7-2 Subsidence Profile (Pools 24 to 26)**




The design slope of the Aqueduct for Pool 25 is 0.00004, which is approximately 2.08 feet over the course of the 9.85-mile long pool. That slope should also be the slope of the water surface when the Aqueduct is flowing at design capacity. When Aqueduct flows are less than the design capacity, the slope of the water surface will be flatter. Figure 7-1 shows the profile of Pool 25 with water surface (at design capacity slope) in blue, and the top of the concrete lining as it was surveyed in 2013, in green. For Pool 25, the top of liner at the upstream end is lower than the downstream end.

The magnitude of the subsidence shown in Figure 7-2 in Pool 25 compares well with the difference between the top of concrete lining (as built), and the top of concrete lining (2013) as shown in Figure 7-1. The subsidence (or difference) is greater on the upstream end of Pool 25, and decreases as it goes downstream to the end of the pool.

Under normal Aqueduct operation, the use of a pool's downstream level sensor, at the check structure, is an appropriate measure of the flow conditions of a pool. At design flowrates, the amount of unlined freeboard would be constant. Under flowrates less than designed, the unlined freeboard would increase going upstream in the pool. With larger subsidence magnitudes on the upstream ends of Pools 18 and 25, the downstream level sensors do not represent the conditions of those pools.

### 7.4.2 Embankment Breach

On the afternoon of June 20, 2011, the eastern access road at Milepost 208.08, in Pool 25, was breached by sustained high water above the top of concrete liner. Figure 7-3 is a photograph of the breached section looking downstream. It shows the saturated unlined freeboard approximately 8 inches to 10 inches above the concrete liner and the 5- foot to 25-foot wide collapse of the access road. The sustained high water had seeped through the embankment, which escalated into a piping failure of the embankment and collapse of the access road. At that location, the canal embankment is approximately 6 feet to 8 feet higher than the surrounding grade.

**Figure 7-3 Milepost 208.08 Embankment Failure, June 2011**

Figure 7-4 is a photograph of the access road failure at Milepost 208.08. Shortly after the breach was reported, the water surface was dropped in Pool 25 and the flow ceased. Figure7-4 also shows very little lined freeboard and little to no gap between the water surface and the overchute that is just downstream at Milepost 208.11.

**Figure 7-4 Milepost 208.08 Embankment Failure**

### 7.4.3 Embankment Breach Investigation

The DWR Division of Operations and Maintenance (O&M) conducted a root cause investigation of the Pool 25 washout event. Root causes for the failure were attributed to decrease freeboard in some pools, equipment (or sensor) difficulties, rodent holes, regulatory constraints, trainings, communication, and management systems. While not all of the factors of this failure are attributable to subsidence, two of them are. The subsidence in Pool 25, which is higher on the upstream end and lower on the downstream end, made the downstream level sensor to be a poor indicator of the conditions in the pool. The normal operational maximum WSE in Pool 25 is 2.1 feet above the design water surface elevation (the blue line in Figure 7-1). But, the top of the concrete liner on the upstream end is only 8 inches to 10 inches above the design WSE.

The second factor of subsidence that contributed to this failure is the lack of an air gap under the overchute just downstream that can be seen in Figure 7-3 and Figure 7-4. The subsidence in the pool has caused the overchute to sink, while the water surface did not. The lack of an air gap increases the water friction under the overchute, which causes the water to back up on the upstream side of the overchute. The increases in WSE will extend a significant distance upstream, and if the Aqueduct is flowing near design capacity, the increase will extend all the way to the next upstream check structure.

## 7.5 Impacts to Turnout Facilities and Other Structures

There are approximately 755 structures along the canal in the SLFD from Milepost 70.79 to Milepost 172.40 (Pools13 to 21) and 575 structures along the canal in the SJFD from Milepost 172.40 to Milepost 294.93 (Pools 22 to 40). The structures include siphons, inlet structures, check structures, turnouts, turn-ins, bridges, pipeline crossings (gas lines, oil lines, irrigation lines, domestic water lines, and communication lines), culverts, drain inlets, manhole installations, sump pumps, recorder stations, berm drains, power line crossings, pump pads, pump backs, and canal drains, all of which cross the canal, or are adjacent to it. Table 7-2 has a summary of structures. The list of structures was compiled using three of O&M's references, (1) strip maps that were revised in September 2012, (2) Water Operations Manuals OP-350R and OP-450R, and (3) a structures list for the San Luis Canal that was done in June 1979.

Many of the structures are impacted because of subsidence. Of the 755 structures located in the SLFD, approximately 342 structures are located in areas that have undergone at least 2 feet of subsidence. The areas with at least 2 feet of subsidence are Mileposts 102.20, 106.38 to 107.88, 110.10 to 137.32, and 144.88 to 165.03. Of the 575 structures located in SJFD, approximately 43 structures are impacted in areas that have undergone at least 2 feet of subsidence. The areas with at least 2 feet of subsidence are Mileposts 196.57 to 202.22, 206.06 to 210.61, and 212.10 to 213.39.

DOE met with SLFD on July 8 and 9, 2015, and with SJFD on July 29 and 30, 2015, as part of the *California Aqueduct Subsidence Study*. The goal behind those visits was to meet with personnel from each division and discuss operational problems and issues that each division deals with on a regular basis. It also gave DOE the opportunity to field verify subsidence trends at critical locations and make sure precise survey data matched field conditions. Aside from visiting the area control center at both field divisions (and their respective field offices), DOE visited Pools 17 through 21, and Pools 23 through 26. Among the many topics discussed, and later assessed in the field, were loss of capacity, problems at Checks 17 and 24, liner freeboard deficiencies, and overchute problems (all discussed further in the following paragraphs). Appendix E has a complete set of notes summarizing both trips.

**Table 7-2 Structures in the Aqueduct Alignment**

| Structure Type | San Luis Field Division | | San Joaquin Field Division | |
|---|---|---|---|---|
| | Total Number | Total Affected[a] | Total Number | Total Affected[b] |
| Berm Drains | 6 | - | - | - |
| Bridges | 52 | 23 | 59 | 3 |
| Canal Drains | 1 | - | 1 | - |
| Canal Inlet Structure | 1 | - | - | - |
| Check Structures | 8 | 4 | 15 | 2 |
| Culverts | 5 | 1 | 26 | 1 |
| Drain Inlets | 68 | 11 | - | - |
| Manhole Installations | 20 | - | - | - |
| Overchutes | - | - | 80 | 10 |
| Overflow Weir | - | - | 1 | - |
| Permanent Turnouts | 94 | 52 | - | - |
| Phone Lines | - | - | 23 | 1 |
| Pipeline Crossings | 157[c] | 66[d] | 135[e] | 5[f] |
| Powerline Crossings | 115 | 57 | 86 | 6 |
| Pump Backs | 2 | 2 | - | - |
| Pump Pads | 36 | 26 | - | - |
| Recorder Stations | 18 | 8 | - | - |
| Sand Traps | - | - | 4 | - |
| Siphons | 1 | - | 12 | - |
| Stilling Wells | - | - | 30 | 4 |
| Sump Pumps | 14 | - | 41 | - |
| Temporary Turnouts | 74 | 33 | - | - |
| Turnins | 83 | 59 | 4 | 2 |
| Turnouts | - | - | 58 | 9 |
| **Total** | **755** | **342** | **575** | **43** |

Notes:
[a]Greater than 2 feet of subsidence at Mileposts 102.20, 106.38 to107.88, 110.10 to 137.32, and 144.88 to 165.03.
[b]Greater than 2 feet of subsidence at Mileposts 196.57 to 202.22, 206.06 to 210.61, and 212.10 to 213.39.
[c]Includes gas lines, oil lines, irrigation lines, domestic water lines, and communication lines. Lines are above and below ground.
[d]Includes gas lines, oil lines, irrigation lines, domestic water lines, and communication lines. Forty-four lines are underground, 23 are above ground.
[e]Pipeline crossings are above and below ground.
[f]Pipeline crossings are above ground.

### 7.5.1 Submergence of Trunnions

While visiting Checks 17 and 24, it became apparent that both of the trunnions are very close to the water surface and their gates are always operated out of the water. Figure 7-5 provides a look at Check 20 and its submerged trunnions. Figure 7-6 provides a view of Check 24 with its gates out of the water.

### 7.5.2 Liner Freeboard Deficiencies

While driving from Check 20 toward Milepost 160, it became apparent that the distance between the WSE and the top of the liner was getting smaller. Normally (at design flows), the WSE and the top of the liner should be equidistant. At 1,000 cfs (flow during site visit), the distance between the WSE and the top of the liner should get larger, not smaller. This observation was consistent with the Precise Survey unit's data that showed a large drop (about 6 feet of subsidence) at Milepost 160.45. Figure 7-7 provides a view of Milepost 160.45 (right side). It shows the height of the WSE while flowing 1,000 cfs (maximum design flow in this pool is 8,350 cfs). Very little freeboard is visible between the WSE and the top of liner.

**Figure 7-5 Check 20, Trunnions in the Water**

**Figure 7-6 Check 24, Gates Operated out of the Water**

**Figure 7-7 Turnout at Milepost 160.45**

# Chapter 8. Quantification of Hydraulic Impact

## 8.1 Hydraulic Model

In order to quantify hydraulic impacts (flow capacity and lined freeboard) of subsidence, a hydraulic model was developed by DOE using 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, check structures, siphons, canal transitions, turnout locations, and pumping plants located within the study reach.

The Aqueduct geometry, configurations of the check structures, and turnout locations were taken from as-built drawings. The original canal cross sections were modified using the subsidence data obtained from the most current field surveys conducted in 2015 from DAPP to Kettleman City, and in 2013 from Kettleman City to BVPP.

The Aqueduct was originally sized for a Manning's coefficient, n-value, of 0.016, bed slope of 0.00004, and a minimum lined freeboard of 3 feet for the San Luis Canal and 2.5 feet for the San Joaquin section of the Aqueduct. Recommended Manning's n-values for open channel flow are given in Table 8-1 (Phillips, J.V. and Tadayon, S. 2006). The concrete panels that line the Aqueduct are settling at different rates and each panel is settling in different directions; the non-uniform settlement of adjacent concrete panels has increased the channel roughness. In addition, the Aqueduct liner inspection survey conducted by DWR in 2014 reported the presence of debris (including cars and pickup trucks), sediment deposits on the channel bed, weed growth, cracks and holes in the liner panels, and uneven movements of adjacent concrete panels (California Department of Water Resources 2014). All of those factors increased the channel's n-value.

Table 8-1 shows the Manning's n-value is dependent upon the canal type and its condition. Considering the current condition of the Aqueduct (California Department of Water Resources 2014) and after years of operation, the n-value has increased over its design value. But, there is not enough flow data to estimate the value of Manning's coefficient, n. For the purpose of modeling, two models were developed varying Manning's roughness coefficient at values of 0.016 (which is the design n-value) and 0.020 (which is about 25 percent increase over the design n-value). The Manning's roughness coefficient of 0.020 was selected to model the current conditions of the Aqueduct. That value was selected to be more consistent with the sediment and the debris that is covering the canal invert, and the uneven liner panels that have been observed in underwater dive photos and videos.

For both models, the upstream boundary is the outflow from the DAPP and the downstream boundary is the minimum operating WSE at the BVPP forebay. As specified in Standing Operating Order 600.22, the minimum WSE at the BVPP forebay is 294.6 feet. The inflow into each pool was set to maximum design capacity or the current maximum capacity. It was assumed that the excess flow capacity between two adjacent pools would be diverted into turnout structures.

**Table 8-1 Composite Values of Manning's Coefficient, n, for Channels**

| Type of Channel | Minimum N-Value | Normal N-Value | Maximum N-Value |
|---|---|---|---|
| Concrete Lined Channels, Finished | 0.011 | 0.015 | 0.016 |
| Concrete Lined Channels, Unfinished | 0.014 | 0.017 | 0.020 |
| Earth Bottom, Straight, and Uniform Channel (Clean, after weathering) | 0.018 | 0.022 | 0.025 |

Each model was run twice. The first model run, or the base case, was made assuming the design inflow at each pool. The HEC-RAS model was run to determine the WSE. The WSEs were compared with most current top of liner survey elevations to determine the available lined freeboard along the Aqueduct profile. The second run consisted of multiple trials and errors in order to determine the maximum pool capacity for a pre-specified minimum freeboard. As in the base case, the HEC-RAS model was run for design upstream flow (inflow) and downstream boundary (minimum pool elevation at BVPP forebay) conditions and the freeboard along the Aqueduct profile was examined. If the freeboard at any pool was found to be low (less than some minimum specified level), the inflow into the pool was reduced and the model was re-run. The trial was continued until the lined freeboard at each pool was within the specified limit. The discharge obtained from the model, for each pool, was then the estimated pool capacity.

