CROWS LANDING INDUSTRIAL BUSINESS PARK

WATER SUPPLY (POTABLE & NON-POTABLE) INFRASTRUCTURE AND FACILITIES STUDY

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EXECUTIVE SUMMARY

The proposed Crows Landing Business Park Project (Project) will require the conversion of the decommissioned Crows Landing Air Facility (CLAF) and surrounding agricultural area to industrial use. Planning for this project requires an evaluation of viable water supply sources in the area. Changes in water demands due to land use conversions also need to be considered in order to accommodate various demand scenarios for both potable and non-potable water. Once the demands are determined the layout, alignment, and sizing of the water systems need to be designed to serve the Project area. The aim of this study was to establish viable supply sources, demands, and system layout and sizing of potable and non-potable water facilities for the Project.

Possible water sources for the project area include both surface water and groundwater. Surface water sources in the area include the Delta-Mendota Canal (DMC), the California Aqueduct (CAQ), and the San Joaquin River. To access these sources requires entitlements or rights which the Project does not have. Additionally, these allocations have been unreliable in recent years due to drought. The lack of entitlements and reliability of surface water have caused the Project to rely solely on groundwater as the water supply source. No surface water is planned to be used by the Project. Although there has been a decline in groundwater elevation in the project area, the groundwater has been deemed stable and a suitable water source for the Project based on the findings in *Groundwater Resources Impact Assessment: Crows Landing Industrial Business Park* prepared by Jacobson, James and Associates (Appendix A).

Water demands were determined by the Project's total developable acres and total water demand rate recommended by the Stanislaus County Department of Public Works (SCDPW). It is projected that 60% of this total demand will be for potable use and the remaining 40% for non-potable use. These demands must consider average day, maximum day, and peak hour demands as well as operational and emergency storage for the potable water system. In order to accommodate these demands, Phases 1A, 2, and 3 of the Project will require the installation of a well with wellhead treatment during each phase, for a total of four new potable water wells. Phases 1A and 2 will include the addition of water storage tanks. The non-potable system requires the installation of a well, storage tank, and booster pump during Phase 1B and fire hydrants for Phases 1, 2, and 3 in order to meet the area's irrigation and fire flow demands. The Project's potable water system consists of 12-inch PVC pipes, while the non-potable system includes 12- and 18-inch PVC pipes. The master planned water system for the Project and associated costs are presented in Table ES-1.

The most recent revisions to this study incorporate the findings of a water supply alternatives study to address the State Water Resources Control Board's Division of Drinking Water requirements for new water service areas to evaluate the feasibility of consolidation, annexation, or extension of water services. The study evaluated three alternatives: A) combine CLIBP with the Crows Landing Community Services District water system, B) pursue a new permit for CLIBP alone, and C) combine CLIBP with the City of Patterson water system.

Description	Cost
Phase 1A	
Potable Water System	$ 10,771,000.00
Non-Potable System	$ 2,213,000.00
Phase 1B	
Potable Water System	$ 3,275,000.00
Non-Potable System	$ 9,283,000.00
Total for Phase 1	**$ 25,542,000.00**
Phase 2	
Potable Water System	$ 12,180,000.00
Non-Potable Water System	$ 3,735,000.00
Total for Phase 2	**$ 15,915,000.00**
Phase 3	
Potable Water System	$ 8,597,000.00
Non-Potable Water System	$ 2,981,000.00
Total for Phase 3	**$ 11,578,000.00**
Total Estimated Opinion of Probable Construction Cost	**$ 53,035,000.00**

1.0 INTRODUCTION

Section 1 states the study background and purposes, Study Area, and overall system planning assumptions.

1.1 STUDY BACKGROUND

The Crows Landing Industrial Business Park project (Project) is an approximately 1,528-acre conceptually planned development that encompasses the reuse of the former Crows Landing Air Facility, which was decommissioned by NASA in the late 1990s.

The Project site lies west of State Route 33 and east of Interstate 5, southwest of Patterson, and approximately 1 mile west of the unincorporated community of Crows Landing. The Project site is further bounded on the east by Bell Road, on the south by Fink Road, on the west by Davis Road, and on the north by Marshall Road and State Route 33. The Delta-Mendota Canal traverses the southern portion of the Project in a northwest/southeast direction. Little Salado Creek enters the Project site along the western property boundary slightly northeast of the Delta-Mendota Canal, and discharges to the Marshall Drain. The Marshall Drain then transitions to an underground pipe near the intersection of Marshall Road and State Route 33. The Project site topography generally slopes down in a northeasterly direction with an elevation change of approximately 80 feet, with the lowest elevation near the intersection of State Route 33 and Marshall Road. The Project site falls within the Del Puerto Water District. The site includes vehicle and aviation improvements associated with the former air facility and approximately 1,200 acres which are currently being used for agricultural purposes. Figure 1.1 provides a project layout and phasing plan for the development.

The Study Area includes the Project site, the Western Hills Water District water treatment plant, and large surface water storage and transport systems in the area including the California Aqueduct and the Delta Mendota Canal, and groundwater within the San Joaquin River Hydrologic Region.

1.2 STUDY PURPOSE

This Water System (Potable and Non-Potable) Infrastructure and Facilities Study provides information required for the County to better assess the feasibility of the planned development by defining the necessary potable and non-potable water system infrastructure improvements. The scope of this plan includes the following major tasks:

- Discuss alternative potable and non-potable water supply sources and treatment considerations
- Determine the projected potable and non-potable water demands for the Project, based on the proposed land uses
- Determine the overall preliminary potable and non-potable water system layout and sizing, using the Land Use Plan and the Circulation Plan as a guide for preliminary alignments and locations.

The findings of this study are based on available information and are subject to change once more detailed engineering analyses are performed as the Project progresses.

1.3 OVERALL SYSTEM PLANNING ASSUMPTIONS

Stanislaus County Department of Public Works Standards and Specifications Section 6.4 states:

> The water system shall conform to the requirements of the water district in which the development is located. The governing water district shall sign the improvement plans prior to the plans being approved by the County. If the development is located outside of a water district, then the water system shall be designed and constructed in conformance with the City of Modesto water standards. Compliance with the applicable water standards shall be certified by the design engineer.

Overall planning assumptions for the water system in this study are determined based on a comparative analysis of water duties for local cities and agencies, including the *City of Modesto Standard Specifications 2014* (COM 2014). In the case where design guidelines and criteria are not published by a local agency, assumptions are made based on typical values published in the *Water Distribution System Handbook* (Mays, 2000).

2.0 BACKGROUND INVESTIGATION

2.1 TOPOGRAPHY

The Crows Landing Industrial Business Park project (Project) site terrain is composed of gently sloping land. Terrain in the Project site rises from about 120 feet above sea level in the northeastern corner of the development, near the Marshall Road / State Route 33 intersection, to around 200 feet above sea level at the southwestern corner of the development immediately north of Fink Road.

2.2 EXISTING CONDITIONS AT CROWS LANDING AIR FACILITY

The Crows Landing Air Facility (CLAF) was commissioned as an auxiliary airfield to Naval Auxiliary Air Station Alameda in 1942 and decommissioned by the National Aeronautics and Space Administration (NASA) as the Crows Landing Flight Facility/NASA Ames Research Center in 1999 In the same year, the U.S. Congress passed Public Law 106-82 which directed NASA to convey the CLAF to Stanislaus County in several phases following environmental remediation activities. In 2004, NASA conveyed 1,352 acres of the CLAF, known as Parcel A, to the County. One hundred seventy-six (176) acres remain to be conveyed to the County once environmental remediation activities have been completed. Currently, the land is being used for agricultural purposes. Historically, as much as approximately 1,200 acres have been used for agricultural production, but the total amount of land in production has varied greatly due to water availability.

2.3 EXISTING WATER SOURCES AND FACILITIES

Existing water sources within and near the Project site include both natural and man-made surface water conveyance facilities and groundwater wells as described in this section.

2.3.1 Surface Water Background

The term “surface water” refers to water from natural precipitation which is made available through natural or man-made bodies of water such as canals, lakes, reservoirs or rivers. Surface water sources in the vicinity of the Project site include the Delta-Mendota Canal (DMC), the California Aqueduct (CAQ), and the San Joaquin River. The ephemeral Salado Creek and the Little Salado Creek periodically contain flows in the Project vicinity, but do not contain flows year-round. Therefore, these will not be considered as potential sources for potable nor non-potable use for the project.

2.3.1.1 Delta-Mendota Canal

A portion of the DMC crosses the project site. The DMC, completed in 1951, is part of the federal Central Valley Project (CVP) and is operated by the United States Department of the Interior Bureau of Reclamation (USBR) and the San Luis Delta Mendota Water Authority (SLDMWA). The DMC carries water pumped from the Sacramento-San Joaquin Delta (Delta) southeasterly along the west side of the San Joaquin Valley for agricultural as well as Municipal and Industrial (M&I) uses, for use in the San Luis Unit, for replacement of San Joaquin River water stored at Friant Dam, and for use in the Friant-Kern and Madera systems.

Water from the DMC is primarily used for irrigation, though it is also used for M&I purposes, as well as environmental purposes for fish & wildlife and habitat restoration. In an average year, deliveries of CVP water total approximately 7 million acre-feet per year (AFY), with 5 million AFY of that total used for agricultural purposes, 600,000 AFY for M&I uses, and 1.2 million AFY used for environmental purposes including fish & wildlife habit. (USBR, M&I, 2014). Deliveries from the CVP are determined by the USBR on an annual basis through allocations, or portions, of contracted water amounts. During years with wetter conditions and more plentiful supplies, allocations are higher. Conversely, during dryer years, allocations are reduced.

Most CVP contractors have provisions in their contracts allowing the use of CVP water for M&I purposes, though these contractors may also have their own regulations prohibiting the use of this water for M&I purposes. Water allocations for M&I purposes are governed by the USBR's Municipal and Industrial Water Shortage Policy, which was most recently finalized in 2005. (USBR, M&I, 2005). This current policy assigns a higher priority to M&I allocations than agricultural allocations. For example, in early 2014, CVP water allocations were estimated to be 0% of contracted amounts for agricultural purposes due to the dry conditions for that year. By comparison, allocations for M&I uses were much higher at up to 50% of contracted amounts. The USBR has recently issued a draft environmental impact statement which considers alternatives to the current policy. (USBR, M&I, 2014). This draft environmental impact statement describes five alternatives for revisions to the current policy:

Alternative 1: "No Action", resulting in no change to the current policy

Alternative 2: Equal Agriculture and M&I allocations

Alternative 3: Full M&I Allocation, regardless of agricultural allocation

Alternative 4: Updated M&I Water Service Policy –similar to "No Action" alternative, though with some revisions to determination of historic use

Alternative 5: M&I Contractor Suggested Water Service Policy –similar to Alternative 4, though with modifications to CVP operational practices to meet public health & safety requirements.

As of this report, a preferred alternative has not been identified.

All waters conveyed by and stored within the CVP are fully appropriated; therefore, surface water must be obtained from permitted users that are willing to sell portions of their entitlements. Existing water districts near the Project and their corresponding CVP water contract entitlements include: the Del Puerto Water District, with a contract entitlement of 140,210 acre-feet per year; Patterson Irrigation District with a contract entitlement of 22,500 acre-feet per year; and West Stanislaus Irrigation District with a contract entitlement of 50,000 acre-feet per year (USBR, Contractors List, 2014).

2.3.1.2 California Aqueduct

The Project site is approximately 2.5 miles from the California Aqueduct (CAQ). The CAQ is part of the State Water Project (SWP), the nation's largest state-built water conveyance and power development system, operated by the California Department of Water Resources (DWR). Water from the Delta is pumped by the Harvey O. Banks Pumping Plant (Banks Pumping Plant) into the 440-mile long concrete-lined canal which ultimately delivers water to millions of California residents and farms. Water from the

SWP is mostly used for M&I purposes, though a substantial portion is also used for agricultural, as well as environmental purposes.

Water deliveries from the SWP are based on long-term contracts between the DWR and 29 public agencies and water districts (contractors) throughout the state. These contracts specify the maximum amount of water a contractor may request each year from the SWP, commonly referred to as "Table A" water. Table A allocations to each contractor for a given year are traditionally determined by the DWR near the end of the preceding year based on predicted supplies, though the allocations can be adjusted later during the year. As an example, 2014 water allocations from the SWP were initially predicted in November 2013 to be 5% of Table A amounts, though were later reduced to 0% in January 2014 due to historic dry conditions. However, later in 2014, allocations were increased back to 5% of Table A amounts in April 2014, and then further increased to 20% in May 2014 due to late precipitation and successful conservation efforts. (DWR Notices, 2013-2014).

Two local SWP water contractors near the Project site receive water from the CAQ: the Oak Flat Water District (OFWD) and the Western Hills Water District (WHWD). OFWD has a Table A contract amount of 5,700 acre-feet of water for agricultural use. (DWR, 2014) WHWD supplies water to the Diablo Grande master-planned development, located west of the Project. WHWD receives water from the CAQ through an agreement with the Kern County Water Agency, which allows for deliveries of up to 8,000 acre-feet of water from the CAQ. (DWR, 2004). WHWD treats the raw water from the CAQ with an existing 1 MGD conventional treatment plant, Public Water System No. 5010039. The Diablo Grande development currently utilizes water from the CAQ for all of its normal water needs, though groundwater from WHWD sources may be utilized in the event of catastrophic emergencies.