## 8.2 Results

Figure 8-1 shows the modeled freeboard along the Aqueduct alignment under the assumption that the channel roughness, n, equals the original design value of 0.016, and each pool is carrying the design flow. The geometry used in the model corresponds to the subsided geometry of the Aqueduct. It is seen that at multiple pools, the lined freeboard (red line) is lower than the designed minimum freeboard (blue line). Table 8-2 shows the design freeboard of the Aqueduct has been reduced for a length of about 12.7 miles. The lowest freeboard is 0.9 foot in Pool 20, and near Milepost 160.45, extending about 750 feet in length.

Given the decades of canal operation with accumulation of sediments and debris in the canal bed, along with movement of panels and accumulation of algae, the channel roughness, or the Manning's coefficient, has likely increased. An increase in the channel roughness decreases the canal conveyance capacity. The results, as shown in Figure 8-1, represent the upper limit of the conveyance capacity.

In the second run, and in order to assess the impacts of subsidence on freeboard, the HEC-RAS model was rerun with a revised freeboard requirement. The HEC-RAS model was run multiple times by changing the inflow into a pool so that the freeboard at any point within a pool is at least 1.5 feet. For all trial runs, and in order to maximize the flow, the check structures gates were kept open. The modeled freeboards along the Aqueduct profile are shown in Figure 8-2. Pools 17, 18, 20, 24, and 25 are some of the pools with reduced freeboard. The original and modeled capacities of the pools are shown in Figure 8-3. In order to maintain a minimum freeboard of 1.5 feet, the maximum flow at Pool 20 had to be limited to approximately 7,600 cfs. This relates to about a 9 percent reduction from the original capacity of 8,350 cfs. As shown in Table 5-1, the original design capacities of Pools 21 and 22 were 8,350 cfs and 8,100 cfs, respectively. The reduction in the Pool 20 flow capacity also reduces the capacities of Pools 21 and 22. As a result, Pool 20 acts like a choke point for pools downstream.

**Figure 8-1** **Lined Freeboard with Original n-value (0.0160) and Design Flow Capacity**

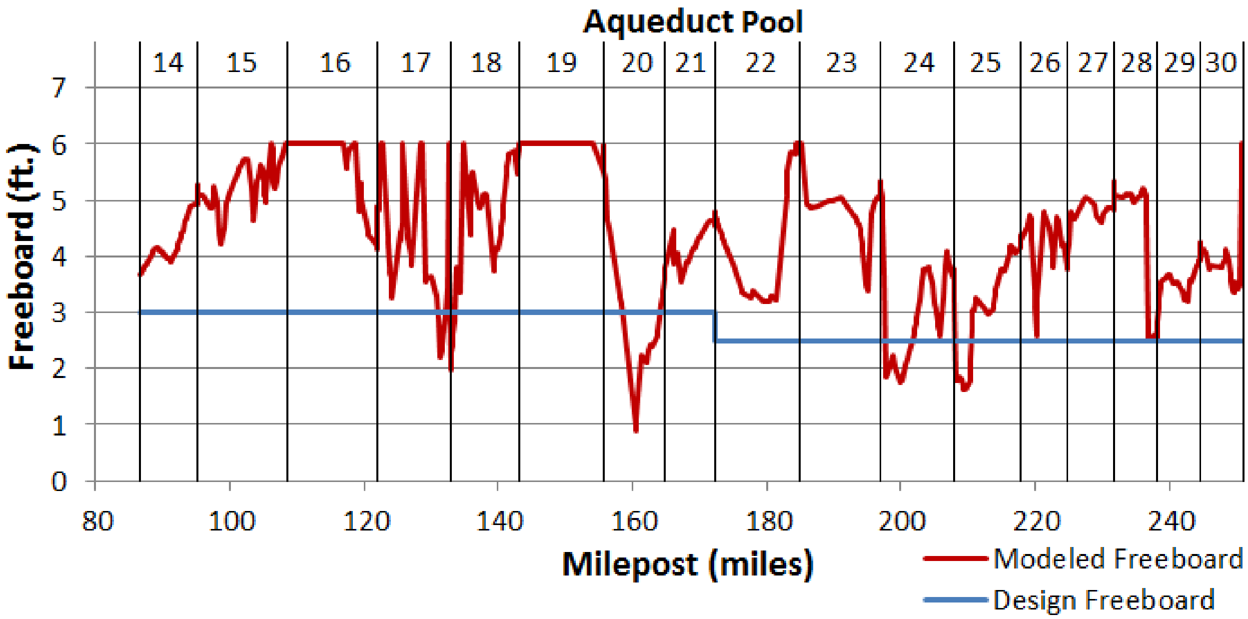



**Figure 8-2** **Lined Freeboard with Original n-value (0.016) and Reduced Flow Capacity**

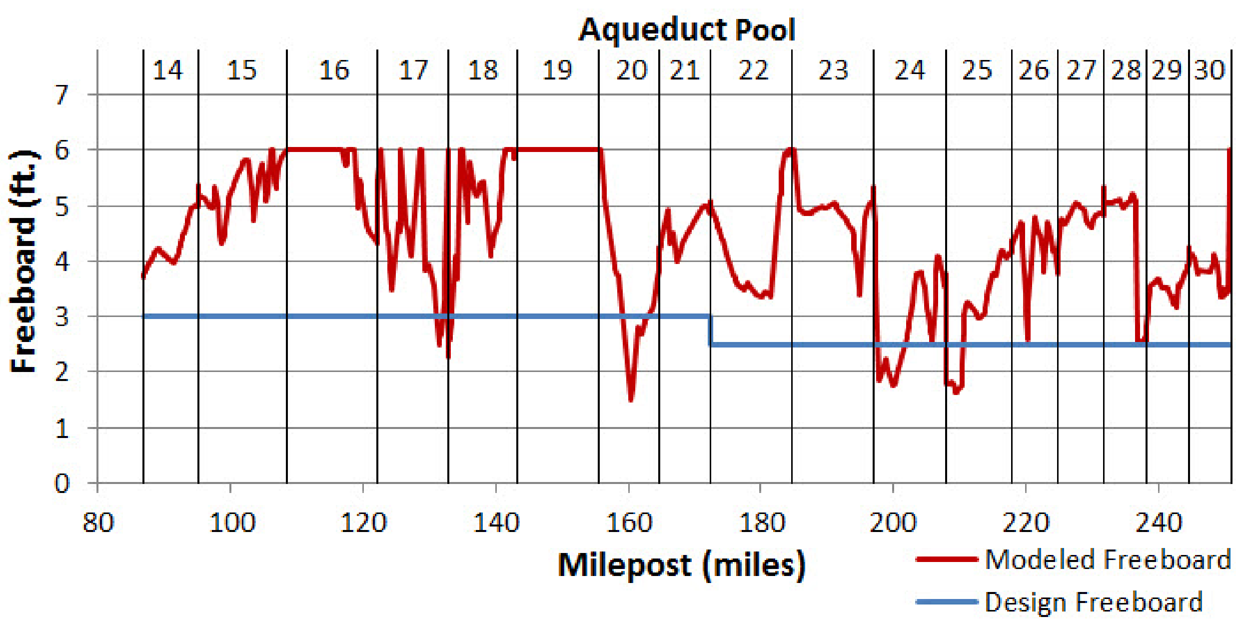


**Figure 8-3 Original Design and Reduced Aqueduct Flow Capacity (n=0.016)**

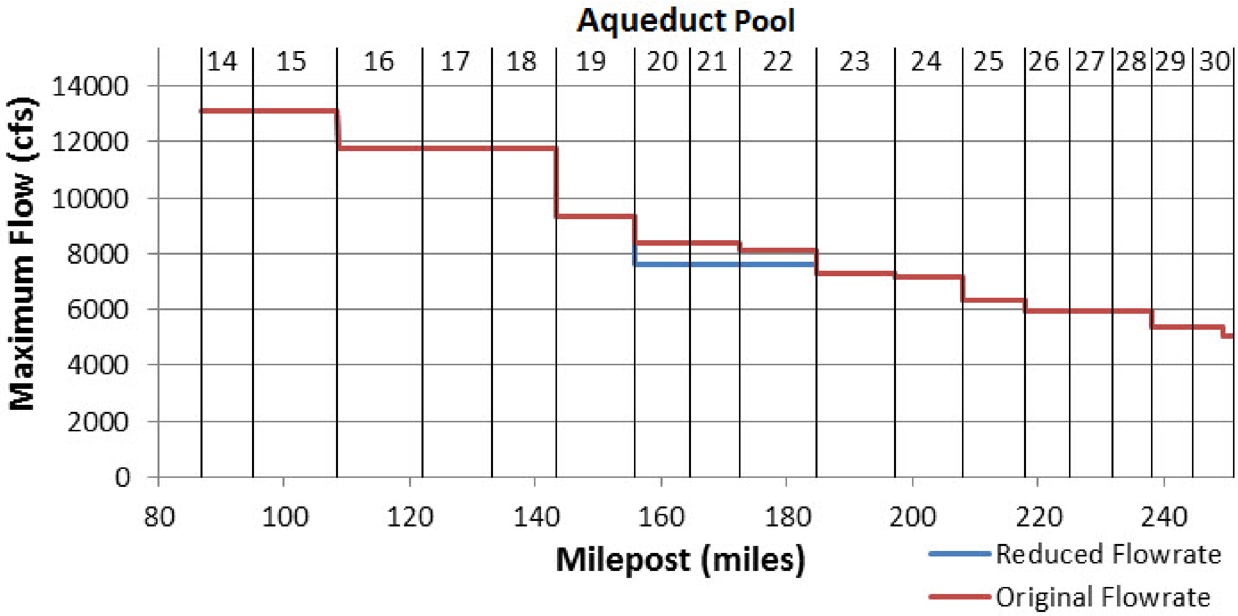



**Table 8-2 Lengths of Less than Design Freeboard**

| Freeboard Category | Impacted Length of Aqueduct n=0.016 | Impacted Length of Aqueduct n=0.020 |
|---|---|---|
| Freeboard Less than the Design Value (3 or 2.5 feet) | 12.68 miles | 95.84 miles |
| Freeboard Less than 2 feet | 5.36 miles | 68.58 miles |
| Freeboard Less than 1 foot | 0.14 miles | 28.33 miles |
| Freeboard Less than 0 foot | 0 miles | 9.54 miles |

To model the current conditions, the HEC-RAS model was modified by setting the Manning's n-value to 0.020. The upstream boundary condition corresponds to the design flow capacity and the downstream boundary condition is the minimum WSE at BVPP forebay pool. The lined freeboard along the Aqueduct profile is shown in Figure 8-4. Model results show that, with the increased bed roughness, the liner is overtopped at multiple locations. As shown in Table 8-2, out of the 164.2 mile long Aqueduct section that was considered in the model, the freeboard has been reduced below designed minimum of 3feet for about 96 miles. In total, the liner is overtopped for about a 9.5 mile stretch of Aqueduct. At Pool 20 and near Milepost 160.45, which experienced the greatest subsidence, the liner is over topped by 1.6 feet. The liner is also overtopped at Pools 17, 18, 20, 24, and 25. Table 8-2 shows the length of Aqueduct that has less than the design freeboard height, for both ideal conditions (n=0.016) and the estimated current conditions (n=0.020).

A model run was developed to estimate flows that would result in a minimum of 0.5 foot of freeboard given a Manning's n of 0.020. The minimum freeboard of 0.5 foot represents average of the design freeboards for the check structures. The check structures were designed for 0.75 foot of freeboard during ponding and 0.3 foot of freeboard when the water is flowing. As in the previous case, the HEC-RAS model was run multiple times, using trial error, by changing the inflow into a pool so that the freeboard at any point within a pool is about 0.5 foot. In the run, the flow rate within a pool was kept constant. Figure 8-5 shows the freeboard along the aqueduct section. The minimum freeboard is seen near Mile Post 160.42 (Pool 20), which has experienced the greatest subsidence. At multiple locations in Pools 17, 18, 24, and 25, the freeboard is less than 1 foot. Figure 8-6 shows the design flow rate and the maximum flow rate that result in 0.5 foot freeboard. The figure allows a comparison to see the reduction flow rate caused by subsidence in each pool. As shown in Figure 8-7, the flow capacity has been reduced at multiple pools. For Pool 20, which has the most subsidence, the maximum capacity is 6,650 cfs, a reduction of 1,700 cfs, or about 20 percent from the original capacity.

**Figure 8-4 Lined Freeboard with Current n-value (0.020) and Design Flow Capacity**

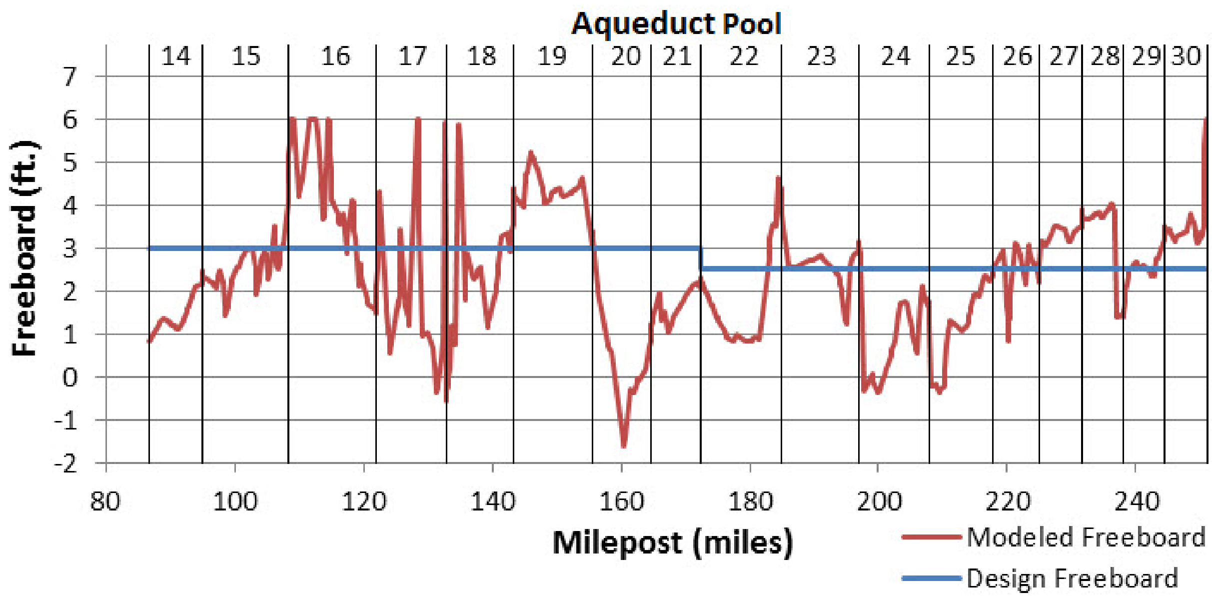



**Figure 8-5 Lined Freeboard with Current n-value (0.020) and Reduced Flow Capacity**

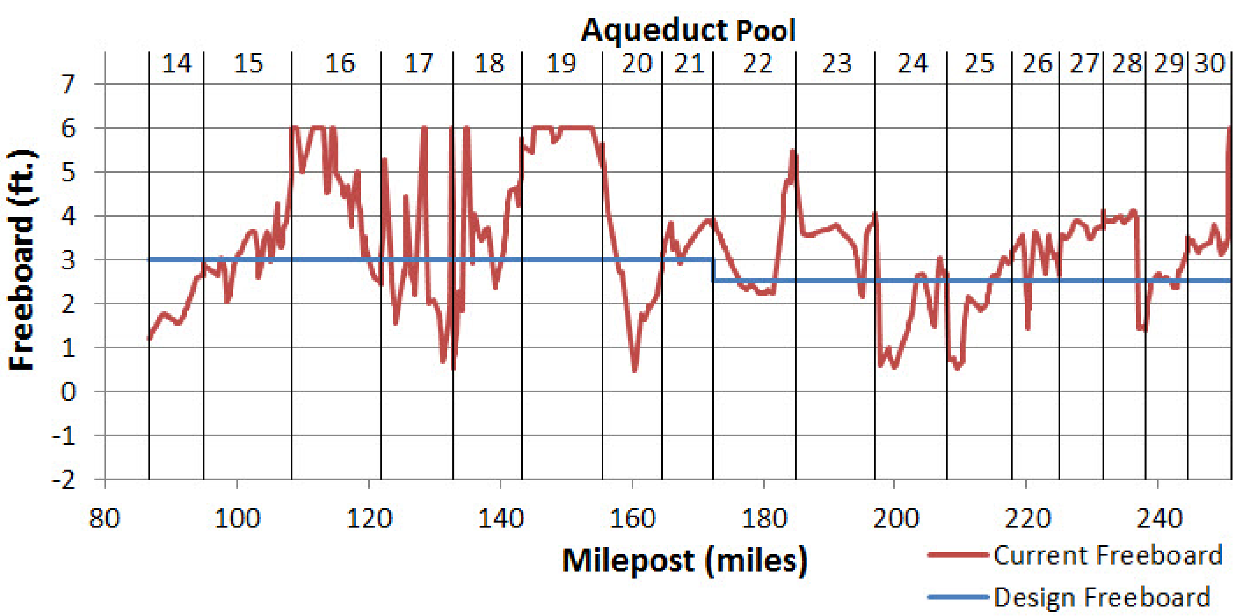


**Figure 8-6** **Original Design and Reduced Aqueduct Flow Capacity (n=0.020)**

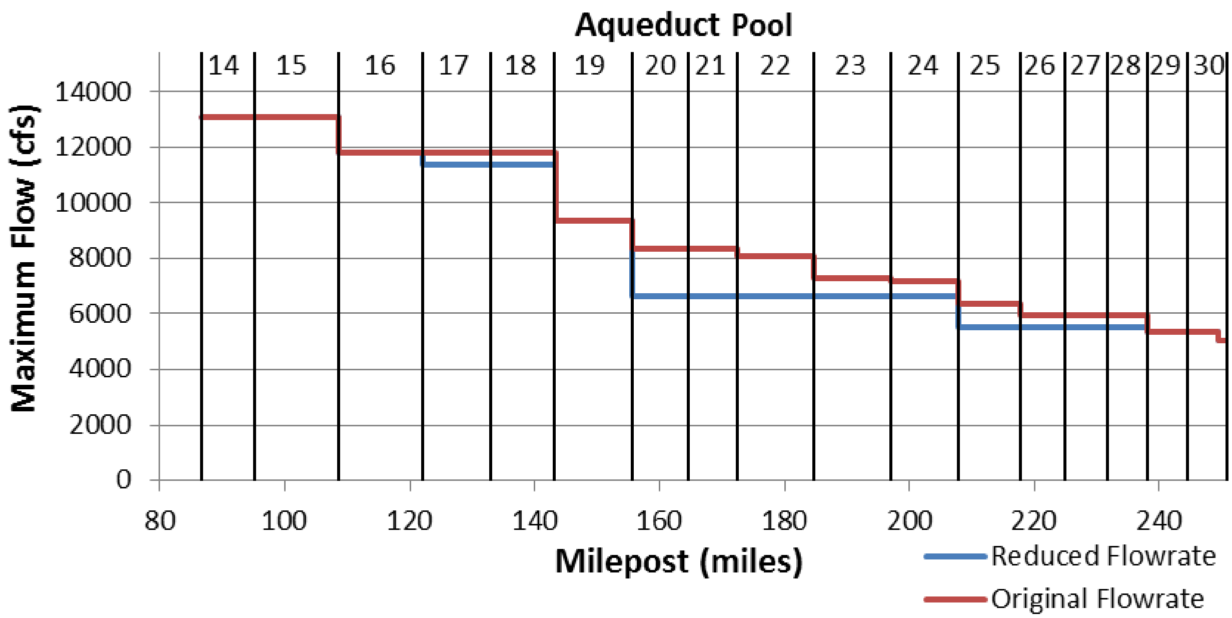
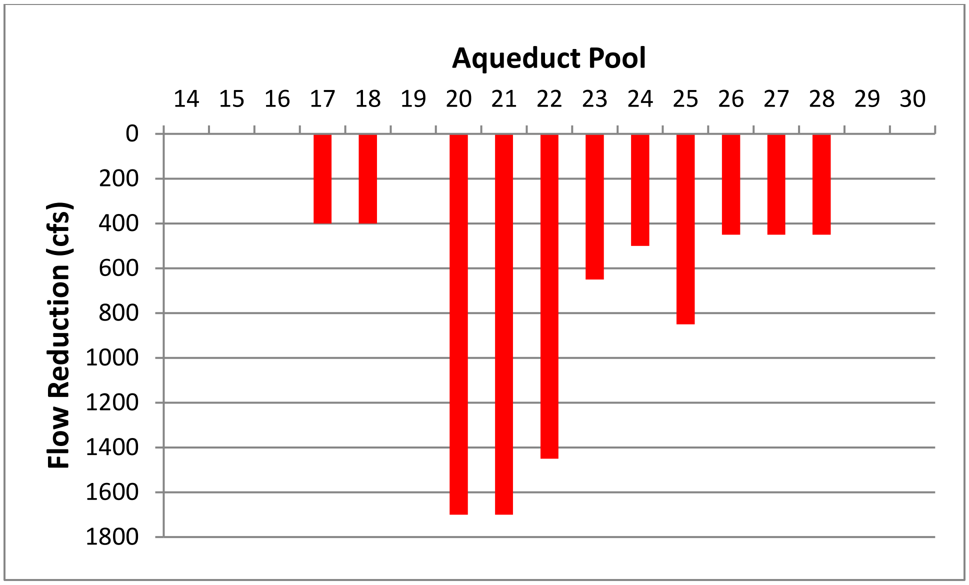



**Figure 8-7** **Reductions in Flow Capacity, for n=0.020 and Minimum Lined Freeboard of 0.5 feet**

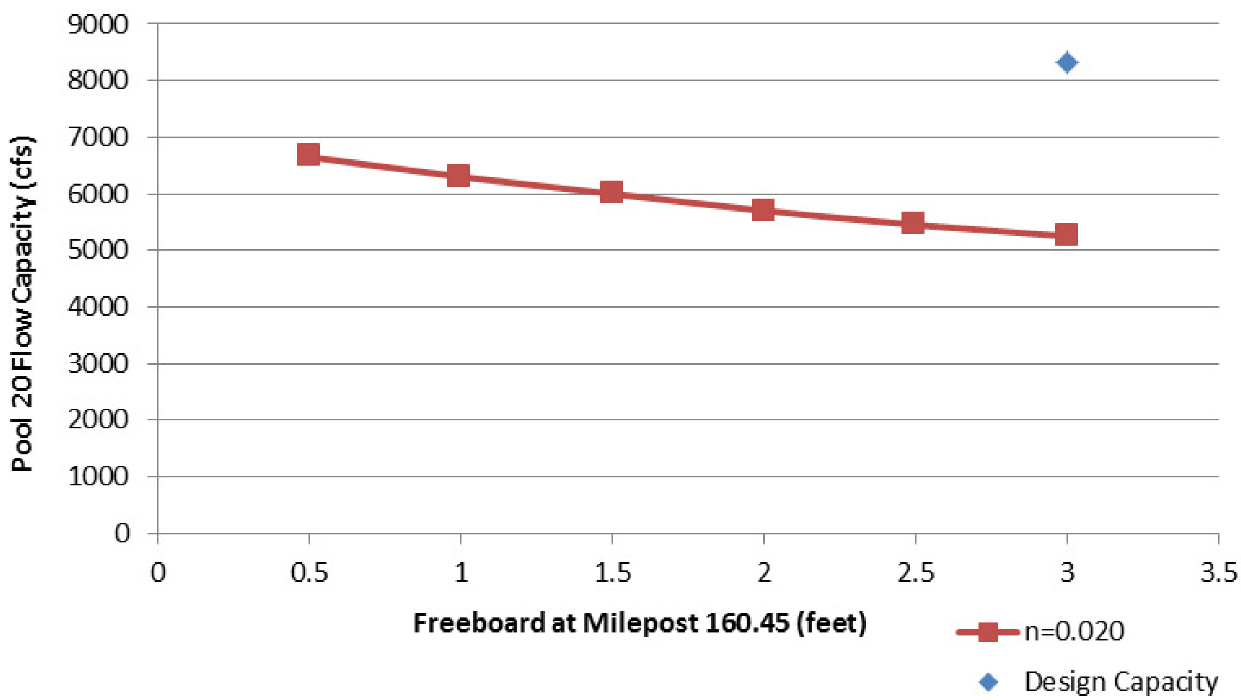


## 8.3 Additional Subsidence and Pool 20 Capacity

For a desired freeboard of 0.5 foot, the capacity of Pool 20 has been reduced by about 1,700 cfs. It is anticipated that the subsidence will continue in the future, impacting the Aqueduct conveyance capacity. In order to assess impact of additional subsidence on the Aqueduct capacity, the HEC-RAS model was re-run for increased freeboard. The model was run separately to estimate the maximum flow for a freeboard of 1 foot, 1.5 feet, 2 feet, 2.5 feet, and 3.0 feet for Pool 20 near Milepost 160.42. The minimum operating elevation for Pool 20 is 313.4 feet, and based on the 2015 GPS survey, the lining's top elevation at Milepost 160.45 is 316.31 feet. Because the additional freeboard based upon the standard operating order is 2.9 feet, the sensitivity analysis was stopped at 3 feet.