2.3.1.3 San Joaquin River

The Project site lies approximately 4 miles west of the San Joaquin River. In the vicinity of the Project, the San Joaquin River runs in a northwesterly direction, roughly parallel to the Coast Range Mountains, draining towards the Sacramento-San Joaquin Delta, which ultimately discharges to the San Francisco bay. The upstream hydrology of the river's watershed has been highly modified by man since the early 20th century.

The State has prohibited use of the river as a drinking water source; however, some water is pumped for non-potable irrigation uses by local irrigation districts. (City of Patterson, UWMP 2010) The City of Patterson further discusses the possibility for use of San Joaquin River flows to meet non-potable demands, as well as for groundwater recharge purposes in its 2010 General Plan Update Water Supply Assessment. (2010 City of Patterson General Plan WSA). This possibility is based on the "senior" pre-1914 water rights of the Patterson Irrigation District which allows the full use of their allotted water flows of 340 ac-ft/day without restrictions. In addition, the Westside Irrigation District may utilize up to 545 ac-ft/day, but is subject to restrictions as it is a "junior" post-1914 water rights holder. However, restrictions had not been placed on the Westside Irrigation District in the 30 years prior to that report. (City of Patterson, UWMP 2010) In addition, both the WSID and PID have current "Warren Act" contracts with the USBR that allow water obtained from the San Joaquin River to be conveyed and stored within the DMC and other downstream CVP facilities. PID's contract was recently renewed, though WSID's will expire in 2015, and has recently been circulated for environmental approval for renewal. (USBR, Warren Act, 2014 & 2015)

Although PID and WSID have long-standing rights to San Joaquin River, these districts are still subject to curtailments of water diversions from the river in times of drought or limited supply. These curtailments, implemented by the State Water Resources Control Board (SWRCB), can apply to all water right holders, including those with pre-1914 senior water rights. On January 17, 2014, the SWRCB issued an informational notice to all water right holders throughout the state warning of potential curtailments due to dry conditions. This notice was followed by a May 27, 2014 curtailment notice to all water right holders within the Sacramento and San Joaquin River watersheds. (SWRCB 2014) The notice directed the immediate curtailment of all diversions for “junior” post-1914 water right holders, except under certain conditions. In addition, the notice warned of potential curtailments even for “senior” pre-1914 water right holders, which are typically not subject to limitations on diversions. These curtailments were eventually relieved later in 2014. In 2015, water rights were curtailed due to unprecedented drought conditions.

2.3.1.4 Surface Water Reliability

The reliability of surface water as a source of supply depends on a combination of factors, including the availability of source water based on seasonal rainfall and storage, the condition of the conveyance facilities, and competing demands. In addition, both the DMC and CAQ have been subject to shutdowns due to emergencies, due to scheduled maintenance and repairs, and due to environmental concerns. As previously discussed, water deliveries from the DMC and CAQ have been severely reduced in recent times from contracted amounts due to drought conditions. Furthermore, diversions of surface water from natural courses may be curtailed by the SWRCB due to drought conditions, even for pre-1914 water rights holders.

The reliability of surface water as a supply may be improved by constructing additional improvements to the system. As an example, The Delta-Mendota Canal/California Aqueduct Intertie project (Intertie) was completed in 2012 as a means to eliminate the DMC conveyance conditions caused by a restricted Tracy Pumping Plant capacity. Modeling studies indicate that the Intertie project will enable the CVP to deliver a long-term average of 35,000 acre-feet per year of additional water to its service area by enabling the CVP to use available capacity in the CAQ (USBR Record of Decision, 2009). Surface water reliability may also be extended through creative banking agreements or other transfers of water rights. The WHWD successfully “banked” 2000 acre-feet of water from 2010 to 2013 through agreements for transfers with other irrigation districts. This water was then available for delivery from the CAQ to the WHWD, despite the fact that SWP allocations were initially limited to 0% at the beginning of 2014. (Diablo Grande 2014)

Still, efforts to improve or extend the reliability of surface water deliveries from the DMC and CAQ are costly. For example, the Intertie project did provide some improvements in operational flexibility of DMC and CAQ, though it provided only modest increases in water delivery for the price. The total construction cost of the Intertie project was approximately $28M, yet only achieved an estimated average of 35,000 acre-feet per year of additional CVP water deliveries, or about 0.5% of the total annual delivery of 7 million acre-feet. Similarly, water delivered through banking agreements or transfers from other contractors often must be purchased at a premium price. Furthermore, such agreements or transfers may also be temporary, with no guarantee of future renewal.

2.3.2 Groundwater Background

The term "groundwater" refers to water held underground in naturally occurring aquifers which must be extracted through the use of wells. The majority of the area surrounding the Project site is heavily reliant on groundwater as a water supply source for agricultural and urban use.

The Project site is located within the Delta-Mendota Sub-Basin, a portion of the San Joaquin River Hydrologic Region. The geological characteristics of the groundwater basin consist of the Tulare formation, terrace deposits, alluvium, and flood-basin deposits. Regionally, the upper water bearing zone and lower water bearing zone are separated by the Corcoran clay layer, a relatively impermeable layer lying between 220 ft. to 300 ft. below the ground surface of the project site.

2.3.2.1 Existing Groundwater Facilities

Groundwater wells are heavily relied-upon throughout the vicinity of the Project site for potable and non-potable uses. The nearby City of Patterson and community of Crows Landing both rely exclusively on groundwater wells to meet potable water demands. There are four existing active wells onsite, though details regarding their construction (e.g. type of screens, depths) are unknown. Although the *Groundwater Resources Impact Assessment* prepared by Jacobson James & Associates, Inc. (Appendix A) confirmed that there are adequate groundwater supplies available for the Project, the exact supply capacity of the existing wells cannot be determined without further study. In addition, an existing 16-inch water transmission main delivers emergency water from a groundwater well, located along Davis Road west of the northern area of the Project, to the Diablo Grande development located approximately 7 miles west of the Project. The distribution main alignment continues north from the well site then runs west along West Marshall Road, south along Ward Road, and west along Oak Flat Road.

2.3.2.2 Groundwater Level Trends

Several previous studies over the last several years have summarized measurements of groundwater elevations in the vicinity of the Project site. The results of these studies can be used to ascertain the sensitivity of the aquifer to periods of drought, and the sustainability of groundwater as a water source. The results of these studies suggest that, over time, the groundwater levels at the Project site and in the vicinity are stable. This would imply that groundwater resources in the vicinity of the Project site are not in an overdraft condition. However, as part of the Sustainable Groundwater Management Act, the State of California Department of Water Resources (DWR) recently released a final list of California groundwater basins that are in a state of critical overdraft (http://water.ca.gov/groundwater/sgm/index.cfm, accessed 2/15/2016). This final list includes the Delta-Mendota groundwater basin and therefore includes the Project site. While the area near the Project site may appear to have stable groundwater levels, the basin as a whole shows areas of declining water elevations. This section describes the findings and results obtained from pertinent studies of local groundwater elevations.

2.3.2.2.1 California Department of Water Resources, Bulletin 118

The California Department of Water Resources (DWR) originally published Bulletin 118 in 1975, which provided a characterization of 248 of 461 identified groundwater basins. Bulletin 118 was updated most recently in 2003, though errata have been published more recently. As a supplement to Bulletin 118, the DWR publishes a separate description for each sub-basin within the 10 hydrologic regions. The description for the Delta-Mendota Sub-basin, updated in 2006, indicates the general trend for the groundwater level in the sub-basin showed an increase from 1970 through 1985, consistent with increased surface water deliveries to the San Joaquin Valley, with a maximum groundwater level of 7.5 feet above the 1970 water level. From 1985 through 1994 groundwater levels declined. The groundwater level in 1994 was similar to the 1970 groundwater level. In 1995 the groundwater level rose to 2.2 feet above the 1970 level. Since 1995, groundwater levels have fluctuated around 2.2 feet above the 1970 water level until 2000.

Based on a specific yield of 11.8 percent, the Delta-Mendota Sub-Basin has a total of 26,600,000 acre-feet of groundwater stored to a depth of 300 feet as of 1995. The total storage capacity is estimated to be 30,400,000 acre-feet to a depth of 300 feet and 81,800,000 acre-feet to the base of fresh groundwater (DWR, 2003).

2.3.2.2.2 San Luis Delta Mendota Water Authority, Groundwater Management Plan

In 1995, the agencies comprising the SLDMWA entered into an agreement to jointly fund the preparation of a coordinated regional groundwater management plan (GMP). The groundwater management area (GMA) covered by the GMP includes portions of the Tracy and Delta-Mendota sub-basins of the San Joaquin River hydrologic region, and fully encompasses the Project site. This GMP was most recently updated in 2011.

The study includes an analysis of groundwater level trends in the GMA between 1993 and 2008. Findings of the study characterize the groundwater levels in the GMA as generally hydrologically balanced. The study further indicates minimal apparent net change in groundwater level elevations over the study period, which seem to indicate equilibrium in the GMA between use and recharge. The study does describe consistent declines in elevation for certain localized areas of the GMA, such as areas west of Newman, which could be indicative of a developing local overdraft condition. As noted previously, DWR has listed the Delta-Mendota groundwater basin, which includes areas within the GMA and includes the Project site, as being in a state of overdraft (http://water.ca.gov/groundwater/sgm/index.cfm, accessed 2/15/2016).

Regarding the Project site, the study indicates some decline in groundwater elevation of approximately up to 8 ft. between 1998 and 2008. However, the study also indicates an overall increase in groundwater elevations from 1993 to 2008 in the area of the Project site of up to approximately 8 ft. This groundwater elevation data would suggest that, over time, the groundwater in the area of the Project site has been in a hydrologically balanced condition.

2.3.2.2.3 City of Patterson

Owing to COP's proximity to the project within the Delta-Mendota Subbasin and the fact that it's deep municipal wells are similar to those which could serve the Project, the existing groundwater well data and information for the COP is considered representative of the deep aquifer groundwater conditions at the Project site. A recent review of groundwater within and around the COP is included in *the Supplement to Water Supply Assessment for Arambel Business Park/KDN Retail* Center, prepared by Kenneth D. Schmidt and Associates (KSA) in 2013. The study indicates that between 1990 and 2012 water levels for wells tapping the upper aquifer above the Corcoran Clay Layer were relatively stable within the study area, though with an average decline of 0.3 feet per year. This decline was attributed to a number of dry years during this period. The study also discusses water levels within the lower aquifer below the Corcoran Clay layer. Manual depth measurements were taken of 4 different wells which tapped the lower aquifer within the City of Patterson between 2006 and May 2013. These measurements did not indicate any decline in water levels during this period.

2.3.2.2.4 Former NASA Crows Landing Flight Facility, Groundwater Monitoring

Groundwater monitoring studies focusing on the Project site and the area in the immediate vicinity of the Project site are ongoing as part of Navy's Base Realignment and Closure Program. The most recent study available for analysis was prepared in June 2014, and provides the results of groundwater levels in monitoring wells from April 2013 through February 2014 at the Project site. This study indicates that groundwater levels within the upper and shallow water-bearing zones have declined by an average of 3.72 feet in the shallow water-bearing zone; and by 7.25 feet in the deeper water bearing zone directly above the Corcoran clay layer, compared to the groundwater levels measured in February 2013 (OTIE, 2014).

2.3.2.2.5 California Statewide Groundwater Elevation Monitoring (CASGEM)

The DWR established the CASGEM program to provide statewide monitoring of groundwater in response to Senate Bill X7 6, which added provisions requiring groundwater monitoring to the Water Code. CASGEM maintains an online system with available well details and groundwater information at numerous locations throughout the state. (http://www.water.ca.gov/groundwater/casgem/)

CASGEM includes detailed information for recent groundwater elevations for 4 wells within the project area:

- Local Well ID MP45.78R: An existing irrigation well just east of Davis Road, approximately one mile north of the Fink Road / Davis Road intersection. CASGEM records indicate this well has a total depth of 721 feet. Water level measurements indicate a drop of approximately 14 ft between March 15, 2012 and March 14, 2014.
- Local Well ID's P259-1, P259-2 and P259-3: Three monitoring wells with total depths of 430 ft, 255 ft, and 115 ft, respectively. Water level measurements indicate a drop of approximately 6 to 8 ft. in all three wells between November 16, 2011 and December 22, 2014.

Measurements from these wells indicate a modest decline in groundwater elevation data between 2011 and 2014; however, earlier groundwater elevation data is not available for these wells through CASGEM. Additionally, 2013 and 2014 were abnormally dry years which is likely the reason for the decline in groundwater levels during this period. Given the relatively short monitoring period and the abnormally dry conditions during this period, the water level information does not necessarily indicate an overdraft condition, despite the decline between 2011 and 2014. Copies of information obtained from CASGEM for these wells are included in the Appendices.