The maximum flow passing through Pool 20 as a function of freeboard is shown in Figure 8-8. The modeling results show that for a freeboard of 0.5 foot the flow capacity is 6,650 cfs, and for a freeboard of 3.0 feet the flow capacity is 5,250 cfs. The figure also shows flow capacity of Pool 20 decreases linearly with subsidence. This suggests additional subsidence will further reduce the capacity of Pool 20.

**Figure 8-8 Reductions in Pool 20 Flow Capacity with Increase in Freeboard for n=0.020**




Note: At 2.6 feet of freeboard, Pool 20 would be lower than minimum elevation; turnouts may be inoperable.

## Table 8-3 Design and Revised Flow Capacity for Aqueduct Pools

| Pool Number | Original Capacity (cfs) | Original Minimum Freeboard (feet) | n=0.016 and Minimum Freeboard 1.5 feet | | | n=0.020 and Minimum Freeboard 0.5 feet | | |
|---|---|---|---|---|---|---|---|---|
| | | | Reduced Capacity (cfs) | Decrease in Flow (cfs) | New Freeboard (feet) | Reduced Capacity (cfs) | Decrease in Capacity (cfs) | New Freeboard (feet) |
| 14 | 13100 | 3.0 | 13100 | 0 | 3.72 | 13100 | 0 | 1.20 |
| 15 | 13100 | 3.0 | 13100 | 0 | 4.31 | 13100 | 0 | 2.02 |
| 16 | 11800 | 3.0 | 11800 | 0 | 4.33 | 11800 | 0 | 2.43 |
| 17 | 11800 | 3.0 | 11800 | 0 | 2.27 | 11400 | 400 | 0.50 |
| 18 | 11800 | 3.0 | 11800 | 0 | 2.59 | 11400 | 400 | 0.76 |
| 19 | 9350 | 3.0 | 9350 | 0 | 6.00 | 9350 | 0 | 5.14 |
| 20 | 8350 | 3.0 | 7600 | 750 | 1.50 | 6650 | 1700 | 0.50 |
| 21 | 8350 | 3.0 | 7600 | 750 | 3.99 | 6650 | 1700 | 2.90 |
| 22 | 8100 | 2.5 | 7600 | 500 | 3.36 | 6650 | 1450 | 2.22 |
| 23 | 7300 | 2.5 | 7300 | 0 | 3.39 | 6650 | 650 | 2.17 |
| 24 | 7150 | 2.5 | 7150 | 0 | 1.76 | 6650 | 500 | 0.57 |
| 25 | 6350 | 2.5 | 6350 | 0 | 1.62 | 5500 | 850 | 0.50 |
| 26 | 5950 | 2.5 | 5950 | 0 | 2.58 | 5500 | 450 | 1.45 |
| 27 | 5950 | 2.5 | 5950 | 0 | 4.02 | 5500 | 450 | 2.80 |
| 28 | 5950 | 2.5 | 5950 | 0 | 2.48 | 5500 | 450 | 1.40 |
| 29 | 5350 | 2.5 | 5350 | 0 | 2.55 | 5350 | 0 | 1.44 |
| 30 | 5050 | 2.5 | 5050 | 0 | 3.36 | 5050 | 0 | 3.12 |

Note:
cfs = cubic feet per second

## 8.4 Chapter Summary

The following are the preliminary findings based upon the modeling discussed in this chapter:

- Subsidence of the Aqueduct has reduced the freeboard between the maximum WSE and the top of the lined canal.
- The Aqueduct was designed with a Manning's roughness coefficient of 0.016. Because of subsidence and accumulation of debris, the channel roughness is probably higher.
- The current operation data was not sufficient to estimate the Manning's n-value. But, after reviewing underwater photos, videos, and survey data, an n-value of 0.020 was selected to represent the average roughness coefficient.
- The subsidence has reduced the flow capacity of multiple pools. The model results show that the flow capacity of Pool 20 and 21 is reduced by 1,700 cfs; the capacity of Pool 22 is reduced by 1,450 cfs.
- Any future subsidence will decrease the Aqueduct pool capacity almost linearly.

# Chapter 9. Summary, Conclusions, and Recommendations for Future Study

## 9.1 Conclusions

The purpose of this project was to research and study past and present subsidence reports and data to understand and summarize the location and magnitude of subsidence and its effects on the Aqueduct. This report has presented the data and information found and analyzed for this project. This section summarizes the findings of the study that are directly related to the purpose. 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.

For the purpose 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.

### 9.1.1 Location and Magnitudes of Subsidence

There are three "bowls" of subsidence along the alignment of the Aqueduct in the SLFD and the SJFD. The first bowl is the largest, located in Pool 15 through Pool 18, in the SLFD. The second bowl of subsidence is located in Pool 19 through Pool 21 in the SLFD. The third bowl is in Pool 23 through Pool 25 in the SJFD. Outside of these three areas, there has been a small amount of subsidence observed in Pool 34 and 35 in SJFD. For the rest of the two field divisions the only small amounts of ground surface movement have occurred. But, the UAVSAR results have shown that those areas could also be affected by subsidence in the future if excessive ground water pumping occurs.

#### 9.1.1.1 Pool 15 to Pool 18

The first bowl of subsidence begins at Milepost 98.31 in Pool 15 and ends at Milepost 142.40 in Pool 18, in the SLFD. The magnitude of subsidence steadily increases from Pool 15 downstream until it reaches -5.14 feet in Pool 17, at which point is begins to decrease and end in Pool 18. Subsidence magnitudes are based on survey data shortly after Aqueduct construction and the latest survey performed in 2015.

Proximity to the Coast Ranges seems to play a large part in the amount of subsidence. Generally, the farther the Aqueduct is into the Valley the greater subsidence; the closer to the hills, the less subsidence.

In this segment, the Aqueduct crosses five alluvial fans and the Aqueduct alignment arcs east away from the Coast Range foothills, to maintain the design gradient. As it does, the depth of the alluvium to the Tertiary bedrock increases from 850 feet to 1,900 feet below ground surface (bgs) in Pool 15. Through Pools 16 and 18 the depth of the alluvium ranges from 1,500 to 2,000 feet. The Corcoran clay profile becomes deeper as the Aqueduct moves away from the hills. Within Pool 15, the Corcoran clay ranges from 20 feet to 50 feet thick. The top of the clay ranges in depth from about 350 bgs at the beginning to about 850 feet bgs by the end of Pool 15. The depth of the clay remains around 850 feet bgs through Pools 16, 17, and 18. The thickness of the clay layer does decrease in Pools 17 and 18, in spots, to 10 feet thick and as much as 25 feet thick in Pool 18. This can be seen on Plates 3 through 6 (for Pools 15 through 18).

Figures 6-1 and 6-5 illustrate and tabulate the subsidence rates within SLFD. Within this first bowl, the subsidence at Mileposts 98.67, 116.27, 127.07, and 136.05 were plotted and evaluated, as they represent the location of greatest subsidence in each pool. At those locations there were steep initial rates of subsidence during Aqueduct construction, and during the time before significant deliveries were made. After water deliveries began, the subsidence rates flattened, to around 0.5 inch per year. But during dry hydrologic years there were 3.4 to 4.5 times increases in the subsidence rates in Pools 16 through 18. The 2013-2015 drought saw an even greater increase in the subsidence rates, as much as two to three times those of the historic dry hydrologic years.  Table 9-1 shows the average subsidence rates at the locations in the four pools.

**Table 9-1 Average Subsidence Rates of Pools 15, 16, 17, and 18 (inches per year)**

| Location | Construction to Beginning of Deliveries | Normal to Wet Years | Dry Years | 2013-2015 Drought |
|---|---|---|---|---|
| Pool 15 (MP 98.67) | -2.0 | -0.3 | -0.5 | -0.4 |
| Pool 16 (MP 116.27) | -5.7 | -0.3 | -1.2 | -2.1 |
| Pool 17 (MP 127.07) | -9.2 | -0.5 | -1.7 | -5.4 |
| Pool 18 (MP 136.05) | -8.0 | -0.4 | -1.8 | -3.5 |

Note: MP = milepost

In this area, 1 foot to 9 feet of extra lined freeboard was constructed with the Aqueduct and then the liner was raised at least 24 inches in some spots, and as much as 58 inches in other spots, in 1982 to maintain adequate freeboard.

### 9.1.1.2 Pool 19 to Pool 21

The second bowl of subsidence begins at Milepost 143.84, near the upstream end of Pool 19, and ends at Milepost 170.82 in Pool 21, in the SLFD. Within this area, the magnitude of subsidence is relatively constant between -2.4 and -3.6 feet from the middle of Pool 19 to the end of Pool 20. There is a sharp spike of 5.8 feet in subsidence centered on Milepost 160.45 in Pool 20. Subsidence magnitudes are based on survey data shortly after Aqueduct construction and the latest surveys performed in 2015. The alignment of this segment crosses one large alluvial fan and arcs out into the Valley.

Pool 19 through the upstream end of Pool 21 was constructed on the large Los Gatos Creek alluvial fan. The alignment of Pools 19 and 20 arcs away from, and then toward the Coast Range foothills. The alignment of Pool 21 parallels, yet moves closer to, the toe of the foothills, which matches the decrease in subsidence in the pool. Through Pools 19 and 20 the depth of the alluvium ranges from 2,000 feet to

3,200 feet bgs. The depth of the alluvium ranges from 2,100 feet to 2,400 feet bgs in Pool 21. The top of the Corcoran clay is a fairly constant depth below the three pools at about 800 feet to 1,000 feet bgs. The clay's thickness ranges from 10 feet to 50 feet bgs in this area, but pinches out between Mileposts 161 (Pool 20) and 166 (Pool 21). This can be seen on Plates 7 through 9 (for Pools 19 through 21).

Figures 6-1 and 6-5 illustrate the subsidence rates within SLFD. Within this second bowl, the subsidence at Mileposts 148.56, 160.45, and 166.45 were plotted and evaluated, as they represent the locations of greatest subsidence in each pool. As with the first bowl, in these locations there were steep initial rates of subsidence during Aqueduct construction, before significant water deliveries were made. After water deliveries began, the subsidence rates flattened, to less than 0.25 inch per year. During dry hydrologic years there was an increase in the subsidence rates in Pools 19 through 20 to approximately 1.5 inches per year. The 2013-2015 drought saw a 3 to 3.4 times increase in the subsidence rates in Pools 19 and 20. Pool 21 saw the greatest increase in subsidence. It appears to have subsided more than twice as much during the 2013-2015 drought than it had subsided since Aqueduct construction. Table 9-2 shows the average subsidence rates at these locations in the three pools.