2.3.2.2.6 Groundwater Resources Impact Assessment

Groundwater Resources Impact Assessment prepared by Jacobson James & Associates, Inc. (Appendix A) assesses the groundwater resources present at the Project site and the impact of groundwater pumping. The site has a shallow unconfined aquifer as well as a deeper, confined aquifer separated a relatively impermeable regional aquitard layer referred to as the Corcoran Clay. The Project potable water supply will be developed using new wells installed into the confined aquifer beneath the site. The Project will develop a non-potable water supply using a combination of the existing irrigation wells that derive water from both the shallow and deep aquifer (assumed to provide 834 acre-feet per year based on historical pumping rates), and new non-potable supply wells installed into the shallow aquifer beneath the site to meet non-potable Project water demand in excess of what is provided by the existing irrigation wells.

Recent reductions in surface water deliveries due to drought have caused increased pumping of groundwater in the area. Coupled with low levels of precipitation, the aquifer has been classified as being in a state of overdraft. This area and the area northwest of the site have experienced pronounced cones of depression in the fall. Despite these low groundwater levels, the aquifer is stable as indicated by consistent water elevations by season.

The Groundwater Resources Impact Assessment (JJ&A, 2016) evaluated the on-site groundwater withdrawals to determine the potential impacts on on-site pumping on the local groundwater conditions. The results showed that groundwater withdrawals to support the Project could result in a drawdown within 1% to 10% of the total available saturated thickness of the aquifer, thus having a minimal impact on the groundwater storage conditions. The study also indicated that the potential for limited land subsidence does exist due to pumping from the confined aquifer beneath the Project site; however, there are no other impacts to any groundwater dependent ecosystems or water quality. Ongoing groundwater monitoring is recommended to regularly evaluate groundwater conditions to prevent adverse impacts in the future

2.3.2.3 Groundwater Quality and Constituents

Groundwater obtained from the region's aquifers has been known to contain constituents, such as iron, manganese, arsenic, nitrates and nitrites, and other inorganic and organic compounds.

According to monitoring reports taken from the SWRCB website (https://sdwis.waterboards.ca.gov/PDWW/ accessed 2/15/16), groundwater in the surrounding area, specifically the Crows Landing Community Services District (CLCSD) area, has been found to contain several contaminants that exceed the state maximum contaminant level (MCL). If the Project intends to source its water supply from the same aquifers, wellhead treatment systems may be necessary. These contaminants include:

- Nitrate
- Nitrite
- Hexavalent chromium
- 1,2,3-trichloropropane

Nitrate and nitrite (as nitrate + nitrite) has been detected as high 5,424 milligrams per liter (mg/L) in 2012. Hexavalent chromium has been detected as high as 15 micrograms/liter (µg/L) in 2014. 1,2,3-trichloropropane has been detected as high as 0.5 µg/L in 2009. It is not certain that the groundwater within the Project area contains the same contaminants as the groundwater utilized by the CLCSD. Comprehensive water quality samples of groundwater pumped from wells which are located nearby the study area as well as from test and production wells onsite would need to be evaluated to more fully ascertain the constituent, which may be expected in supplies pumped from new wells and the required methods for treatment.

In addition to the CLCSD data, initial test results of groundwater obtained from wells in the region were obtained from the SWRCB, formerly known as California Department of Public Health (CDPH) and constituents of concern were noted for various regions. For this project, the data for the COP and Newman are listed in Table 2.1 (CDPH 2014).

Table 2.1 – Groundwater Constituents of Concern for Crows Landing, COP, and Newman

Constituent	Result Range	MCL	Units
Alkalinity (Total) as CaCO3	78-381.1	-	mg/l
Aluminum	30-120	1000	µg/l
Arsenic	2-7.2	10	µg/l
Barium	15.5-560	1000	µg/l
Bicarbonate Alkalinity	78-407.4	-	mg/l
Boron	300-600	-	µg/l
Bromodichloromethane (THM)	0.5-0.9	-	µg/l
Bromoform (THM)	0.7-12.3	-	µg/l
Calcium	47-142.4	-	mg/l
Chloride	32-2,100	500	mg/l
Chloroform (THM)	0.9-1.49	-	µg/l
Chromium (Total)	5.8-29.3	50	µg/l
Chromium (Hexavalent)	3.5-36	10	µg/l
Color	3-9	15	units
Copper	4-53	1000	µg/l
Dibromoacetic Acid (DBAA)	1-2	-	µg/l
Dibromochloromethane (THM)	0.5-1.2	-	µg/l
Dibromochloropropane (DBCP)	0.01-0.04	0.2	µg/l
Dichloroacetic Acid (DCAA)	19.9	-	µg/l
Fluoride (natural source)	0.1-0.4	2	mg/l
Gross Alpha	1.26-12.1	15	pci/l
Gross Alpha MDA95	1.09-3.09	-	pci/l
Haloacetic Acids (5) (HAA5)	2-49.3	60	µg/l
Hardness (Total) as CACO3	237-1901	-	mg/l

Constituent	Result Range	MCL	Units
Iron	22-300	300	µg/l
Lead	0.3	-	µg/l
Magnesium	4.2-130	-	mg/l
Manganese	10-19	50	µg/l
Mercury	0.03-32.2	2	µg/l
Monobromoacetic Acid (MBAA)	1.1-1.8	-	µg/l
Monochloroacetic Acid (MCAA)	3.3-5.7	-	µg/l
Nickel	1-17	100	µg/l
Nitrate (as NO3)	0.5-92	45	mg/l
Nitrate + Nitrite (as N)	565-12000	10000	µg/l
Nitrite (as N)	900	1000	µg/l
Perchlorate	3.5	6	µg/l
Ph (Laboratory)	6.1-8.2	-	
Potassium	2.1-4.7	-	mg/l
RA-226 for CWS or Total RA for NTNC by 903.0	0.039	-	pci/l
RA-226 or Total RA by 903.0 C.E.	0.174-0.232	-	pci/l
Radium-228	0.009	-	pci/l
Radium-228 MDA95	0.286-0.319	-	pci/l
Radium, Total, MDA95-NTNC Only, by 903.0	0.366	-	pci/l
Selenium	3-10	50	µg/l
Sodium	58-350	-	mg/l
Specific Conductance	640-5700	1600	us
Sulfate	18-688	600	mg/l
Tetrachloroethylene	2.5-13.1	5	µg/l
Total Dissolved Solids (TDS)	460-4100	1000	mg/l
Total Trihalomethanes	1.8-15.5	80	µg/l
Trichloroacetic Acid (TCAA)	2-26.1	-	µg/l
Turbidity (Laboratory)	0.05-2.9	5	ntu
Uranium	1.31-12	20	pci/l

Constituent	Result Range	MCL	Units
Uranium MDA95	0.409-0.475	-	pci/l
Vanadium	4-10	-	µg/l
Zinc	20-260	5000	µg/l

Note: Other constituents that are not included in the preliminary data obtained from CDPH are also regulated and will need to be evaluated.

High iron and manganese levels were observed in many wells in the City of Modesto and COP, and in some remote cases, aluminum was also found above MCLs. Arsenic, a metal, was also found in some wells in the COP, Newman, Modesto, and other valley regions, which is an expensive compound to treat. The MCL for arsenic has recently been lowered to 10 µg/l. If arsenic is prevalent, many purveyors prefer to seek new well sites or alternative well construction methods rather than to treat for this contaminant. Additionally, TDS was found to be elevated and over the MCL in some wells. TDS reduction can be very expensive and can sometimes require the use of reverse osmosis (RO) and/or blending to achieve allowable levels.

Some organic compounds have been found in COP and Newman wells, which can be a concern and is largely dependent on sources of contamination relative to the well and plume migration patterns. These data show that dichlorobenzene, low levels of dibromochloropropane (DBCP), and low levels of tetrachloroethylene have been identified, but of these, only dichlorobenzene was over the MCL.

Nitrates, odor, and high color have been observed in numerous wells in the City of Modesto and the COP. Elevated chloride levels were observed in some COP and Tracy wells. High alpha and uranium have also been observed in some regions south of City of Modesto.

2.3.2.4 Groundwater Remediation Efforts at CLAF

The Navy currently maintains a 2,000 foot pumping restriction around a contamination plume within the Project site known as the Installation Restoration Program (IRP) Site 17 Administration Area Plume. The contamination plume includes benzene and other volatile organic compounds, and is a result of underground fuel storage tanks serving the former facility. Contaminants from this plume appear to be limited to the upper aquifer above the Corcoran Clay layer. The Department of the Navy is currently conducting a program of enhanced bioremediation, including monitored natural attenuation and carbon substrate to remediate the contamination. This program has successfully reduced contamination levels, and will continue to be monitored by the Navy. The pumping restriction will remain in effect until remediation efforts have been completed (CH2M Hill Kleinfelder, A Joint Venture [KCH], 2014).

3.0 WATER SUPPLY ALTERNATIVES

Alternatives for meeting the water demands of the Crows Landing Industrial Business Park (Project) may include the use of surface water, the use of groundwater, or a Conjunctive Use of both surface water and groundwater.

3.1 SURFACE WATER

As discussed previously, a portion of the DMC traverses the Project site, and the CAQ is near the Project site. These two canals would be the only nearby sources of potable surface water for the Project, as other naturally occurring surface water bodies in the Project vicinity lack the quantity or quality to be a feasible potable water source. The San Joaquin River is not considered a potential direct source of potable or non-potable surface water for the Project due to the distance of the river from the Project site, quality of the river water, and limited available surface water rights. Therefore, surface water will not be considered as a water supply source for this project.

3.1.1 Surface Water Entitlements

The County currently does not have any surface water entitlements or rights for the Project from natural sources, nor from the DMC or CAQ. Accordingly, use of the CAQ or DMC as a water supply source would require acquisition of existing water rights, entitlements, or water transfer agreements.

Delta-Mendota Canal: Owing to severely reduced surface water deliveries in recent years, no apparent opportunities exist at this time for the exchange of surface water from the DMC with most local water agencies as they continue working to secure adequate supplies for existing customers. Use of water from the DMC may also require approval by the USBR to allow for a conversion from agricultural use to M&I uses.

California Aqueduct: As discussed previously, The Diablo Grande community, a portion of the WHWD, obtains all of its normal water use from the CAQ. This water is pumped from the CAQ, and then treated at WHWD's 1- MGD treatment plant. The planned "Phase 1 Expansion" of the WHWD treatment plant will increase treatment capacity of the plant to 2 MGD. A portion of these improvements have been completed; however, the full completion of the Phase I Expansion project has been on hold since 2006.

3.2 GROUNDWATER SUPPLY

Groundwater would be used in this option to meet the water demands of the Project. The use of groundwater would likely involve treatment to remove the known constituents in the region's aquifers. Initially, well head treatment would address water quality requirements.

As discussed in section 2.3.2.2, groundwater resources in the Project area are not in an overdraft condition, and groundwater can be a viable source of water supply to the Project.

3.2.1 Groundwater Treatment

Overall, many well contaminants are region-specific, and many of the constituents listed in Section 2.3.2.3 can be successfully reduced through the appropriate treatment methods, although the costs for each treatment method can vary widely.

Groundwater pumped from some nearby wells in the COP, Newman, and Modesto requires treatment and/or blending. It is likely that new municipal groundwater wells in the project would also require treatment to reduce constituents under the MCLs.

It is also possible that an effective process to drill and case new wells could substantially reduce the need for treatment of certain contaminants by avoiding the lenses whereby these compounds are concentrated.

Many metals, such as Iron and Manganese, can be cost-effectively treated by an oxidation/precipitative process. Although more costly to treat, arsenic can sometimes be treated with this process as well and/or through the use of adsorption or ion exchange resins or filter media. Nitrates can be removed by utilizing ion exchange or reverse osmosis. Taste, color, and odor may be greatly improved through the use of GAC filters, and this process can also reduce level of certain organic compounds.

The cost-effectiveness of well head treatment is typically based on the levels and type of treatment processes needed. Adsorptive types of processes may be cost-effectively applied at the well head. However, more complex treatment methods dealing with the removal of arsenic, nitrates, and volatile organic compounds (VOCs), and those requiring chemical oxidants, may be more costly to treat at the well head.

The cost of the treatment processes will be dependent on the types and amounts of the constituents to be removed, which can be further evaluated through a comprehensive test well drilling and sampling program, prior to drilling production wells.

3.2.2 Groundwater Regulations

Prior to 2014, extracting or pumping groundwater in California required no rights or entitlements from the State or any Federal agency. Permits were, and still are, required for the design and construction of groundwater facilities to prevent contamination. However, in regards to groundwater quantity, no approval was previously necessary that would limit the quantity of groundwater extracted from a well or wellfield.

This “pump as you please” policy has recently changed with the passage of new State legislation. SB1168, SB1319 and AB1739, otherwise known as the Sustainable Groundwater Management Act (SGMA), was signed into law by Governor Jerry Brown on September 16, 2014. The law requires local agencies to create or join a groundwater sustainability agency by 2017. The law further requires these agencies to develop a plan for managing wells and groundwater pumping by 2020 or 2022, depending on the status of their specific groundwater basin. The intent of the laws is to achieve full groundwater sustainability by 2040.

In addition, Stanislaus County has recently adopted new groundwater ordinances. In October 2013, the County Board of Supervisors adopted the Groundwater Mining and Export Prevention Ordinance which prohibits “unsustainable groundwater extraction” and export of water to areas outside of the County.

This ordinance was further amended in 2014 to require all new wells constructed in the County after November 25, 2014 to demonstrate that pumping from the well will not constitute an "unsustainable extraction of groundwater". The ordinance also includes provisions for exemptions from the ordinance requirements, such as for uses that extract less than two acre-feet per year, or for wells that are addressed in a Groundwater Sustainability Plan, adopted per California Water Code section 10727 et seq.

However, given the passage of the recent State legislation and County ordinance, the Project will need to demonstrate that the new groundwater pumping facilities will not create an unsustainable extraction of groundwater. Alternatively, new wells constructed by the Project may be exempt from the County's ordinance if they are included in a Groundwater Sustainability Plan.

3.3 WATER SUPPLY ALTERNATIVES DISCUSSION

Utilization of surface water as the only supply source for the project is not a viable alternative. As previously discussed, the County currently has neither surface water rights nor entitlements, and there are limited opportunities for transfers from agencies with existing water rights to the DMC or CAQ. Even if rights are obtained, the delivery of surface water is still subject to extreme fluctuations depending on annual storage and rainfall.

Utilization of groundwater as the only supply source for the project is a viable alternative. Groundwater is much more reliable than surface water from the DMC or CAQ, as it would avoid interruptions due to maintenance, allocation concerns, or environmental concerns. Groundwater is also not as dependent on fluctuations in annual precipitation. As shown in prior studies, there is evidence to support that groundwater levels in the area are relatively stable over time. Monitoring and additional studies, if needed, would be required to demonstrate that the new wells would sustainably extract groundwater, but there would not be a need to purchase entitlements. The preferred alternative for the Project would be use of groundwater

Wellhead treatment systems would be required for all Phase 1A, 2, and 3 potable water supply wells servicing the Project area. The County will need to perform routine water sampling from areas throughout the Project site to determine exactly which contaminants are present in the underground aquifers along with their associated concentrations. Capital and operations and maintenance (O&M) costs for wellhead treatment systems are discussed in Chapter 6.

For all supply alternatives, non-potable water may be utilized for irrigation and fire protection purposes. Use of non-potable water for these purposes will significantly reduce costs for water treatment normally required for drinking water standards.

3.3.1 Groundwater Supply Alternatives for Consideration in the Environmental Impact Report

The proposed project requires water supplies for both potable and non-potable water demands. Estimates of these demands are developed in Section 4.0. Stanislaus County commissioned a separate study for updated concepts for water supply that consider the impacts and implications of California Senate Bill (SB) 1263. Under SB 1263, any new drinking water system seeking a permit from the State Water Resources Control Board's Division of Drinking Water (DDW) must conduct a meaningful dialogue with all existing systems within three miles of any portion of the respective water service areas to evaluate the feasibility of consolidation, annexation, or extension of water services. The CLIBP is within three miles of both the COP and CLCSD water systems. Preliminary discussions have been held by Stanislaus County with both systems' engineering and administrative staff to assess viable alternatives to extend their respective service areas to include the CLIBP. The full memo report is included in Appendix C.

The three water supply alternatives to the considered are:

- Option 1: extension of the CLCSD service area to the CLIP to cooperatively supply water and system improvements under the existing drinking water supply permit.
- Option 2: the County performs all steps necessary to obtain a new permit to provide drinking water to the CLIBP including the required evaluations with nearby systems.
- Option 3: the COP's water service area is extended to include the CLIBP under its existing drinking water supply permit.

The infrastructure requirements for these three alternatives are discussed in Section 6.0. All three alternatives are presented for consideration in the EIR.

4.0 PROPOSED DEMAND

Section 4 presents an overview of the proposed land uses and water demand projections for the Project.

4.1 PROPOSED LAND USE

The Project proposes to develop the 1,528-acre site from its current land use into a business park with primarily industrial land uses. This study assumes that 1,274 acres of the Project will be developable and of the 1,274 developable acres, 1,274 acres will require potable and non-potable water service. Figure 1.1 shows the phasing plan for the Project based on the Crows Landing Industrial Business Park. The Project area designated in Figure 1.1 as Phase 1A (Fink Road Corridor) will be developed first during Phase 1.

4.2 POTABLE WATER AND NON-POTABLE WATER DEMAND PROJECTIONS

Water demand projections developed for this study are based on the total acreages of developable areas within the limits of the Project and a total water demand rate of 2,500 gallons per day per acre (gpd/ac) per direction of the Stanislaus County Department of Public Works (SCDPW). This demand rate is based on typical values published in the *Water Distribution System Handbook* for industrial and commercial land uses, and a comparison of local agency planning demand values. This demand rate is slightly higher than the City of Modesto's demand unit water use factor of 2.75 af/ac/yr for industrial land use designations, which is equivalent to 2,455 gallons per day per acre. (COM 2014) Potable water demands to meet domestic needs are estimated by the SCDPW to be 60% of the total water demand and non-potable water demands for fire protection and irrigation uses are estimated to be, on average, 40% of the total water demand. Actual irrigation demand will vary seasonally, with much higher demands in the summer dry season, and low to none during winter wet season. These projections are based on land use acreage rather than population projections, which will account for expected potable water for domestic use and non-potable water for irrigation and fire flow use within the Project. Development of the demand projections is achieved by multiplying unit water use factors for each land use category, based on typical average water duty values and peaking factors, by the acreage for each land use area. The land use-based projection methodology applies for all land uses except the airport and multimodal trails. The airport and multimodal land uses were deemed to use significantly less water than the other land uses and, therefore, an alternative approach was considered for each. Potable airport water demands were calculated based on the sewer loading factor stated in Table 3-2 of the textbook entitled *Wastewater Engineering Treatment and Reuse*, which states that a person in an airport generates approximately 5 gallons of sewage per day.[1] A potable water demand can be derived from this sewer loading factor by assuming a return-to-sewer percentage. It has been shown that approximately 90 percent of per-capita water used is returned to the sewer[2]. Using the return-to-sewer percentage, the sewer loading factor can be used to estimate water demand by dividing the sewer

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[1] Tchobanoglous, et. al. Wastewater Engineering Treatment and Reuse, 4th Edition, McGraw-Hill, New York, New York, 2003. page 157.

[2] Tchobanoglous, et. al. Wastewater Engineering Treatment and Reuse, 4th Edition, McGraw-Hill, New York, New York, 2003. page 155.

loading factor by the return-to-sewer percentage (5 gallons per capita per day/0.9 = 5.6 gallons per capita per day). The County anticipates that approximately 100 people will utilize the airport/multimodal facilities per day. Assuming 100 people per day and 5.6 gallons per person per day yields a calculated average day demand of approximately 0.39 gallons per minute for the airport/multimodal land uses. The non-potable airport demand was calculated using the Simplified Landscape Irrigation Demand Estimation (SLIDE) methodology. The airport is approximately 370 acres in size, of which approximately 297 acres will be covered in runway. The remaining 75 acres will be unpaved and assumed to be landscaped. The SLIDE method calculates annual water demand by adjusting reference evapotranspiration by specified plant factors using the following formula:

$$\textit{Water Demand (gallons per year)} = \sum_{1}^{n} ((ET_O x PF_{n)} x LA_n) \ x \ 0.623$$

Where:

ET_O = Reference Evapotranspiration (inches)

PF = Plant Factor (unitless)

LA = Landscaped Area (square feet)

0.623 = conversion factor

For this study, it is assumed that all irrigated areas in the airport will be planted with trees, shrubs, groundcovers, and vines and have a plant factor of 0.5[3]. In addition to irrigation demand, the aviation/ multimodal non-potable demand also includes an estimate of non-potable water to be utilized at the airplane wash rack areas. Landscaping will be drought tolerant and subject to local water drought and conservation policies (Stanislaus County Code Title 21.102 Landscape and Irrigation Standards) and the Model Water Efficient Landscape Ordinance (MWELO).

Potable water demand projections are calculated for Average Day Demand (ADD), Maximum Day Demand (MDD), and Peak Hour Demand (PHD). ADDs are representative of the total annual quantity of water production for an agency or municipality, divided by 365; these values are typically determined based on an average day unit demand as determined by the governing agency. MDDs are representative of the highest water demand of the year during any 24-hour period. PHDs are representative of the highest water demand of the year during any 1-hour period. Water projections for fire flow demands typically range from 1,500 gallons per minute (GPM) to 8,000 GPM, depending on the type of land use.

4.3 FIRE FLOW REQUIREMENTS

The system must be adequately sized to provide the required fire flows for the specified duration in accordance with the California Fire Code (CFC) and any other local agency criteria. Numerous factors

[3] Published on the Division of Agriculture and Natural Sciences, University of California website (http://ucanr.edu/sites/UrbanHort/Water_Use_of_Turfgrass_and_Landscape_Plant_Materials/SLIDE__Simplified_Irrigation_Demand_Estimation/) accessed on 2/8/2016.

may impact fire flow requirements, such as building size, type of construction, use of automatic sprinkler systems, number of stories, exposure, etc. For the purposes of this study, because the Project land use is predominantly industrial, the modeling analysis assumes a single fire flow requirement of 3,000 GPM for a 4-hour duration.

This fire flow and duration assumes that buildings are equipped with automatic sprinkler systems per the California Fire Code (CFC), Title 24, Part 9, Appendix B. The CFC allows for a reduction of required fire flow of up to 75% if buildings are provided with approved automatic sprinkler systems. The largest fire flow required by the CFC is 8,000 gpm for 4 hours. The application of the 75% reduction factor to 8,000 gpm requires a fire flow of 2,000 gpm for 4 hours. The County has indicated that all new construction related to the Project will have a fire sprinkler systems and therefore should be eligible for a required fire flow reduction of 75%. This master plan uses a fire flow demand of 3,000 gpm to plan for potential changes to the fire flow requirements when the project develops. Residual pressures during fire flow conditions are to be maintained at no less than 20 pounds per square inch (psi) at the most remote junction node in the system. The most restrictive condition may not necessarily be at the service location. Any required fire flows in addition to those indicated in Table 4.1 will be provided by individual developments through additional on-site storage or through other mitigation measures. Typical fire flows/durations for the Project land use categories are listed in Table 4.1.

Table 4.1 – Required Fire Flows

Use Category	Required Fire Flow (GPM)	Duration (Hours)
Industrial/Business Park	3,000	4

4.4 PEAKING FACTORS AND SUPPLY REQUIREMENTS

MDD projections are used to determine the Project's overall supply requirements and the capacity of supply components. PHD projections and MDD with fire flow projections are typically used in the sizing of pipelines, storage, and pumping facilities. City of Patterson MDD and PHD to ADD peaking factors of 2 and 4, respectively, are assumed for all land uses. This is considered a conservative assumption, as the City of Modesto utilizes lower peaking factors of 1.75 and 2.46 for MDD and PHD, respectively. (COM 2014) The proposed water supply system must be capable of conveying MDD, which will be met by water supply facilities. Storage facilities will provide equalization storage during ADD to provide water supply during higher demand periods such as PHD, as further discussed in Section 5.3.

4.4.1 Potable Water Buildout ADD, MDD, and PHD Projections

Projected ADD, MDD, and PHD for the Project are 1.34 million gallons per day (MGD), 2.67 MGD, and 5.35 MGD, respectively, as shown in Table 4.2. Actual demands may vary somewhat from initial projections, based on numerous factors, such as different types of industry, density, employees per acre, conservation, etc.

Table 4.2 – Projected Buildout Potable Water Demand

Land Use	Acreage	Estimated Daily Visitors	Avg. Day Unit Demand[1,2] (GPCD or GAL/ACRE/DAY)	Avg. Day Potable Demand[3] (GAL/ACRE/DAY)	Demand (MGD)		
					Avg. Day	Max Day[4]	Peak Hour[4]
Aviation/Multimodal	383	100	5.6	5.6	0.0006	0.0011	0.0022
Aviation Related	46	-	2,500	1,500	0.07	0.14	0.28
Logistics/Distribution	349	-	2,500	1,500	0.52	1.05	2.09
Industrial	350	-	2,500	1,500	0.53	1.05	2.10
Business Park	78	-	2,500	1,500	0.12	0.23	0.47
Public Facilities	68	-	2,500	1,500	0.10	0.20	0.41
Total	**1,274**				**1.34**	**2.67**	**5.35**

Notes:

[1]Unit demand values are based on direction of the Stanislaus County Department of Public Works and typical published values published in the Water Distribution System Handbook, Larry W. Mays, The McGraw-Hill Companies, Inc. Copyright 2000, Table 3.2 Typical Water Duties

[2] Unit Demand is calculated estimated from typical sewer loading for aviation land use. Factor is calculated on the assumtion that 90% of water becomes sewage (5/0.9 = 5.6)

[3]Non -aviation/multimodal unit demands based on potable water accounting for 60% of unit demand

[4]The ratio of average day demand to maximum day demand is estimated at 1:2.0. The ratio of average day demand to peak hour demand is estimated to be 1:4.0.