**Table 9-2 Average Subsidence Rates of Pools 19, 20, and 21 (inches per year)**

| Location | Construction to Beginning of Deliveries | Normal to Wet Years | Dry Years | 2013-2015 Drought |
|---|---|---|---|---|
| Pool 19 (MP 148.56) | -7.4 | +0.1 | -1.3 | -3.9 |
| Pool 20 (MP 160.45) | -10.8 | -0.2 | -1.6 | -5.5 |
| Pool 21 (MP 166.45) | -2.3 | +0.0 | +0.0 | -4.4 |

Note: MP = milepost

In this 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 3 feet in 1982.

#### 9.1.1.3 Pool 23 to Pool 26

The third bowl of subsidence begins at Milepost 193.85 on the downstream end of Pool 23, through Milepost 219.38 on the upstream end of Pool 26, in SJFD. Within this area, the subsidence quickly spikes to -3.3 feet between Mileposts 193.85 and 196.74, then decreases to -0.97 feet at Milepost 203.92. The subsidence then increases back to Milepost 207.94 near Check 24, then decreases to -0.52 feet at Milepost 219.38 on the upstream end of Pool 26. Subsidence magnitudes are based on survey data shortly after Aqueduct construction and the latest survey performed in 2013.

The thickness of the alluvium along Pools 23 through 25 remains fairly constant, about 2,700 feet thick. The depth to the Corcoran clay ranges from 900 feet bgs in Pool 23, to a high point 500 feet bgs in the middle of Pool 24, then down to 800 to 600 feet bgs through Pool 25. The Corcoran clay ranges in thickness from 0 feet to 75 feet thick on the downstream half of Pool 23, decreasing to 20 feet thick in Pool 24, then staying at a fairly constant thickness of 20 feet though Pool 25.

Figures 6-2 and 6-6 illustrate the subsidence rates within SJFD. Within this third bowl, the subsidence at Mileposts 196.74, 199.55, and 207.94 were plotted and evaluated, as they represent the locations of greatest subsidence in each pool. Within this bowl, the subsidence rates have been relatively constant. The initial rates of subsidence during Aqueduct construction and those of the dry hydrologic years are about 1.5 to 2 times higher than those of the normal to wet years. At the time of preparing this report, the most recent survey of SJFD was in 2013. As a result, there are no comparable subsidence rates or magnitudes for the 2013-2015 drought. Table 9-3 shows the average subsidence rates at these locations in the three pools.

**Table 9-3 Average Subsidence Rates of Pools 23, 24, and 25 (inches per year)**

| Location | Construction to Beginning of Deliveries | Normal to Wet Years | Dry Years |
|---|---|---|---|
| Pool 23 (MP 196.74) | -2.2 | -0.8 | -1.3 |
| Pool 24 (MP 199.55) | -1.1 | -0.7 | -1.1 |
| Pool 25 (MP 207.94) | -1.6 | -1.1 | -1.0 |

Note: MP = milepost

In this 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.

### 9.1.2 Local Subsidence

The term “hotspot” has been used in this report to define an area of subsidence that has a localized circular/bowl shape, develops quickly, and unexpectedly. The hotspot discussed in Chapter 6 (near Avenal Cutoff Road, Milepost 163.69) showed more subsidence than the surrounding area. After the creation of the first few sets of UAVSAR interferograms the area was noticed near Milepost 163.69, which had more subsidence than the surrounding area. The results of those comparisons are discussed in section 6.6.1.

Figure 6-9 shows a detailed plan view of the hotspot bowl area. The bowl is approximately 6,200 feet when measured north to south; but the subsidence “plume” caused the bowl to widen to about 7,200 feet when measured east to west. In this interferogram, the magnitude of subsidence of the bowl is -6 inches to -7 inches on its edges and as much as -14.2 inches in the center. The subsidence in the surrounding areas, which is uncolored in Figure 6-9, ranges between -2 inches and -5 inches.

It was reported that there are four wells located within this hotspot bowl, but no records were found in DWR databases. Because of the rapid development of these hotspot locations, adjacent to the Aqueduct, DWR needs to be informed of the construction of new wells within a certain distance of the Aqueduct. Also, a monitoring program is needed to observe the initial development of hotspots and be able to respond with an appropriate fix.

### 9.1.3 Hydraulic Impacts

To evaluate and quantify the impacts of subsidence on the hydraulic capacity of the Aqueduct, hydraulic models were built in HEC-RAS. The models were built to match the subsided geometry of the Aqueduct based on the most recent survey data. To evaluate the Aqueduct’s current capacity in the model, 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.

Within the three subsidence bowls, the reduction of the Aqueduct freeboard elevation required a reduction of the flowrate to maintain 0.5 foot of freeboard. Each of the bowls has a low spot in the freeboard that creates a hydraulic "choke point" that limits the flow capacity. The reduced flow capacity results in reduced capacity of the pools downstream. The reduced Aqueduct capacities from the model are shown in Table 9-4. Though capacities are reduced, contracted deliveries have not been curtailed through 2016.

**Table 9-4 Modeled Reduction of Aqueduct Capacity because of Subsidence**

| Pools | Design Capacity (cfs) | Current Capacity (cfs) | Reduction (cfs) |
|---|---|---|---|
| 17[a], 18 | 11,800 cfs | 11,400 cfs | -400 cfs (-3.4%) |
| 20*, 21, (22, 23, 24)[b] | 8,350 cfs | 6,650 cfs | -1700 cfs (-20%) |
| 25* | 6,350 cfs | 5,500 cfs | -850 cfs (-13%) |

Notes:

cfs = cubic feet per second

[a]Actual location of the hydraulic "choke point". Other locations are a carry-over downstream reduction.

[b]Design Capacity and Reduction of these pools are less than shown, but current capacity will remain.

Comparing the hydraulic model results with recent subsidence rates and magnitudes presented in Chapter 6, it can be concluded that all locations of the hydraulic choke points in the model (Mileposts 118.0, 160.45, 209.37) will continue to subside. None of the locations, or similar locations that are close to being choke points, show signs that the subsidence has plateaued or slowed. Accompanying each of the subsidence bowls is a downstream segment that shows little to no subsidence. As a result, future subsidence will continue to reduce the capacity of the Aqueduct. The site-specific subsidence rates for the choke points are tabulated in Table 9-5, for various hydrologic years.

**Table 9-5 Hydraulic Model Choke Point Subsidence Rates (inches per year)**

| Locations | Wet to Normal Year | Dry Year | 2013-2015 |
|---|---|---|---|
| MP 118.0 (Pool 17) | -0.1 | -0.7 | -2.1 |
| MP 160.45 (Pool 20) | -0.2 | -1.5 | -4.7 |
| MP 209.37 (Pool 25) | -0.8 | -0.8 | N/A |

Note: N/A = not available, MP = milepost

As part of the hydraulic modeling, the flowrate in Pool 20 was varied to create a relationship between flowrate and the minimum freeboard, which is at Milepost 160.45. Figure 8-8 was created to show that relationship. The relationship is the same as that between future subsidence and flowrate capacity, except for one small difference. The geometry of models created for a future subsidence and flowrate capacity would each need to be modified to account for the incremental subsidence. The slight change in the geometry would only slightly change the hydraulic results in the models.

The capacity of Pool 20 is currently 6,650 cfs, with 0.5 foot of freeboard (Figure 8-8). From the figure, at 1.5 feet of freeboard the capacity would be 6,000 cfs. That amount of freeboard (1.5 feet) is the same as 0.5 feet of freeboard with an additional 1 foot of subsidence. As a result, if an additional foot of subsidence occurs, the capacity would fall to 6,000 cfs. If an additional 2 feet of subsidence occurs, the capacity would fall to 5,450 cfs. At that point some of the turnouts would start becoming inoperable, because of low water levels in the pools.

Table 9-5 above shows that there is a wide range of subsidence rates for various hydrologic years. For this location in Pool 20 the subsidence rates ranged from -0.2 inch per year in wet to normal years, -1.5 inches per year in dry years, and -4.7 inches per year during the 2013-2015 drought. Putting these rates in the context of the loss of capacity discussed, at the dry year subsidence rates it would take 16 years for Pool 20 to subside the 2 feet. If the subsidence rates that were observed between 2013 and 2015 continued to occur it would only take 5.1 years for 2 feet of subsidence.

## 9.2 Ongoing Monitoring

During future phases of this project, additional monitoring data will be collected by established programs, or at the request of this project. All new survey data will be incorporated into the ongoing efforts of this project.

The Precise Surveys Unit is scheduled to conduct additional leveling surveys of the SLFD and the SJFD during spring 2016. Once the elevation data are received it will be incorporated into Phase 2 of this project. At that time, changes to the hydraulic model described in Chapter 8, and new reduced flowrates, will be made available.

As new UAVSAR data are received from NASA's JPL, it will be compared to previous sets of data to identify areas of increased subsidence. A third flight line that would bridge a gap between the northern flight line and the southern flight line is being discussed with NASA. That third flight line would cover the large subsidence bowl in the SJFD that spans Pools 23 through Pool 25.

## 9.3 Sustainable Groundwater Management Act

In September 2014, California Governor Edmund G. Brown Jr. signed three bills into law which comprise the Sustainable Groundwater Management Act (SGMA). SGMA is a comprehensive package that provides a framework for sustainable management of groundwater supplies by local authorities, with a limited role for state intervention, only if necessary, to protect the resource. SGMA authorizes local agencies to adopt groundwater management plans which are suited to the resources and needs of their communities.

Those plans are intended to provide a buffer against drought conditions and contribute to reliable water supplies regardless of weather patterns. SGMA does not provide for establishing rights and priorities to the use of groundwater. Pending legislation would streamline the adjudication processes for establishing rights and priorities to the use of groundwater.

SGMA provides five to seven years for local governments to form a groundwater sustainability agency (GSA) and develop a groundwater sustainability plan (GSP) for each groundwater basin that has been designated a high or medium priority by the State. Each GSP would have a 20-year implementation horizon with the opportunity for two five-year extensions, if the GSA is making progress toward sustainability. SGMA protects existing surface water and groundwater rights and does not impact current drought response measures. Preparation of GSPs is exempt from the California Environmental Quality Act.

DWR has developed a strategic plan for its Sustainable Groundwater Management Program. The program will implement the new and expanded responsibilities identified in SGMA. Some of the expanded responsibilities include: (1) developing regulations to revise groundwater basin boundaries, (2) adopting regulations for evaluating and implementing GSPs and coordination agreements, (3) identifying basins subject to critical conditions of overdraft, (4) identifying water available for groundwater replenishment, and (5) publishing best management practices for the sustainable management of groundwater.

This study suggests that O&M participate in the GSAs and contribute to the development of the GSPs. Its participation should be focused on the land and facility, the Aqueduct, which span the groundwater basins. The impacts and issues of subsidence and groundwater pumping on the Aqueduct may be very different than the majority of the other stakeholders involved in the GSAs. DWR and USBR have a great interest in the sustainability of the Aqueduct. Participation in the GSAs would help preserve those interests.

## 9.4 Phase 2 Study

The magnitude and rates of subsidence presented in Chapter 6 show that during dry, normal, or even wet hydrologic years, subsidence continues. As subsidence continues, the capacity of the Aqueduct will continue to decrease. The reduction of flow capacity (Section 8.3), reduction in storage volume, and the decreased freeboard (Sections 7.3 and 8.3) illustrate that under the current conditions the Aqueduct cannot be operated as designed.

The following objectives are planned for Phase 2 of this project:

1. Develop future operational conditions for the Aqueduct. Obtain cost and operational impacts for all reasonable conditions. Examples of future operational conditions include returning to original design, flattening the flow schedule, and scheduling turnout flows.
2. Identify all project alternatives options to meet the operational conditions. Develop, in more detail, an alternative matrix of the top few alternatives for each condition. The alternatives could include raising a canal liner, building a storage facility, or adding a new Aqueduct segment.
3. Collect information on groundwater wells near the Aqueduct from DWR's groundwater well database. Tabulate and analyze the distribution of the wells to aid in future subsidence predictions.
4. Collect right of way boundary information and canal structure elevation data. Data will be used to evaluate impacts and costs for proposed project alternatives.
    A. Plan and schedule a flow test in the SLFD and/or the SJFD to calibrate the HEC-RAS model created in Phase 1.
    B. Engage stakeholders with a developed alternative matrix. Ensure alternatives align with other DWR projects and programs.
    C. Select preferred alternative. Develop the scope, sequencing, and schedule for implementation.

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# Appendix A. Semigraphic Methods for Prediction of Residual Subsidence

# Appendix B. San Luis Canal Past and Future Elevations at Top of Canal Lining (1981 Data)

# Appendix C. Standing Operating Order PC 600.22 Aqueduct Maximum/Minimum Elevations and Drawdown Criteria

# Appendix D. California Aqueduct Re-leveling Adjustments from Dos Amigos to Edmonston

# California Aqueduct Re-leveling Adjustments, Dos Amigos to Edmonston.

February 3, 2015

Let's start with a brief explanation of our leveling procedures along the California Aqueduct. Our leveling procedures are what is called first order, several things must be done for them to be first order. We must keep our turns balanced(distance between instrument and rods), a heavy trivet(this is what the rod rests on and turns forward to back) is used for our turns, a temperature is taken at each mark we close on and used in a program when calculating the DE(difference in elevations). We double ran each section(the area between bench marks), which means we had a forward and a back run, we would then check our DE's between the marks and we needed to be within .003 meters per kilometer or we would have to re-run the section until we achieved this. Our equipment must also meet certain criteria in order to produce precise measurements, our rods must be calibrated and our instrument must be peg tested. We double ran our sections until 2000. In 2000 we started single running the sections in alternate directions each day, for instance we would run Southerly along the Aqueduct one day, then project what we could achieve in the next day and run Northerly until we tie into where we left off the previous day. We found this to be very effective in eliminating any systematic errors we may have. Our equipment also changed from having  to read 3 sets of numbers on the rod to what we have now, bar code rods, so we do not have to read any numbers which eliminates errors in that respect. Our spurs( a section that is not along the aqueduct) are still generally double run and also sections that we find significant changes in the DE from the previous observation. In 2000 we also stopped running our hill ties between Dos Amigos and Kettleman City and  in 1993 most of the hill ties from Kettleman city to Edmonston were eliminated. Hill ties are level runs from Aqueduct to marks that are located in the hills Westerly of the Aqueduct and are located at or near the end of each hill tie, set in a rock outcrop for stability, and were held vertically.( a held mark is a mark that is considered very stable and the elevation is assumed to not change, therefore; it is held and other elevations are adjusted by it.) There were 4 hill ties between Dos Amigos and Kettleman city they range from 6 to 13 miles Westerly of the Aqueduct, and 2 from Kettleman to Edmonston ranging from 7 to 13 miles Westerly of Aqueduct. The elimination of Hill ties and double run levels has given us the ability to conduct these surveys with one crew and in a timelier and cost efficient manner. Also with the addition of GPS capabilities we are able to check our held positions and DE's between check structures.

Now let's go over the various adjustments on the California Aqueduct. The Vertical datum is based on the National Geodetic Vertical Datum of 1929(NGVD 1929), and is the same datum used to construct the water project. The newest datum is North America Vertical Datum of 1988(NAVD 88), these elevations are produced by GPS observation, and are currently being used on any new projects. For monitoring purposes we continue to use the NGVD 29 datum.

As far as adjustments go you first must understand that the Constrained Adjusted Datum is basically, what we have decided is the best elevation for a held mark, so that the rest of the marks can be adjusted to. For instance the 94 Adj, is the 1994 Constrained Adjusted Datum, and that means that we have changed(updated) our held elevation or elevations in 1994 and will adjust all other marks according to those values. How much we adjust is dependent on how our field elevations close on the held mark elevation, so for instance we ran levels in 1986 and it says above the date that it was 94 Adjustment, so what that means is that our 94 held elevations is used to adjust out our miss closure based on our field elevation and held mark, at that time the held mark would be the mark near or at the end of hill tie.

**The basic history of our Adjustments, why and how we obtained them.**
Basically, up until 2000, each re-leveling of the California Aqueduct has been adjusted to a series of rock marks surrounding and leading into the West and South sides of the San Joaquin Valley, the elevations were initially established by the National Geodetic Survey or NGS. The 1967 thru 1975 leveling was provided by NGS. After the 1975 re-leveling by NGS was completed, Ben Lofgren with U.S. Geological Survey, Water Resources Division, Sacramento, wanted the 1975 re-leveling adjusted in a manner identical to previous adjustments. In 1975 the word "Constrained" was attached to these adjustments. All field observed elevations are referred to bench mark C 926 located near the Dos Amigos pumping plant in a large monument well. Beginning with the aqueduct re-leveling date January 1972, the observed field elevations in the Kettleman City area indicated an apparent rising relative to the constrained elevations. Again, in 1975, the aqueduct was re-leveled by the National Geodetic Survey unit and the observed elevations to the hill tie bench marks were higher than the previous observed values. Later re-leveling of the aqueduct indicated that the observed field data sets were not consistent with the 1975 constrained values. In general terms, the observed data sets were showing a gradual rising on the held hill tie marks. In order to bring the observed data set more in line with a constrained reference datum, a decision was made to calculate elevations that were more consistent with the observed values. This is when we developed our July 1994 Constrained Adjusted Datum, it was calculated using the mean absolute differences between the field observed elevations and the constrained 1975 held hill tie values. We then went back and re-adjusted some previous observations as far back as we felt was needed, again adjusting between held marks based on the field elevation closure for each year on the new held elevation.

In 2009 a new held adjusted elevation for the held mark on the Kettleman City hill tie was established by GPS methods. To determine the held elevation on Z 1159 we used our GPS observed elevations on 86.64L(Dos Amigos Outlet), since C 926 is not a GPS able mark, and Z 1159 rock mark at the end of Kettleman city hill tie. This was based on the mean GPS NAVD 88 Ortho HT OPUS(online position users system) processed elevations on 8 meaned observations on 89.64L, 2005 to 2009 and a mean of 5 observations on Z 1159, all of which have at least 4 hours of observation time. I used the leveled DE from C 926 to 86.94L(2009) to calculate an elevation on C 926. Then I took the difference between the meaned GPS elevation on Z 1159 to get the GPS DE between the held mark C 926 and Z 1159. I then took the held C926 NGVD 29 elevation and applied the GPS derived DE to get the new held elevation on Z 1159. The elevation was also verified by using NGS software program, Level height difference computations(LVL_DH program) . This program calculated a DE between two observed GPS points, I used 86.94L and Z 1159 , the calculated DE and our field elevation on 86.94L, and I came up with an elevation on Z 1159 of only -1.4 millimeters different than what I had come up with. I used this for 2013, 2009, 2006 and 2000 adjustments between Dos Amigos and Kettleman City, Kettleman city to Edmonston was not observed in 2009. This same procedure was used to establish a new 2013 adjustment between Z 1159 and G 1220( held mark above Edmonston PP.) The G 1220 held elevation was adjusted -.056 meters from the previous 2006 adjustment. The bottom line is that these held elevations, in the big picture, have not been adjusted that much. Z 1159 was only corrected +.006 meters from 94 constrained adjustment to 2009 constrained adjustment. The biggest change was from the 75 constrained adjustment to the 94 adjustment which was a +.103 meters, or 4.05 inches, although some of these also took into consideration the hill tie held elevations changing.

Each time we observe the Aqueduct elevations we will continue to monitor our held mark elevations by GPS over several hours and multiple days of observations.

We are currently getting ready to observe the Aqueduct elevations by GPS static method for the first time. If we find that this method is accurate enough we will be able to observe these elevations much quicker than with conventional levels and therefore; more often.

**Update**

As of 09/21/2015 we have completed two observations on the California Aqueduct from Dos Amigos Outlet, mile 86.94L to Kettleman City hilltie held mark.

The DWR Precise survey crew has completed the Static GPS survey on the California Aqueduct between Dos Amigos PP and Kettleman City. The first survey was observed from February 11 into March 2015.

This survey was based on the Static GPS method. We started on 86.94 L South, which is located on the Dos Amigos Outlet structure, and continued South along the Aqueduct. At 86 94 L South we set our first GPS base station and on each check structure from there on. Each session consisted of an average of 6 hours with 2 to 4 bases running. The intermediate marks between bases were observed for 8 minutes+ and were adjusted using the calculated held elevations for each base. The base station elevations were determined after the survey was completed and was based on the mean OPUS(Online Position User System, Geoid 12A) elevations for 4 to 6 sessions on each base. The starting elevation on 86.94 L South was observed by conventional levels from C 926 (non GPS mark), the project beginning held mark, and ended on Y 1159, our new current held mark at the end of the Kettleman City Hill tie. Our previous held mark Z 1159 is now enclosed inside a hazardous waste area by a cyclone fence. Y 1159 is located in a rock outcrop about 100 yards from previous held mark and has a stable history, the 2013 adjusted elevation will be held. Continuing along the way updating the elevations on each check structure we ended on Y 1159 and missed the held elevations by -.011 meters, over approx. 90 miles. That is less than a half inch for those non metric people.

Our survey left us with some significant changes from Check 16 to check 20. We came back out of it to a reasonable number at check 21, with our biggest difference in elevation, compared to 2013, coming between check 20 and 21, about .230 meters or approx. 9.5 inches. We decided to check this 8 mile+ span with conventional levels and we missed our GPS DE by only 19 millimeters. Based on this check and our consistent OPUS elevations on the check structures and our closure on Y 1159, I am confident that our measurements represent the settlement along the aqueduct. We could not observe all the marks because some were not GPS friendly.

In August 2015 we had a request to check the DE between check 19 and 20 with conventional levels. We missed our GPS DE by .005 meters. I decided to observe the check structure marks at check 19 and 20 during our leveling, just in case we observed some changes. It turned out that Check 19 was down another -.136 meters and Check 20 was down -.132 meters since March 2015. So, even though the DE matched, since they were almost equally down the elevations had significantly changed. I decided to continue to re-observe all the Check structures from Dos Amigos Outlet to the held mark, with the same procedure as in March of 2015. We missed our March elevation on the held mark(Y1159) by only -.0085 meters, and found more significant movement on most Check structure control marks. We did not observe the marks between check structures during this survey, except for the conventional levels run between Check 19 and 20, and we also observed all the recorders with conventional method, since they are not GPS friendly.  Overall we were very satisfied with our elevations by Static GPS Method and believe that we can continue to monitor the Aqueduct with this method.

The purpose of this letter was to give you a better understanding of DWR Precise survey's California Aqueduct leveling procedures past and present and our adjustments, past to present.

Joseph D. Mello
DWR Precise Survey
San Luis Field Division

# Appendix E. San Luis Field Division and San Joaquin Field Division Field Trip Notes

# San Luis Field Division July 8-9, 2015 Field Trip Notes

- Rich suggested that we use 2006 as our benchmark to compare others years too. It was the 'last' year where they were flowing at full capacity (delivering 100%).

- 2007, 2008, and 2009 were dry years. Last wet year was 2010. We 'moved enough decent water.' After 2010, we've had dry years. Don't know the current capacity of the canal since it has subsided a lot since then.

- Rich mentioned the lost capacity noticed in 2006. In certain areas (pool 20) that subsidence from 2006 to 2013 is the same magnitude as 2013 to 2015. I'd like to check the subsidence rates (in/yr or ft/yr) from 1983 to 2006, 2006 to 2013, and 2013 to 2015.

    - Since 2013, not moving a lot of water. Don't look at the last 2 years, 2014 and 2015.

- Operations doesn't currently open check 27 right now. Water demands from pools 27-40 are being delivered by Kern Water Bank. Water demands from pools upstream, by the Aqueduct.

- Mike pointed that the pumping pattern is changing because of the change in power market.

- Rich pointed that the DAPP is started and water is stacked at Pool 14 for about 45 minutes. Then all the gates are opened simultaneously. It takes about 2 hours to reach the water BVPP. Thus there is a time lag of about 3 hours between DAPP and BVPP.

- OP350R- Operational Criteria for San Luis Canal.

- Pool 14-30 is known as the 'Dos' stretch.