4.4.1.1 Potable Water Phase 1 ADD, MDD, and PHD Projections

Projected Phase 1 ADD, MDD, and PHD for the Project are 0.59 MGD, 1.18 MGD, and 2.37 MGD, respectively, as shown in Table 4.3.

Table 4.3 – Projected Phase 1 Potable Water Demand

Land Use	Acreage	Estimated Daily Visitors	Avg. Day Unit Demand[1,2] (GAL/ACRE/DAY)	Avg. Day Potable Demand[3] (GAL/ACRE/DAY)	Demand (MGD)		
					Avg. Day	Max Day[3]	Peak Hour[3]
Phase 1A							
Logistics/Distribution	52	-	2,500	1,500	0.078	0.156	0.312
Industrial	41	-	2,500	1,500	0.0615	0.123	0.246
Business Park	10	-	2,500	1,500	0.015	0.03	0.06
Subtotal	**103**				**0.15**	**0.31**	**0.62**
Phase 1B							
Logistics/Distribution	138	-	2,500	1,500	0.21	0.41	0.83
Industrial	110	-	2,500	1,500	0.17	0.33	0.66
Business Park	28	-	2,500	1,500	0.04	0.08	0.17
Airport - Phase 1 Infrastructure	370	100	5.6	5.6	0.0006	0.0011	0.0022
Public Facilities	15	-	2,500	1,500	0.02	0.05	0.09
Subtotal	**661**				**0.44**	**0.87**	**1.75**
Total	**764**				**0.59**	**1.18**	**2.37**

Notes:

[1]Unit demand values are based on direction of the Stanislaus County Department of Public Works and typical published values published in the Water Distribution System Handbook, Larry W. Mays, The McGraw-Hill Companies, Inc. Copyright 2000, Tbale 3.2 Typical Water Duties

[2] Unit Demand is calculated estimated from typical sewer loading for aviation land use. Factor is calculated on the assumtion that 90% of water becomes sewage (5/0.9 = 5.6), Tchobanoglous, et. al. Wastewater Engineering Treatment and Reuse, 4th Edition, McGraw-Hill, New York, New York, 2003.

[3]Non -aviation/multimodal unit demands based on potable water accounting for 60% of unit demand

[4]The ratio of average day demand to maximum day demand is estimated at 1:2.0. The ratio of average day demand to peak hour demand is estimated to be 1:4.0.

4.4.1.2 Potable Water Phase 2 ADD, MDD, and PHD Projections

Projected Phase 2 ADD, MDD, and PHD for the Project are 0.35 MGD, 0.71 MGD, and 1.42 MGD, respectively, as shown in Table 4.4.

Table 4.4 – Projected Phase 2 Potable Water Demand

Land Use	Acreage	Avg. Day Unit Demand[1]	Avg. Day Potable Demand[2]	Demand (MGD)		
				Avg. Day	Max Day[3]	Peak Hour[3]
SR 33 Corridor South						
Logistics/Distribution	57	2,500	1,500	0.09	0.17	0.34
Industrial	71	2,500	1,500	0.11	0.21	0.43
Business Park	14	2,500	1,500	0.02	0.04	0.08
Aviation-Related Use	46	2,500	1,500	0.07	0.14	0.28
Multimodal Transportation	13	2,500	1,500	0.02	0.04	0.08
Public Facilities	35	2,500	1,500	0.05	0.11	0.21
Total	**236**			**0.35**	**0.71**	**1.42**

Notes:
[1]Unit demand values are based on direction of the Stanislaus County Department of Public Works and typical values published in the Water Distribution System Handbook, Larry W. Mays, The McGraw-Hill Companies, Inc. Copyright 2000, Tbale 3.2 Typical Water Duties
[2]Based on Potable Water accounting for 60% of Unit Demand
[3]The ratio of average day demand to maximum day demand is estimated at 1:2.0. The ratio of average day demand to peak hour demand is estimated to be 1:4.0.

4.4.1.3 Potable Water Phase 3 ADD, MDD, and PHD Projections

Projected Phase 3 ADD, MDD, and PHD for the Project are 0.41 MGD, 0.82 MGD, and 1.64 MGD, respectively, as shown in Table 4.5.

Table 4.5 - Projected Phase 3 Potable Water Demand

Land Use	Acreage	Avg. Day Unit Demand[1]	Avg. Day Potable Demand[2]	Demand (MGD)		
				Avg. Day	Max Day[3]	Peak Hour[3]
SR 33 Corridor North						
Logistics/Distribution	102	2,500	1,500	0.15	0.31	0.61
Industrial	128	2,500	1,500	0.19	0.38	0.77
Business Park	26	2,500	1,500	0.04	0.08	0.16
Public Facilities	18	2,500	1,500	0.03	0.05	0.11
Total	**274**			**0.41**	**0.82**	**1.64**

Notes:
[1]Unit demand values are based on direction of the Stanislaus County Department of Public Works and typical values published in the Water Distribution System Handbook, Larry W. Mays, The McGraw-Hill Companies, Inc. Copyright 2000, Tbale 3.2 Typical Water Duties
[2]Based on Potable Water accounting for 60% of Unit Demand
[3]The ratio of average day demand to maximum day demand is estimated at 1:2.0. The ratio of average day demand to peak hour demand is estimated to be 1:4.0.

4.4.2 Non-Potable Water Irrigation + Fire Flow Projection

The projected average daily irrigation demand for the Project is 1.18 million gallons per day (MGD) as shown in Table 4.6. Fire flow demands will be serviced by the non-potable system via a non-potable storage tank and are considered separate from the irrigation demands. Actual demands may vary somewhat from initial projections, based on numerous factors, such as different types of industry, density, employees per acre, conservation, etc. The fire flow volume for all areas of the Project will be satisfied by the non-potable water storage tank to be provided in Phase 1. The projected irrigation demands for Phases 1, 2, and 3 are shown in tables 4.7, 4.8, and 4.9, respectively.

Table 4.6 – Projected Buildout Non-Potable Water Irrigation Demand

Land Use	Acreage[7]	Plant Factor	Flights Per Week	Avg. Day Unit Demand[1] (GAL/ACRE/DAY)	Avg. Day Nonpotable Demand[2] (GAL/ACRE/DAY)	Demand (MGD) Avg. Day	Demand (MGD) Max Day[5]
Aviation Wash Rack[3]	-	-	20	-	-	0.00058	0.00115
Aviation Landscape[4]	75	0.5	-	-	-	0.29	0.58
Aviation Related	46			2,500	1,000	0.05	0.09
Multimodal Transportation	13	-	-	2,500	1,000	0.01	0.03
Logistics/Distribution	349	-	-	2,500	1,000	0.35	0.70
Industrial	350	-	-	2,500	1,000	0.35	0.70
Business Park	78	-	-	2,500	1,000	0.08	0.16
Public Facilities	68	-	-	2,500	1,000	0.07	0.14
Total	**979**					**1.20**	**2.39**

Notes:

[1]Unit demand values are based on direction of the Stanislaus County Department of Public Works and typical published values in the Water Distribution System Handbook, Larry W. Mays, The McGraw-Hill Companies, Inc. Copyright 2000, Table 3.2 Typical Water Duties

[2]Based on Potable Water accounting for 40% of Unit Demand

[3] Demand estimated to be approximately 208,000 gallons/year. Calculated as 20 washes/week x 52 weeks/year x 20 gallons/minute x 10 minutes/wash. 20 gallons/ minute is based on wash rack manufactured by Hydro Engineering (http://www.hydroblaster.com/InstantAircraftWashRack.htm) accessed 2/8/16.

[4]Demand is estimated by using SLIDE methodology which applies a plant factor to the area reference ET_O. ET_O is estimated to be 58.41 inches per year for the CLIBP area.
http://ucanr.edu/sites/UrbanHort/Water_Use_of_Turfgrass_and_Landscape_Plant_Materials/SLIDE__Simplified_Irrigation_Demand_Estimation/)

[5] Maximum day demand is equal to 2 times the nonpotable water average day demand.

[6]Fire flow demand is 3,000 gpm for a duration of 4 hours. This demand is not considered part of the average day non-potable water irrigation demand as it will be accounted for by the planned storage tanks

[7] This area is representative of the area planned to receive nonpotable water. The reminaing 295 acres is runways which do not require water. (966+295 = 1,261)

Table 4.7 – Projected Phase 1 Non-Potable Water Irrigation and Fire Flow Demand

Land Use	Acreage[7]	Plant Factor	Flights Per Week	Avg. Day Unit Demand[1] (GAL/ACRE/DAY)	Avg. Day Nonpotable Demand[2] (GAL/ACRE/DAY)	Demand (MGD) Avg. Day	Demand (MGD) Max Day[5]
Phase 1A							
Logistics/Distribution	52	-	-	2,500	1,000	0.05	0.104
Industrial	41	-	-	2,500	1,000	0.04	0.082
Business Park	10	-	-	2,500	1,000	0.01	0.02
Subtotal	**103**					**0.103**	**0.206**
Phase 1B							
Logistics/Distribution	138	-	-	2,500	1,000	0.14	0.276
Industrial	110	-	-	2,500	1,000	0.11	0.22
Business Park	28	-	-	2,500	1,000	0.03	0.056
Aviation Wash Rack[3]	-	-	20	-	-	0.0006	0.00116
Aviation Landscape[4]	75	0.5	-	-	-	0.29	0.58
Public Facilities	15			2,500	1,000	0.02	0.03
Subtotal	**366**					**0.582**	**1.163**
Total	**469**					**0.68**	**1.37**

Notes:

[1]Unit demand values are based on direction of the Stanislaus County Department of Public Works and typical published values in the Water Distribution System Handbook, Larry W. Mays, The McGraw-Hill Companies, Inc. Copyright 2000, Table 3.2 Typical Water Duties

[2]Based on Potable Water accounting for 40% of Unit Demand

[3] Demand estimated to be approximately 208,000 gallons/year. Calculated as 20 washes/week x 52 weeks/year x 20 gallons/minute x 10 minutes/wash. 20 gallons/ minute is based on wash rack manufactured by Hydro Engineering (http://www.hydroblaster.com/InstantAircraftWashRack.htm) accessed 2/8/16.

[4]Demand is estimated by using SLIDE methodology which applies a plant factor to the area reference ET_O. ET_O is estimated to be 58.41 inches per year for the CLIBP area.
http://ucanr.edu/sites/UrbanHort/Water_Use_of_Turfgrass_and_Landscape_Plant_Materials/SLIDE__Simplified_Irrigation_Demand_Estimation/)

[5] Maximum day demand is equal to 2 times the nonpotable water average day demand.

[6]Fire flow demand is 3,000 gpm for a duration of 4 hours. This demand is not considered part of the average day non-potable water irrigation demand as it will be accounted for by the planned storage tanks

[7] This area is representative of the area planned to receive nonpotable water.

Table 4.8 – Projected Phase 2 Non-Potable Water Irrigation and Fire Flow Demand

Land Use	Acreage	Avg. Day Unit Demand[1] (GAL/ACRE/DAY)	Avg. Day Nonpotable Demand[2]	Demand (MGD) Avg. Day	Demand (MGD) Max Day[3]
SR 33 Corridor South					
Logistics/Distribution	57	2,500	1,000	0.06	0.114
Industrial	71	2,500	1,000	0.07	0.142
Business Park	14	2,500	1,000	0.01	0.028
Aviation Related	46	2,500	1,000	0.05	0.092
Multimodal Transportation	13	2,500	1,000	0.01	0.026
Public Facilities	35	2,500	1,000	0.04	0.07
Total	**236**			**0.24**	**0.47**

Notes:

[1]Unit demand values are based on direction of the Stanislaus County Department of Public Works and typical published values in the Water Distribution System Handbook, Larry W. Mays, The McGraw-Hill Companies, Inc. Copyright 2000, Table 3.2 Typical Water Duties

[2]Based on Potable Water accounting for 40% of Unit Demand

[3] Maximum day demand is equal to 2 times the nonpotable water average day demand.

[4]Fire flow demand is 3,000 gpm for a duration of 4 hours. This demand is not considered part of the average day non-potable water irrigation demand as it will be accounted for by the planned storage tanks

Table 4.9 - Projected Phase 3 Non-Potable Water Irrigation Demand

Land Use	Acreage	Avg. Day Unit Demand[1] (GAL/ACRE/DAY)	Avg. Day Nonpotable Demand[2]	Demand (MGD) Avg. Day	Demand (MGD) Max Day[3]
SR 33 Corridor North					
Logistics/Distribution	102	2,500	1,000	0.10	0.204
Industrial	128	2,500	1,000	0.13	0.256
Business Park	26	2,500	1,000	0.03	0.052
Public Facilities	18	2,500	1,000	0.02	0.036
Total	**274**			**0.27**	**0.55**

Notes:

[1]Unit demand values are based on direction of the Stanislaus County Department of Public Works and typical published values in the Water Distribution System Handbook, Larry W. Mays, The McGraw-Hill Companies, Inc. Copyright 2000, Table 3.2 Typical Water Duties

[2]Based on Potable Water accounting for 40% of Unit Demand

[3] Maximum day demand is equal to 2 times the nonpotable water average day demand.