- Check 21- Split between 2 divisions.

- Original drawdown criterion 1 ft per day has been modified.

- Pools 14-21 = 24" drawdown per day.

- Pools 22-40 = 18” drawdown per day.

- Check 20= Well installed 0.25 miles away from aqueduct and ground has sub-sided 0.8’ in 1 year. Could this be MP 163?

- OP admitted that they run flows above the liner pretty frequently. In 2013, pool 25 lost its primary road (MP 208.8).

- Pool 24 = a 30” cap was added (in 1996. See 96-19).

- Pool 20 = constantly flowing above liner; sometimes at pool 18 as well. Water Ops will catch it and call it in. Pool 20 has maximum WSE limitation whereas Pool 18 has minimum WSE requirement.

- Canal is pretty much still operated as it was originally designed to be operated.

- After physically seeing the canal, you can definitely see the limitations at pool 20. You are only able to see 1 rung of the canal ladder, where in other ‘ok’ areas you are able to see up to 7.5, averaging about 5-6.

- Wendy pointed out that they are constantly watching laterals 25R and 33L at MP 160.45 (pool 20). These two turnouts have really subsided.

- DAP was designed for 24/7 operation. But it is not being operated that way and there is reverse flow.

    - Check 17 and 24 are very close to the water surface elevation. Their gates are out of the water.

    - Pools 19 & 20 have very little freeboard, Pools 16-18 and 24 appear to have enough (for now).

    - Subsidence values that we have plotted correlate with field observations.

**San Joaquin Field Division 7/29-7/30, 2015 Field Trip Notes**

- Met with Ron Wolfe on 7/29 and talked about MP 245.09 where they have some severe cracking (broken panels) at Buena Vista Bridge (pool 30). He mentioned that the problem has been getting worse and worse over the years and no efforts have been made to repair it. He also told us about issues at MP 185 and about a boil near Taupman Rd. Apparently Geology has been out there (not sure when?) and took some soil samples (exploration work). SJFD is still waiting on that report from Project Geology.

- While inspecting pools 24 and 25, noticed that the overchutes at MP 196.58 and 208.11 were in the water. The chute at MP 208.11 is also known as Check #24.5.

- Also noticed that Check #24 is operated with the gates out of the water. And in talking with ACC operators, it seems like they've always operated it this way.

- While driving along the canal, noticed that at some point the canal didn't have any unlined freeboard. It was somewhere in Pool 24, near the check structure.

- One of the things that really stuck with me is that this part of the canal has a lot of overchutes, and has either low embankments, or is in cut.

- During our 7/30 meeting, and in talking to ACC operators, subsidence issues with Coastal were brought up as well. Apparently pool 3 or 4 has some issues and they have a hard time pushing water through this part of the canal. Not sure exactly what pool, Darren Choyce mentioned pool 3, but Rossi in ACC, mentioned pool 4.

- In general, I got the sense that SJFD is under the impression that this subsidence project is going to fix all of their issues along the canal. We were given a list of problem areas, or hotspots, nowhere near our pool 24 and pool 25 regional subsidence areas.

- Check 17 and 24 are very close to the water surface elevation. Their gates are out of the water.

- Pools 19 & 20 have very little freeboard, Pools 16-18 and 24 appear to have enough (for now).

- Subsidence values that we have plotted correlate with field observations.

# Appendix F. Geology of San Luis Canal Excavation

# GEOLOGY OF SAN LUIS CANAL EXCAVATION

## General

The San Luis Canal is located on the western side of the San Joaquin Valley along the eastern flank of the Diablo Range. The Diablo Range is formed by a broad anticlinal structure which has a central core composed of slightly metamorphosed sedimentary and igneous rocks of the Franciscan Formation. The eastern flank of this large structure is formed by a thick sequence of Cretaceous age marine sandstone, shale and conglomerate. These sedimentary beds, which are part of the Panache Formation, dip fairly uniformly eastward toward the center of the San Joaquin Valley. In some areas the Panache Formation is in turn overlain by soft Tertiary age marine sedimentary rocks also dipping to the east and forming low foothills along the western side of the valley. Unconsolidated continental deposits of Tertiary and Quaternary age also occur in these foothills of the Diablo Range.

The San Joaquin Valley is a structural trough filled with several thousand feet of alluvium derived from the Sierra Nevada Range to the east and the Diablo Range to the west. The San Luis Canal traverses an area of piedmont deposits composed of detritus derived almost entirely from the Diablo Range. Mostly alluvial fan material was encountered in the canal excavation which had a maximum depth of about 30 feet. Units of material that were distinct enough to be given field names, were named after the stream depositing the material. The Corcoran Clay member of the Tulare Formation and gravel deposits in the Tulare Formation were the only previously named, mappable units encountered in the excavation.

Geologic mapping was accomplished by logging selected sections along the 2:1 excavation slopes and examining intervals between sections for any significant features. The field data were then plotted on a geologic map to show the lateral extent of the various soil types. Spacing of the logged sections was determined in the field, depending on how rapidly the materials changed. Most sections were made at 100- to 500-foot intervals, although in many areas the spacing was 50 feet or less.

The geologic maps of the canal excavation prepared during construction are on file in the USBR Regional Office; a list of drawing numbers for these maps can be found in USBR, 1971, Geology Construction Report, San Luis Canal, California Aqueduct, Mile 70 to Mile 172.

## Mile 70 (O'Neill Forebay) to Mile 77 - Invert Elevation 192 feet

The San Luis Canal excavation actually begins in the eastern most arm of the O'Neill Forebay reservoir. Terrace deposits, recent alluvial deposits, and the Corcoran Clay member of the Tulare Formation encountered the first 7 miles of the canal excavation were correlative with Post-Cretaceous units encountered in borrow areas and the foundation trenches for the San Luis Dam. The general stratigraphic sequence indicated by preconstruction core drilling in this general area was Panache Formation conglomerate, sandstone, and shale; Tulare Formation gravels; Corcoran Clay member of the Tulare Formation (Frink and Kues, 1954); and recent terrace and alluvial deposits.

The Corcoran Clay was the most distinctive mappable unit encountered in the canal excavation. It was almost continuous throughout this 7-mile section. In several sections of the excavation the entire prism was cut in Corcoran Clay showing a thickness of at least 30 feet. The Corcoran, designated originally as Moraga Clay (MC) on the geologic maps, was usually an olive-gray color, and was composed almost entirely of clay or silt of high to medium plasticity. The unit is massive showing only a slight horizontal structure. Shrinkage cracks are common both horizontal and vertical; often these cracks are coated with black stains. In some areas there are scattered limonite stains in the clay and a calcareous caliche zone near the top of the unit. Small gypsum crystals usually about 1-inch long occur in some areas. Only scattered vertebrate fossil fragments (Horse, Mammoth, and Bison) were found in the canal excavation between mile 70 and 79. However, several complete fossil vertebrates were found within the Corcoran in the foundation trenches at San Luis Dam and in the San Luis Canal excavation near Mile Post 92. Diatoms were usually common in the Corcoran.

The existence of Corcoran Clay in the canal prism, although of academic interest, was undesirable because the highly plastic material excavated from the canal prism was not suitable for the construction of canal embankments. Also, special drains were required to insure the stability of the canal where the Corcoran was encountered. These and other construction problems related to the Corcoran are discussed further in Part III of USBR, 1971.

The lower contact of the Corcoran Clay was exposed in only one small area along this section of the excavation, just upstream from the Volta Road Bridge (Mile 76.78). Here the Corcoran rests apparently conformably upon a water lain deposit of loose silty sand with scattered channels of clean sand and fine gravel. This sand unit is probably equivalent to the Tulare and derived from the Panoche sandstone which underlies the Corcoran at shallow depth in this area and crops out just to the west of this section. Excellent exposures of the Panache Formation and the Corcoran Clay can be seen in the highway cuts along Interstate 5 about one-half mile west of the San Luis Canal in this area.

The upper contact of the Corcoran Clay is an erosional surface partly covered by terrace gravels deposited by San Luis Creek (designated slg on geologic maps). The upper surface of the Corcoran Clay is marked by deep channels, some filled with more recent deposits of sand and gravel also attributed to deposition by San Luis Creek and its tributaries. "San Luis Sediments" was the field name assigned to all the fluvial deposits overlying the Corcoran in this 7 mile reach of the canal.

These deposits are tan to gray, with distinct bands of black and yellow stains, generally at least 50 percent gravel with up to 25 percent silt and clay fines. Pebble counts performed on several gravel samples showed 85 to 90 percent hard platy graywacke, chert and vein quartz derived from the Franciscan Formation, 5 to 10 percent well to subrounded meta-volcanic pebbles and cobbles from the Panache Formation conglomerates, and about 5 percent well rounded weathered Tertiary volcanic cobbles.

These deposits are mostly unconsolidated; but, hard cemented layers up to 4 feet thick were encountered near the base of the unit between Mile 70 and Mile 73.5. These cemented layers required blasting and special excavation equipment to remove them from the canal prism. The treatment of the cemented material is discussed in USBR, 1971.

Mile 77 to Mile 80 - Canal Invert Elevation 192 feet

The canal prism through this 3-mile section was excavated mostly through the terrace gravels and channel fill alluvium deposited by Los Banos Creek. Finer grained material, silty sand and sandy clay, occurred mostly in the area downstream from Mile 79. The Los Banos Creek gravel deposits are light brown to gray with a high percentage of well rounded material containing up to 9 percent of cobbles greater than 3-inches in size. The most recent gravels deposited by Los Banos Creek average about 45 percent well rounded Tertiary volcanics and 46 percent angular to subrounded Franciscan derived pebbles, mostly graywacke and chert. Older terrace gravels contain an average of 72 percent Tertiary volcanics and 28 percent Franciscan types. The gravel deposits are generally calcareous and occasionally pebbles and cobbles are partly coated with calcareous white material. These gravel deposits are quite extensive both sides of the canal excavation and are the major commercial source of concrete aggregate and road fill in the Los Banos area.

The Corcoran Clay occurred almost continuously in the lower most part of the canal excavation to Los Banos Creek (Mile 79). The depth to the Corcoran Clay below the canal excavation increases from Los Banos Creek to the Dos Amigos Pumping Plant where the top of the Corcoran is at elevation 150, about 10 feet below the foundation of the plant.

Mile 80 to Mile 87 (Dos Amigos Pumping Plant) - Canal Invert 192 feet

This 7-mile section of the canal was excavated through the alluvial fan material deposited by Salt Creek, Ortigalita Creek and minor streams between Ortigalita Creek and the Dos Amigos Pumping Plant.

The "Salt Creek Sediments" are predominately light brown clayey sand with minor beds of sandy clay and silty gravel. The gravel beds are discontinuous and fine grained. A pebble count of several gravel beds showed the following averages: 64 percent angular to subrounded Franciscan Formation type pebbles, mostly graywacke and chert; 33 percent rounded Tertiary volcanic pebbles and cobbles; and 3 percent Panoche Formation shale and sandstone pebbles.

The alluvial fan material from Ortigalita Creek (Mile 81.5 to 84) appear to be far too extensive to have been deposited by the present day creek, whose small flow is handled by a drain inlet into the canal. The large gravel deposits attributed to Ortigalita Creek, which incidentally have no surface expression in the immediate vicinity of the canal, were probably deposited by a much larger ancient creek with amore continuous flow.

All of the concrete aggregate for the concrete lining of the first 16.8 miles of the San Luis Canal was obtained from a gravel pit in the Ortigalita Creek gravel deposits near the Mervel Road Bridge (Mile 82.18). Here the gravel averaged about 15 percent non-plastic fines. The Ortigalita gravels are mostly silty gravel (GM) with some clayey gravel (GC) and poorly graded gravel (GP). The gravels average about 80 percent angular to subrounded Franciscan type pebbles mostly graywacke and chert, 16 percent rounded Tertiary volcanics; and 4 percent Panoche sandstone and shale.

From Mile 84 to Mile 87 the canal excavation cut mostly fine grained alluvium deposited by minor unnamed streams. Light brown sandy clay and clayey sand were most common. Scattered lenses of gravel, usually impervious clayey gravel, were small and discontinuous. The gravels were mostly of Franciscan type, chert pebbles and cobbles eroded from exposures of the Tulare Formation gravels in the Panache Hills to the west.