[4]Fire flow demand is 3,000 gpm for a duration of 4 hours. This demand is not considered part of the average day non-potable water irrigation demand as it will be accounted for by the planned storage tanks

5.0 SYSTEM OPERATING CRITERIA

Section 5 discusses system operating criteria for the Project.

5.1 TRANSMISSION / DISTRIBUTION DESIGN CRITERIA

Hydraulic modeling criteria for backbone distribution and transmission mains are typically established to keep velocities and head losses per thousand lineal feet within acceptable ranges. The potable water system must also be capable of meeting domestic demands at adequate service pressures. The non-potable water system must deliver the required irrigation demands and fire flow demands to all regions of the system.

The service velocity and criteria used in this analysis are consistent with the typical values used in general engineering practice. The minimum and maximum pressure requirements for system service criteria used for this study are shown in Table 5.1.

Table 5.1 – System Service Criteria

Demand Scenario	Minimum Pressure (psi)	Maximum Pressure (psi)
Potable Water: Average Day Demand	40	120
Potable Water: Maximum Day Demand	40	120
Potable Water: Peak Hour Demand	30	120
Non-Potable Water: Irrigation Demand plus Fire Flow Demand	20	120

The maximum fluid velocity criteria used in the evaluation of large distribution mains (16-inch-diameter pipe and greater) and standard distribution mains (pipe diameter less than 16 inches) is shown in Table 5.2 in feet per second (fps).

Table 5.2 – Water Main Velocity Criteria

Demand Scenario	Maximum Velocity (fps)	
	Large Main	Standard Main
Average Day Demand	3	5
Maximum Day Demand	5	5
Peak Hour Demand	8	8
Irrigation Demand plus Fire Flow	10	10

5.2 TANK AND BOOSTER PUMP STATION SIZING CRITERIA

Storage tanks serve as equalization measures to meet variable water demands and are typically sized to meet peak operational needs as well as emergency needs and fire flows. Fluctuations in water usage rates can be met by continuously varying source production, by continuously varying pumping rates, or by filling and draining storage tanks. The process of filling and draining storage tanks is much easier operationally and is generally less expensive than the other methods. Facilities serving portions of a distribution system with storage tanks generally need to be sized only to meet maximum daily demands, with the storage tanks providing additional water during instantaneous peak demands. Typically, the volumetric difference between peak demands and the available supply is retained in above-ground tanks as a practical method to meet operational fluctuations in demands and to maintain reasonably sized mains and to comply with California Code of Regulations (CCR) Title 22 requirements. Potable water storage requirements are shown in Table 5.3.

Table 5.3 – Potable Water Storage Requirements

Component	Storage Volume
Potable Water: Operation	25% of MDD
Potable Water: Emergency	150% of ADD
Non-Potable Water Fire Flow	Fire Flow x Duration

Booster pump stations will need to be sized to meet the higher requirements of irrigation demand plus fire flow demand, and/or PHD, as required for each facility.

5.3 TANK / BOOSTER PUMP STATIONS

Section 6 discusses the sizing of storage tanks and booster pump stations.

5.3.1 Tank and Booster Pump Stations

The Project will need to meet its own potable water and non-potable water storage requirements at buildout.

5.3.2 Buildout Storage and Pumping Requirements

Water storage tanks and a booster pumping facility will be needed to serve the Project and will be sized as shown in Table 5.4.

Table 5.4 – Buildout Water Storage Requirements

Component	Phase 1A Storage (MG)	Phase 1+2 Storage (MG)	Buildout Storage (MG)
Potable Water: Operation (0.25*MDD)	0.30	0.68	0.68
Potable Water: Emergency (1.25*ADD)	0.89	2.03	2.03
Potable Water Total	1.19	2.71	2.71
Non-Potable Water: Fire (3,000 gpm for 4 hours)	0.72	0.72	0.72
Non-Potable Water Total	0.72	0.72	0.72

Based on the storage requirements, it is estimated that a total of 3 water storage tanks (2 potable water and 1 non-potable water) are required for the Project. The buildout of the Project requires approximately 2.71 MG potable water storage and 0.72 MG non-potable water storage.

The non-potable water booster pump station at the non-potable water storage tank site will need to meet the irrigation demand and 3,000 GPM fire flow demand.

5.3.2.1 Phase 1A Storage and Pumping Requirements

The initial phase of the Project shall provide one potable water tank with a 1.19-MG capacity and a non-potable tank with a 0.72-MG capacity and a non-potable water booster pump station with capacity to meet the required fire flow demand. This infrastructure is shown in Figure 7.1 at the end of this report.

Phase 1A, Option 1
This alternative described in Appendix C includes combining the needs of the CLCSD and CLIBP to one water system. Phase 1A infrastructure is shown in Figure A2 in Appendix C. Components include the 1.19 MG potable water storage tank and water treatment system at the corner of Bell and Fink Roads, and the 0.72 MG non-potable water tank. Two new wells would be installed at the CLIBP, and supply water to both the potable and non-potable water tanks. Additional water supply would come from two existing wells at the CLCSD and conveyed through a water supply pipeline along Fink Road. This alternative allows for blending water supplies from both CLCSD and CLIBP, potentially eliminating the need for additional treatment.

Phase 1A, Option 2
This alternative includes supplying all water needs from the CLIBP. Phase 1A infrastructure is shown in Figure B2 in Appendix C. Components include two new wells that supply water to both the potable and non-potable water tanks and a water treatment system.

Phase 1A, Option 3
This alternative has the same infrastructure as Option 2, with the exception of an intertie to COP that occurs in Phase 2.

5.3.2.2 Phase 2 Storage Requirements

An additional potable water tank with a 1.52-MG capacity shall be provided as part of Phase 2 of the Project. The addition of this second tank will increase the potable water storage capacity to 2.71 MG. The required non-potable water storage is provided by the tank installed in the initial phase of the Project. This infrastructure is shown in Figure 7.2 at the end of this report.

Phase 2, Option 1
This alternative described in Appendix C includes additional Phase 2 infrastructure as shown in Figure A2 in Appendix C. Components include the additional 1.52 MG potable water storage tank and two new wells at the CLIBP, supplying water to both the potable and non-potable water tanks.

Phase 2, Option 2
This alternative includes construction of the same additional Phase 2 infrastructure as Alternative A.

Phase 2, Option 3
This alternative has the same infrastructure as Option 2, plus the intertie pipeline to COP that is constructed in Phase 2.

5.3.2.3 Phase 3 Storage Requirements

The potable water storage requirements were accounted for in the sizing of the potable water storage tank that in Phase 2. This infrastructure is shown in Figure 7.3 at the end of this report.

Phase 3, Alternatives A, B and C
Additional infrastructure for these alternatives is shown in Appendix C in Figures A3, B3 and C3, respectively. No additional wells or storage tanks are constructed in Phase 3.

6.0 POTABLE AND NON-POTABLE WATER INFRASTRUCTURE

Based on criteria and demands discussed in Sections 4 and 5, a preliminary design can be determined for the project site. This section discusses the preliminary design and provides preliminary costs.

6.1 PROPOSED ON-SITE POTABLE WATER INFRASTRUCTURE

Development of Phase 1 proposes construction of backbone infrastructure to provide potable water service to the airport, the Phase 1 area immediately south of the airport, and 15 acres of Public Facilities. Potable water infrastructure required as part of Phase 1 improvements includes distribution piping, valves, a potable water storage tank (1.2 MG) east of the intersection of Davis Road and Fink Road, and a water well and booster pump station located adjacent to the potable water storage tank. Estimated construction costs for the Phase 1 potable water system construction are provided in Table 6.1.

Table 6.1 Estimated Phase 1 Onsite Potable Water System Construction Costs

Description	Quantity	Unit	Unit Price	Cost
Phase 1A				
12-Inch PVC	4,240	LF	$ 65.00	$ 275,600.00
12-Inch Gate Valve	4	EA	$ 1,000.00	$ 4,000.00
Water Well and Booster Pump Station	1	EA	$ 2,500,000.00	$ 2,500,000.00
Potable Water Storage Tank (1.4 MG)	1	EA	$ 2,550,000.00	$ 2,550,000.00
Wellhead Treatment System[1]	1	LS	$ 2,150,000.00	$ 2,150,000.00
Subtotal				**$ 7,479,600.00**
Phase 1B				
12-Inch PVC	34,460	LF	$ 65.00	$ 2,239,900.00
12-Inch Gate Valve	34	EA	$ 1,000.00	$ 34,000.00
Subtotal				**$ 2,273,900.00**
Subtotal of Phase 1				$ 9,753,500.00
Engineering Costs (20%)				$ 1,951,000.00
Contingencies (20%)				$ 2,341,000.00
Total Estimated Opinion of Probable Construction Cost				**$ 14,046,000.00**

Notes:
[1]This line item is for capital costs associated with a hexavalent chromium removel system, operations and maintenance costs are in addition to capital costs and are estimated at $186/acre-foot of water produced. Estimated costs were prepared by Gilmore Engineering, Inc. in January 2015 and provided to AECOM for use in this study.

Phase 1, Option 1

The infrastructure in this alternative described in Appendix C is similar to Table 6.1 except for the following:

- Second Well and Pump $1,250,000
- 1.8 mile Water Supply Pipeline from CLCSD $990,000
- Additional engineering costs $448,000
- Additional contingency costs $537,600
- Revised Total Opinion of Probable Construction Cost 17,271,600

Phase 1, Option 2

The infrastructure in this alternative described in Appendix C is similar to Table 6.1 except for the following:

- Second Well and Pump + $1,250,000+
engineering and contingency costs $550,000
- Revised Total Opinion of Probable Construction Cost $15,846,000

Phase 1, Option 3

The infrastructure in this alternative described in Appendix C is similar to Table 6.1 except for the following:

- Second Well and Pump + $1,250,000 +
  engineering and contingency costs $550,000
- Revised Total Opinion of Probable Construction Cost $15,846,000

Development of Phase 2 proposes construction of backbone infrastructure to provide potable water service to the Phase 2 areas north of the airport. Potable water infrastructure required as part of Phase 2 improvements includes distribution piping, valves, a potable water storage tank (1.47 MG) in the northeast portion of the project area, and a water well and booster pump station located adjacent to the potable water storage tank. Estimated construction costs for the Phase 2 potable water system construction are provided in Table 6.2.

Table 6.2 Estimated Phase 2 Onsite Potable Water System Construction Costs

Description	Quantity	Unit	Unit Price	Cost
SR 33 Corridor South				
12-Inch PVC	32,700	LF	$ 65.00	$ 2,125,500.00
12-Inch Gate Valve	32	EA	$ 1,000.00	$ 32,000.00
Water Well and Booster Pump Station	1	EA	$ 2,500,000.00	$ 2,500,000.00
Potable Water Storage Tank (1.4 MG)	1	EA	$ 1,650,000.00	$ 1,650,000.00
Wellhead Treatment System[1]	1	LS	$ 2,150,000.00	$ 2,150,000.00
Subtotal				$ 8,457,500.00
Engineering Costs (20%)				$ 1,692,000.00
Contingencies (20%)				$ 2,030,000.00
Total Estimated Opinion of Probable Construction Cost				**$ 12,180,000.00**

Notes:
[1]This line item is for capital costs associated with a hexavalent chromium removel system, operations and maintenance costs are in addition to capital costs and are estimated at $186/acre-foot of water produced. Estimated costs were prepared by Gilmore Engineering, Inc. in January 2015 and provided to AECOM for use in this study.

Phase 2, Alternatives A and B

The infrastructure and cost opinions in this alternative described in Appendix C are similar to Table 6.2 except for the following:

- Second Well and Pump + $1,250,000 +

engineering and contingency costs $550,000
- Revised Total Opinion of Probable Construction Cost $13,980,000

Phase 2, Option 3

The infrastructure in this alternative described in Appendix C is similar to Table 6.2 except for the following:

- Second Well and Pump + $1,250,000 +
  engineering and contingency costs $550,000
- Intertie Pipeline from CLIB to COP (Cost to be determined)

Development of Phase 3 proposes construction of backbone infrastructure to provide potable water service to the Phase 3 areas south of Marshall Road. Potable water infrastructure required as part of Phase 3 improvements is primarily limited to distribution piping, valves, and a water well and booster pump station located near Marshall Road between Davis Road and State Route 33. Estimated construction costs for the Phase 3 onsite potable water system construction are provided in Table 6.3.

Table 6.3 Estimated Phase 3 Onsite Potable Water System Construction Costs

Description	Quantity	Unit	Unit Price	Cost
SR 33 Corridor North				
12-Inch PVC	20,000	LF	$ 65.00	$ 1,300,000.00
12-Inch Gate Valve	20	EA	$ 1,000.00	$ 20,000.00
Water Well and Booster Pump Stati	1	EA	$ 2,500,000.00	$ 2,500,000.00
Wellhead Treatment System[1]	1	LS	$ 2,150,000.00	$ 2,150,000.00
Subtotal				$ 5,970,000.00
Engineering Costs (20%)				$ 1,194,000.00
Contingencies (20%)				$ 1,433,000.00
Total Estimated Opinion of Probable Construction Cost				**$ 8,597,000.00**
Notes:				
[1]This line item is for capital costs associated with a hexavalent chromium removel system, operations and maintenance costs are in addition to capital costs and are estimated at $186/acre-foot of water produced. Estimated costs were prepared by Gilmore Engineering, Inc. in January 2015 and provided to AECOM for use in this study.				

Phase 3, Alternatives A, B and C

The infrastructure and cost opinions of these alternatives described in Appendix C are similar to Table 6.3 except for the following:

- Removal of Well and Pump + -$1,250,000
  engineering and contingency costs -$550,000
- Revised Total Opinion of Probable Construction Cost $6,797,000

6.2 PROPOSED NON-POTABLE WATER INFRASTRUCTURE

Development of Phase 1 proposes construction of backbone infrastructure to provide non-potable water service to the airport, and the Phase 1 area immediately south of the airport, and 15 acres of Public Facilities. Non-potable water infrastructure required as part of Phase 1 improvements includes distribution piping, valves, a non-potable water storage tank (0.75 MG) located south of the airport, a water well adjacent to the non-potable storage tank, and fire hydrants. Estimated construction costs for the Phase 1 non-potable water system construction are provided in Table 6.4.

Table 6.4 Estimated Phase 1 Non-Potable Water System Construction Costs

Description	Quantity	Unit	Unit Price	Cost
Phase 1A				
12-Inch PVC	3,500	LF	$ 65.00	$ 227,500.00
12-Inch Gate Valve	4	EA	$ 1,000.00	$ 4,000.00
Fire Hydrant, Bury, and Gate Valve	11	EA	$ 5,000.00	$ 55,000.00
Nonpotable Water Storage Tank (0.75 MG)	1	EA	$ 1,250,000.00	$ 1,250,000.00
Subtotal				**$ 1,536,500.00**
Phase 1B				
18-Inch PVC	5,300	LF	$ 100.00	$ 530,000.00
12-Inch PVC	29,500	LF	$ 65.00	$ 1,917,500.00
18-Inch Gate Valve	5	EA	$ 5,000.00	$ 25,000.00
12-Inch Gate Valve	29	EA	$ 1,000.00	$ 29,000.00
Fire Hydrant, Bury, and Gate Valve	89	EA	$ 5,000.00	$ 445,000.00
New Nonpotable Well & Booster Pump Station	1	EA	$ 2,500,000.00	$ 2,500,000.00
Nonpotable Water Well Pump	2	EA	$ 500,000.00	$ 1,000,000.00
Subtotal				**$ 6,446,500.00**
Subtotal of Phase 1				$ 7,983,000.00
Engineering Costs (20%)				$ 1,597,000.00
Contingencies (20%)				$ 1,916,000.00
Total Estimated Opinion of Probable Construction Cost				**$ 11,496,000.00**

Phase 1, Alternatives A, B and C

The infrastructure and cost opinions of these alternatives described in Appendix C are similar to Table 6.4.

Development of Phase 2 proposes construction of backbone infrastructure to provide non-potable water service to the Phase 2 areas north of the airport. Non-potable water infrastructure required as part of Phase 2 improvements is primarily limited to distribution piping, fire hydrants, and valves. Estimated construction costs for the Phase 2 non-potable water system construction are provided in Table 6.5.

Table 6.5 Estimated Phase 2 Non-Potable Water System Construction Costs

Description	Quantity	Unit	Unit Price	Cost
SR 33 Corridor South				
12-Inch PVC	33,000	LF	$ 65.00	$ 2,145,000.00
12-Inch Gate Valve	33	EA	$ 1,000.00	$ 33,000.00
Fire Hydrant, Bury, and Gate Valve	83	EA	$ 5,000.00	$ 415,000.00
Subtotal				$ 2,593,000.00
Engineering Costs (20%)				$ 519,000.00
Contingencies (20%)				$ 623,000.00
Total Estimated Opinion of Probable Construction Cost				**$ 3,735,000.00**

Phase 2, Option 1
The infrastructure in this alternative described in Appendix C is similar to Table 6.5 except for the following:

- Second Well and Pump + engineering and contingency costs — $1,250,000 + $550,000
- Revised Total Opinion of Probable Construction Cost — $5,535,000

Phase 2, Option 2
The infrastructure in this alternative described in Appendix C is similar to Table 6.5 except for the following:

- Second Well and Pump + engineering and contingency costs — $1,250,000 + $550,000
- Revised Total Opinion of Probable Construction Cost — $5,535,000

Phase 2, Option 3
The infrastructure in this alternative described in Appendix C is similar to Table 6.5 except for the following:

- Second Well and Pump + engineering and contingency costs — $1,250,000 + $550,000
- Revised Total Opinion of Probable Construction Cost — $5,535,000

Development of Phase 3 proposes construction of backbone infrastructure to provide non-potable water service to the Phase 3 areas south of Marshall Road. Non-potable water infrastructure required as part of Phase 3 improvements includes distribution piping, a water well, fire hydrants, and valves. Estimated construction costs for the Phase 3 non-potable water system construction are provided in Table 6.6.

Table 6.6 Estimated Phase 3 Non-Potable Water System Construction Costs

Description	Quantity	Unit	Unit Price	Cost
SR 33 Corridor North				
12-Inch PVC	20,000	LF	$ 65.00	$ 1,300,000.00
12-Inch Gate Valve	20	EA	$ 1,000.00	$ 20,000.00
Fire Hydrant, Bury, and Gate Valve	50	EA	$ 5,000.00	$ 250,000.00
Nonpotable Water Well Pump	1	EA	$ 500,000.00	$ 500,000.00
Subtotal				$ 2,070,000.00
Engineering Costs (20%)				$ 414,000.00
Contingencies (20%)				$ 497,000.00
Total Estimated Opinion of Probable Construction Cost				**$ 2,981,000.00**

<u>Phase 3, Alternatives A, B and C</u>

The infrastructure and cost opinions of these alternatives described in Appendix C are similar to Table 6.6.

7.0 WATER SYSTEM MODELING

Section 7 discusses the water model development and hydraulic modeling results.

7.1 MODEL DEVELOPMENT

For this study, the modeling software used to evaluate the Project's potable and non-potable water systems is Innovyze InfoWater. Steady-state Average Day Demand (ADD), Maximum Day Demand (MDD), and Peak Hour Demand (PHD) simulations were performed for the potable water system and an MDD irrigation demand with fire flow simulation was performed for the non-potable water system to confirm that the proposed systems will meet the criteria identified in Section 5.

The Project's total demands were distributed to the junction nodes for each system model per the tributary areas; corresponding unit demand factors per acre and peaking factors or fire flows are applied as discussed in Section 3. Each node is assigned an elevation based on the existing topography at the Project site. When multiple nodes are used for a particular land use that ranges in elevation across the area, each node is assigned an elevation within or spanning that elevation range.

Schematic figures for each of the modeling scenarios in Appendix B incorporates the land use plan and show the conceptual alignments, pipe diameters, and node ID's to aid in correlating the modeling results. Element labeling in each schematic figure is consistent with the Key ID's shown in the accompanying data for each scenario.

7.2 MODELING ASSUMPTIONS

A Hazen-Williams roughness coefficient "C" value of 130 is assigned for all system piping, which incorporates minor losses associated with fittings, valves, etc.

The supplied pressures at the potable water supply sources are approximately 45 psi and 75 psi for Phases 1 and 2, respectively. The supplied pressure at the non-potable water supply sources is 78 psi

No pressure reducing valves (PRVs) are used in the analysis.

The storage and booster pump system is not included in the model. It is assumed that adequate supply and pressure are available.

7.3 MODEL SCENARIOS

The attached model output contains the results for four scenarios:

- Potable Water: Average Day Demand
- Potable Water: Maximum Day Demand
- Potable Water: Peak Hour Demand
- Non-Potable Water: Irrigation Demand with Fire Flow Demand

7.3.1 Fire Flow Analysis

The non-potable water model evaluates the available fire flow in the system for the Irrigation Demand with Fire Flow Demand Scenario by iteratively imposing the required fire flow demand of 3,000 GPM (in addition to assigned base flow demand, if applicable) at all nodes in the model and calculates the available fire flow at each node, while maintaining a residual pressure of 20 psi at any junction. The residual pressure is identified for each node in the analysis. The system pressure and velocities are then evaluated by applying the required demand at the limiting node (system node with the least available fire flow).

7.4 MODEL RESULTS

This section provides a brief description of each analysis for the buildout scenario and a summary of results of the modeling analysis. Data output for each scenario of the modeling analysis is included in Appendix B. Figures 7.1, 7.2, and 7.3 display the potable water system pressures and velocities for the peak hour demand scenario during Phases 1, 2, and 3, respectively. Figures 7.4, 7.5, and 7.6 show residual pressures for the non-potable system under maximum day demand and a 3,000 gpm fire flow for Phases 1, 2, and 3, respectively.

7.4.1 Potable Water Scenario: Average Day Demand

For Phase 1, given a supplied system pressures of 45 psi at the Phase 1 supply source, the system pressure throughout the Project during ADD ranges from approximately 48 psi to approximately 69 psi with maximum velocity in the system of approximately 1.3 fps.

For Phases 1 and 2, given supplied system pressures of 45 psi at the Phase 1 supply source and 75 psi at the Phase 2 supply source, the system pressure throughout the Project during ADD ranges from approximately 48 psi to approximately 77 psi with maximum velocity in the system of approximately 1.4 fps.

At buildout, given supplied system pressures of 45 psi at the Phase 1 supply source, 75 psi at the Phase 2 supply source, and 73 psi at the Phase 3 supply source, the system pressure throughout the Project during ADD ranges from approximately 48 psi to approximately 77 psi with maximum velocity in the system of approximately 1.6 fps.

7.4.2 Potable Water Scenario: Maximum Day Demand

For Phase 1, given a supplied system pressures of 45 psi at the Phase 1 supply source, the system pressure throughout the Project during MDD ranges from approximately 47 psi to approximately 70 psi with maximum velocity in the system of approximately 2.6 fps.

For Phases 1 and 2, given supplied system pressures of 45 psi at the Phase 1 supply source and 75 psi at the Phase 2 supply source, the system pressure throughout the Project during MDD ranges from approximately 48 psi to approximately 77 psi with maximum velocity in the system of approximately 2 fps.

At buildout, given supplied system pressures of 45 psi at the Phase 1 supply source, 75 psi at the Phase 2 supply source, and 73 psi at the Phase 3 supply source, the system pressure throughout the Project during MDD ranges from approximately 48 psi to approximately 77 psi with maximum velocity in the system of approximately 2.2 fps.

7.4.3 Potable Water Scenario: Peak Hour Demand

For Phase 1, given a supplied system pressures of 45 psi at the Phase 1 supply source, the system pressure throughout the Project during PHD ranges from approximately 41 psi to approximately 54 psi with maximum velocity in the system of approximately 5.2 fps. Figure 7.1 shows the system pressures and velocities.

For Phases 1 and 2, given supplied system pressures of 45 psi at the Phase 1 supply source and 75 psi at the Phase 2 supply source, the system pressure throughout the Project during PHD ranges from approximately 46 psi to 75 psi with maximum velocity in the system of approximately 4 fps.

At buildout, given supplied system pressures of 45 psi at the Phase 1 supply source, 75 psi at the Phase 2 supply source, and 73 psi at the Phase 3 supply source at the point of connection to the water supply source, the system pressure throughout the Project during PHD ranges from approximately 46 psi to approximately 76 psi with maximum velocity in the system of approximately 3.7 fps. Figures 7.1, 7.2, and 7.3 show the onsite system pressures and velocities for the peak hour demand scenarios for Phases 1, 2, and 3, respectively. The modeling data output for this scenario are labeled *Scenario: Potable Water Peak Hour Demand.*

7.4.4 Non-Potable Water Scenario: Maximum Day Irrigation Demand with Fire Flow Demand

For Phase 1, given a supplied system pressure of 78 psi at the point of connection to the water supply source, the lowest residual pressure in the system is approximately 29.8 psi at Junction 27 (J27) during irrigation demand with fire flow demand. Distribution system pipe velocities are under 10 fps for the fire flow scenario.

For Phases 1 and 2, given a supplied system pressure of 78 psi at the point of connection to the water supply source, the lowest residual pressure in the system is approximately 20.9 psi at Junction 6 (J6) during irrigation demand with fire flow demand. Distribution system pipe velocities are under 10 fps for the fire flow scenario.