Due to the lack of continuous gravel layers to provide natural drainage in this area of fine grained soils, heavy irrigation over the past 30 years created a high ground water condition that required special treatment during the excavation of the canal between Mile 85 and 87. The treatment of ground water encountered in the canal excavation is discussed in USBR, 1971.

The environment of deposition in the area from Los Banos Creek to Dos Amigos Pumping Plant must have been, at least in recent times, conducive to the development of soil horizons. Well developed buried soil horizons are common throughout this section. Usually the buried soil horizons are composed of a reddish-brown sandy clay of medium to high plasticity containing numerous root holes and occasionally, filled animal burrows. These horizons are from 1-foot to 5 feet thick, usually about 3 feet; and, as many as 3 or 4 separate horizons occur in the 30-foot deep canal excavation. Some of the horizons are continuous for thousands of feet.

Mile 87 (Dos Amigos Pumping Plant) to Mile 95 - Invert Elevation 308.7 to 307 feet

At Dos Amigos Pumping Plant, water in the San Luis Canal is lifted a maximum of 125 feet where it begins a gravity flow toward Kettleman City. Since the canal alinement from this point was located along a higher contour, the excavation was virtually at the base of the hills through much of this 8-mile section. The invert elevation of 309 to 307 feet for this section of the excavation is about 117 feet higher than in the previous 16.8 miles.·

The type of material encountered in the excavation reflected the closeness of the canal to the foothills. Small channel fillings and lenses of unsorted clayey gravel were common cutting across beds of sandy clay, clayey sand and numerous soil horizons. All of the material was light brown color and very dry. The gravels were mostly Franciscan type chert pebbles and cobbles which had been eroded from exposures of the Tulare Formation in the Panache Hills. Fragments of white diatomaceous shale and gypsum from the Tertiary age Kreyenhagen Shale Formation were common in the gravel lenses.

The unconsolidated Quaternary deposits which are at great depths in the San Joaquin Valley sequence occur at very shallow depths along the hill front in this section of the canal. The Corcoran Clay member of the Tulare Formation is at a depth of about 100 feet at the Dos Amigos Pumping Plant, but occurs on the surface along the hills just west of the canal alinement. The Corcoran Clay was encountered in the canal excavation between Mile 91 and Mile 93 where the alinement of the canal swings to the west to avoid a topographic low along the hill front. The Corcoran in this section was more silty and sandy than in the area upstream described earlier where highly plastic clay was more typical.

In the vicinity of Mile 92, Tulare Formation gravels were exposed in the bottom of the canal excavation. These reddish colored clayey gravels underlie the Corcoran Clay beds for about 3,000 feet. A large number of well preserved vertebrate fossils were recovered from the section of the canal between Mile 91 and 93. The largest assemblage was recovered on the west bank of the canal at Mile 92. Fossils from this location were shipped to the University of California at Berkeley for identification. The assemblage included:

1. Two nearly complete mammoth skeletons (Mammoth Imperator)
2. An Antelope jaw section (Antelocaprid).
3. Horse teeth and other bones (Equs).
4. Camel teeth (Camelops).

This assemblage was located at the base of the Corcoran Clay in a rather sandy area. The section of the Corcoran exposed in this area probably represents a near shore deposit and the occurrence of several animals in one concentration suggests a boggy area along the shore of the ancient Lake Corcoran. The age of the Corcoran Clay indicated by this fossil assemblage is middle to late Pleistocence time.

Mile 95 to Mile 98 - Invert Elevation 307 to 306 feet

This 3-mile section of the canal was excavated through the alluvial fan material deposited by Little Panoche Creek. The material encountered was predominately gray to light brown silty gravel (GM) and clayey gravel (GC). In most areas, the gravels were overlain by 3 to10 feet of tan to brown sandy clay (CL) and sandy silt (ML). Buried, dark brown clay, soil horizons are common in the overlying sandy clay beds.

The concrete aggregate and sand for the concrete canal lining between Mile 87 and 108 was obtained from the excavation for the canal prism between Mile 96 and 97 and from a pit located within the canal right-of-way near Mile 96.5.

Mile 98 to Mile 104 - Invert Elevation 306 to 305 feet

Mile 98 marks the approximate beginning of the interfan sediments deposited by Little Panache Creek, Moreno Creek, and Big Panache Creek. The excavation through this 6-mile section encountered complexly interbedded deposits of sand and clay. Individual layers of material were usually 3 to 6 inches thick; sandy clay (CH), clayey sand (SC) and poorly graded sand (SP-SC) being most common. Scattered pockets and small channels of gravel and sand occurred in most areas. Occasional mudflows were noted. Mudflow material commonly contained mud balls, rounded gypsum pieces and dark brown shale fragments probably eroded from the Kreyenhagen Formation to the west. Several buried clay soil horizons were noted in the Little Panache and Moreno fans, but soil horizons were not well developed in the Panache fan material. Many intricate sedimentary structural features were exposed in the canal excavation through this interfan area. In addition to the complex interbedding of the sediments, scour and fill channels, buried mud cracks: and. vertical filled cracks were noted. The first occurrence of these filled cracks was at Station 1495+50. The characteristics and distribution of filled cracks is discussed further in USBR, 1971.

These interfan areas were known from the preconstruction investigations to be susceptible to hydrocompaction (subsidence caused by the application of water on the surface). Approximately 5 miles of the canal through this area was preconsolidated by ponding prior to construction. The method of preconsolidation used is discussed further in USBR, 1971.

Mile 104 to Mile 112 - Invert Elevation 305 to 304.3 feet

The canal in this 8-mile section was excavated across alluvial fan material deposited by Panache Creek. The Panache Creek fan is one of the largest fans crossed by the San Luis Canal. However, the material in the fan does not differ greatly from the material in the adjacent interfans. Mostly light brown sandy clay (CH) and clayey sand (SC), complexly interbedded, were encountered in the excavation. Significant layers of loose poorly graded sand (SP-SC) to 10 feet thick were encountered in addition to scour-fill channels of sand and fine gravel. Light and dark brown shale fragments and rounded gypsum pieces were common, all probably eroded from exposures of the Kreyenhagen

Formation to the west. Mudflows noted but not common. Buried soil horizons were rare and indistinct. A few reddish brown baked clay beds with carbonaceous fragments were found. Large deposits of gravel were not encountered in the Panoche Creek fan at canal excavation depths as they were in the alluvial fans of the streams in the first 30 miles of the excavation. However, preconstruction drilling indicated that large deposits of gravel do exist at depth in the Panache Creek fan, well below the bottom of the canal excavation.

Mile 112 to Mile 128.5 - Invert Elevation 304.3* to 298.7 feet

This section of the canal was excavated through fan and interfan material deposited by the small ephemeral streams emerging from Tumey Gulch, Arroyo Ciervo and Arroyo Hondo. These streams do not maintain flows beyond the hill front for more than a few days following periods of heavy rainfall. Thus they rarely add materials to their fans except for mudflow deposits. Most of the material in this area of numerous mudflow deposits was extremely dry, making the area especially susceptible to subsidence by hydrocompaction. Consequently about 15 miles of the section between Mile 112 and128.5 required preconsolidation by ponding prior to excavation for the canal. Three to five feet of settlement occurred in parts of this area, which was referred to as the southern subsidence area. The method of preconsolidation treatment is discussed further in USBR, 1971.

The material encountered in the excavation for the canal through this subsidence area was predominately complexly interbedded thin layers of light brown colored sand and clay. A few small gravel filled channels were noted. Ancient, mudflow deposits were particularly well shown in the canal prism through the Arroyo Ciervo and Arroyo Hondo fans (Mile 121 to Mile 128.5). Fat clay (CH) layers showing mud cracks, unsorted mudflow deposits, and occasional armored mud balls were noted at several localities occurring at various depths to 30 feet (canal invert).

During the preconstruction investigation conducted along the San Luis Canal several criteria were used in an effort to differentiate between subsiding and non-subsiding material. Criteria included natural soil properties such as dry density, liquid limit, plastic index and moisture content. During the geologic mapping of the canal excavation the search for criteria to identify subsideable soils was continued. Throughout the subsidence area, the material in the canal excavation was characterized by complexly interbedded deposits of clay and sand, with the thickness of individual beds seldom exceeding 6 inches. This interbedded character of the soil would make it very difficult to correctly define the soil properties of say a 5 or 10 foot interval, as was attempted during the preconstruction investigation. In the opinion of the writer, the material examined in the canal excavation through this subsidence area, would support the theory that the type of layered structure in the natural occurring material is one of the most important factors in the soils ability to compact upon wetting. In all areas of the excavation where subsidence had been induced by ponding, thin interbedded layers of clayey sand (SC) and fat clay (CH) were noted. If in any given area of interbedded deposits, the dry strength of the clay layers is high enough to permit successive loading by continued deposition of fan material, and the clay remains dry, "super loading" of the clay layers will result. To produce a deposit containing "super loaded clay layers", the amount of water being introduced onto the alluvial fan material by the depositing streams must be extremely low to prevent wetting of the clays layers that are being "super loaded" at depth.

Mile 128.5 to Mile 135 - Invert Elevation 298.7 to 297.2 feet

The Cantua Creek alluvial fan, which was cut by the canal excavation in this 6.5 mile section, is made up of light brown interbedded clayey sand (SC) and sandy clay (CL and CH) layers; but, unlike the extremely dry condition of the soil encountered in the previous interfan areas, the Cantua alluvial material was quite moist. The moist condition of the Cantua fan material was probably due to prolonged irrigation in the area, and by flooding from Cantua Creek which usually flows intermittently each year during the November to February rainy season.

This section of the canal was not susceptible to hydrocompaction, probably because of the high moisture content. Deep subsidence, however, was pronounced in this area requiring canal lining modification shortly after the canal became operational. The effects of deep subsidence and correction of these effects are discussed in Part III of USBR, 1971.

Mile 135 to Mile 143 - Invert Elevation 297.2 to 295.5 feet

This 6 mile section of the canal was excavated through the coalescing alluvial fans deposited by Salt Creek, Martinez Creek and Domongine Creek. The material encountered was mostly sandy clay (CL and CH) and clayey sand (SC). Gravel and sand channels were common but small and discontinuous. Moisture content in the soil throughout most of this section was high at time of excavation, and a perched groundwater table was encountered in the section between mile 138 and Mile 141. The occurrence of this perched groundwater was due to prolonged heavy irrigation in the area of tight clayey soil. The treatment of this area required during construction is discussed in USBR, 1971.

Mile 143 to Mile 164.7 - Invert Elevation 295. 5 to 291. 5 feet

Beginning at about Mile Post 143 the canal alinement swings to the east across the wide low alluvial fan deposited by Los Gatos Creek. The canal is about 10 miles from the hill front at Mile Post 155 where it swings more to the south, back toward the hill front.

The canal excavation through this 11.7 mile section encountered mostly sandy clay (CL) layers, although there were numerous layers of sandy material (SM and SC). Large sand filled channels were common across this broad fan. Gravel was rare; but, as in the Panache Creek and Cantua Creek fans, large gravel deposits were known from preconstruction drilling to exist below the canal excavation.

A high ground water table was encountered in the excavation near the end of this section that was probably due to past heavy irrigation.

Mile 164.7 to Mile 172.4 - Invert Elevation 291.5 to 288.1

This 8-mile section of the canal excavation was characterized by sandy material (SM and SC) deposited by 6 or 7 small ephemeral streams draining from the Kettleman Hills a short distance to the west. Fossil shell fragments eroded from the Tertiary marine formations to the west were common in the small sand and gravel channels. Clay and silt beds were present in the excavation, but much less common than in previous sections. Although the lower end of this section is very near the limit of the Tulare lakebed deposits, only alluvial fan material was encountered in the canal excavation.

USBR, 1971, Geology Construction Report, San Luis Canal, California Aqueduct, Mile 70 to Mile 172, San Luis Unit, West San Joaquin Division, Central Valley Project.

Frink, J.W. and Kues, H.A., 1954, Corcoran Clay - A Pleistocene Lacustrine Deposit in the San Joaquin Valley California. American Association of Petroleum Geologists Bulletin. Vol. 38, No. 11, pages 2357-2371.

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