At buildout, given a supplied system pressure of 78 psi at the point of connection to the water supply source, the lowest residual pressure in the system is approximately 27 psi at Junction 27 (J27) during irrigation demand with fire flow demand. The model shows that given the same supply pressure, Phase 2 has a low system pressure of approximately 20.8 psi at junction 6 (J6). With the proposed improvements scheduled in Phase 3, this situation is greatly improved because of the establishment of a looped system around J6 in Phase 3. The approximate residual pressure at J6 in Phase 3 is 44 psi. Distribution system pipe velocities are under 10 fps for the fire flow scenario.

7.4.5 Pressure System Interties/Zones

A single pressure zone is anticipated within the Project for both the potable and non-potable water systems based on the existing topography of the Project site and the proposed location of the water wells and storage tanks, considering the maximum pressure of 120 psi established for the system service criteria.

Several pressure zones requiring pressure reducing valves (PRVs) will be necessary for the proposed potable water transmission main from the WHWD treatment plant to the Project. PRVs will operate to maintain a preset downstream pressure independent of the upstream pressure. Further study is required to determine the limits of each pressure zone and the location of PRV valves. Figure 7.7 shows the layout of the offsite piping in relation to the proposed Project.

8.0 FINDINGS

8.1 SUPPLY

Given the inherent annual fluctuations in surface water annual deliveries, as well as the lack of existing entitlements or rights, utilization of surface water as the sole or primary supply source for the project is not considered a viable option. Given these considerations, use of groundwater as the sole or primary source of water for the project is considered the most viable and preferred alternative.

Available previous studies of groundwater elevations in the area indicate some decline in local groundwater elevations in recent years, especially between 2011 and 2014. However, this was a period of abnormally low rainfall throughout the state, which resulted in additional groundwater pumping to meet demands that would normally be met from surface water sources. A recent study by Jacobson, James, and Associates conducted in 2016 also indicate that, over time, groundwater elevations are relatively stable, which would indicate a hydrologically balanced condition.

Monitoring and additional studies, if needed, will be necessary to determine the specific effects the project would have on groundwater elevations and the sustainability of the aquifer. This monitoring will be required by the recent state Sustainable Groundwater Management Act (SGMA), as well as the recent 2014 Stanislaus County ordinance amendment. Currently, the CLIBP area is within the DPWD service area and will remain as such until the second phase of development of the CLIBP. During the second phase of development, land that is converted from agricultural to industrial use will be removed from the DPWD. The County has completed an SB 610 Water Supply Assessment that will examine historic and projected water demands and supplies that relate to the CLIBP. Stanislaus County is also working on a Storm Water Resource Plan, which will identify and prioritize projects within the County to address flood flow management and groundwater supply sustainability.

8.2 INFRASTRUCTURE

The following summarizes the results of the preliminary infrastructure system design and modeling:

- Total potable water demand for the buildout of the Project for ADD, MDD, and PHD are 1.34 MGD, 2.67 MGD, and 5.35 MGD, respectively.
- Total potable water demand for Phase 1A of the Project for ADD, MDD, and PHD are 0.15 MGD, 0.31 MGD, and 0.62 MGD, respectively.
- Total potable water demand for Phase 1 of the Project for ADD, MDD, and PHD are 0.59 MGD, 1.18 MGD, and 2.37 MGD, respectively.
- Total potable water demand for Phase 2 of the Project for ADD, MDD, and PHD are 0.35 MGD, 0.71 MGD, and 1.42 MGD, respectively.
- Total potable water demand for Phase 3 of the Project for ADD, MDD, and PHD are 0.41 MGD, 0.82 MGD, and 1.64 MGD, respectively.
- Required potable water storage volume for buildout of the Project is approximately 2.71 MGD.
- Required potable water storage volume for Phase 1 of the Project is approximately 1.19 MGD.

- Required potable water storage volume for Phase 2 of the Project is approximately 1.52 MGD. This total accounts for the additional storage needed for Phase 3.
- Required potable water storage volume for Phase 3 of the Project is approximately 0.82 MGD. This volume is shown for informational purposes only and is not in addition to the storage requirements shown for Phases 1 and 2.
- The potable water storage tanks will be located on-site.
- Total non-potable water demand for the buildout of the Project for irrigation demand is 1.20 MGD. A fire flow demand of 3,000 gpm is anticipated for the buildout of the project and will be supplied by the non-potable system but is considered separate from the irrigation average demand.
- Total non-potable water demand for Phase 1A of the Project for average day irrigation demand is 0.103 MGD. This flow does not account for the 3,000 gpm fire flow demand.
- Total non-potable water demand for Phase 1 of the Project for average day irrigation demand is 0.68 MGD. This flow does not account for the 3,000 gpm fire flow demand.
- Total non-potable water demand for Phase 2 of the Project for irrigation demand plus fire flow demand is 0.24 MGD. This flow does not account for the 3,000 gpm fire flow demand.
- Total non-potable water demand for Phase 3 of the Project for irrigation demand plus fire flow demand is 0.27 MGD. This flow does not account for the 3,000 gpm fire flow demand.
- The required non-potable water pump station for the Project should be sized to meet the projected maximum day irrigation demand. It is recommended that an additional set of pumps be constructed and sized to meet fire flow demands. These pumps should be separate from the irrigation pumps due to the large disparity between irrigation demands and fire flow demands. The non-potable water pump provided as part of Phase 1 improvements shall be sized to accommodate both the irrigation and fire flow demands at Project buildout. The water supply alternatives presented in Appendix C presents somewhat different phasing for non-potable water, including a second supply well in Phase 2.
- Required non-potable water storage volume for the Project is approximately 0.72 MGD. The total required non-potable water storage volume will be provided on-site as part of Phase 1 improvements.
- The potable and non-potable water systems within the Project will consist of a single pressure zone.
- Several pressure zones requiring pressure reducing valves (PRVs) will be necessary for the proposed potable water transmission main from the WHWD treatment plant to the Project.
- Potable water distribution piping will be a minimum of 12 inches in diameter in order to meet the criteria identified in Section 5.
- Non-potable water distribution piping ranges in size from 12 inches in diameter to 18 inches in diameter in order to meet the criteria identified in Section 5.

- All water supply alternatives presented in Appendix C include two water supply wells in Phase 1 and two wells in Phase 2.
- Option 1 in Appendix C combines the water systems of the CLCSD and the CLIBP under an existing water permit. This would also allow blending of water supplies from either district that may avoid the need for separate water or wellhead treatment.
- Option 2 in Appendix C includes on-site development of water supplies at CLIBP under a new water permit. Proposed infrastructure is similar to the original will Phase 1, 2 and 3 facilities included in this study with additional wells.
- Option 3 in Appendix C includes infrastructure identical to Option 2, plus an intertie corridor and pipeline to the COP with both districts under an existing water permit. All three of the alternatives are to be presented for consideration in the EIR.

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Figure 1.1: Crows Landing Industrial Park – Conceptual Phasing Map

APPENDIX A

Groundwater Resources Impact Assessment

Groundwater Resources Impact Assessment

Crows Landing Industrial Business Park
Stanislaus County, California

October 31, 2016

Stanislaus County
Striving to be the Best

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LIST OF ACRONYMS AND ABBREVIATIONS

AFY	acre-feet per year
amsl	above mean sea level
bgs	below ground surface
CEQA	California Environmental Quality Act
CLIBP	Crows Landing Industrial Business Park
CVP	Central Valley Project
DMGS	Delta-Mendota Groundwater Subbasin
DPWD	Del Puerto Water District
DWR	California Department of Water Resources
EIR	Environmental Impact Report
EPA	U.S. Environmental Protection Agency
ft/day	foot per day
GDE	groundwater-dependent ecosystem
gpd/ft^2	gallon per day per square foot
gpm	gallon per minute
gpm/ft	gallon per minute per foot
GSP	Groundwater Sustainability Plan
JJ&A	Jacobson James & Associates, Inc.
KDSA	Kenneth D. Schmidt and Associates
LID	Low Impact Development
MCL	Maximum Contaminant Level
MM	Mitigation Measure
NASA	National Aeronautics and Space Administration
SGMA	Sustainable Groundwater Management Act
SWP	State Water Project
SWRCB	State Water Resources Control Board
TDS	total dissolved solids
USGS	U.S. Geological Survey
UWMP	Urban Water Management Plan

1.0 INTRODUCTION

1.1 Background

Stanislaus County proposes rezoning of the former National Aeronautics and Space Administration (NASA) Crows Landing Air Facility to construct the Crows Landing Industrial Business Park (CLIBP), located in Stanislaus County south of Patterson, California (the Project). The CLIBP proposes to use groundwater as a water supply during construction and operation. This Groundwater Resources Impact Assessment Report has been prepared by Jacobson James & Associates, Inc. (JJ&A) on behalf of the Stanislaus County Department of Public Works, to provide information regarding groundwater resources that will be incorporated into the environmental analysis of the proposed Project under the California Environmental Quality Act (CEQA). Specifically, this report describes the affected groundwater resources environment, the groundwater resources demand and development activities associated with the proposed CLIBP, and the methods and results of a groundwater resources impact assessment for the proposed Project. The information contained in this report will be incorporated into the Environmental Impact Report (EIR) prepared for the Project.

1.2 Organization

This report includes the following sections:

- Chapter 1*, Introduction*, which identifies the background, purpose and scope of the study.
- Chapter 2, *Project Description*, which provides a brief overview of the proposed Project and discusses the anticipated water demand and proposed groundwater supply development activities.
- Chapter 3, *Project Setting*, which provides an overview of the project setting, with a particular focus on hydrogeology and groundwater resources.
- Chapter 4, *Drawdown Evaluation*, which presents the methods and results of an evaluation of the proposed groundwater extraction on groundwater levels and flow.
- Chapter 5, *Groundwater Resources Impact Analysis*, which presents a reasoned analysis of the potential impacts of the proposed groundwater supply development associated with the project on the environment.
- Chapter 6, *References*, which includes a list of documents cited in this report.

2.0 PROJECT DESCRIPTION

2.1 Project Overview

CLIBP is a conceptually planned development that encompasses the reuse of the former Crows Landing Air Facility, which was decommissioned by NASA in the late 1990s. The proposed CLIBP location is shown on Figure 2.1.1, and includes approximately 1,528 acres of land (hereinafter the Site). The proposed CLIBP layout is shown on Figure 2.1.2. The CLIBP is planned to include aviation, multimodal transportation, industrial and commercial facilities, which are proposed to be constructed on 1,261 developable acres in three phases:

- Phase 1 will be developed between 2017 and 2026, and includes construction of approximately 810 acres of aviation, multimodal, industrial and commercial facilities;
- Phase 2 will be developed from 2027 to 2036, and consists of construction of an additional 177 acres of multimodal, industrial and commercial facilities; and
- Phase 3 will be developed between 2037 and 2046, and includes construction of the final 274 acres of multimodal, industrial and commercial facilities.

2.2 Water Demand and Supply Development

A Water Supply Assessment and Water Supply Feasibility Study were prepared for the CLIBP by AECOM (AECOM, 2016a; AECOM and VVH Consulting Engineers, 2016). The water demand for the CLIBP will include potable, irrigation, fire water, and other non-potable water needs, and is proposed to be supplied from a combination of existing and new groundwater supply wells at the Site. As discussed further in Section 3.4, the groundwater resources beneath the Site that are available for supply development include a shallow unconfined aquifer that is separated from a deeper confined aquifer by a relatively impermeable regional aquitard layer referred to as the Corcoran Clay.

Table 2.2.1 below summarizes the projected water demand as the CLIBP is developed over time. The demand is presented as the estimated total at full buildout of each development phase. The project will develop a non-potable water supply using combination of the existing irrigation wells that derive water from both the shallow and deep aquifer, and new non-potable supply wells installed into the shallow aquifer beneath the Site. The project potable water supply will be developed using new wells installed into the confined aquifer beneath the Site.

Table 2.2.1 Project Groundwater Demand and Supply

Time Period	Annual Groundwater Demand at Completion of Each Buildout Phase (acre-feet/year [AFY])		
	Phase 1 2017 to 2026	**Phase 2 2027 to 2036**	**Phase 3 2037 to 2046**
Estimated Total Potable Demand	739	1,036	1,496
Estimated Total Non-Potable Demand	818	1,014	1,323
Estimated Total Project Demand	**1,557**	**2,053**	**2,819**
Potable Supply from New Confined Aquifer Wells	739	1,036	1,496
Non-Potable Supply from Existing Wells	818	834	834
Non-Potable Supply from New Shallow Aquifer Wells	0	183	489

The Project non-potable water supply will be developed as follows:

- As discussed further in Section 3.4.4 and summarized in Table 3.4.2, the three existing wells at the Site have historically been pumped at an average rate of approximately 834 acre-feet per year (AFY). It is assumed that the existing wells will be capable of supporting groundwater extraction at their historical annual extraction volumes when pumped year round. If the existing wells fail to supply the assumed 834 AFY, they would be supplemented, as needed, through the installation of new wells of similar construction.
- Any non-potable Project water demand in excess of 834 AFY is assumed to be supplied using new shallow aquifer wells that are installed at the Site.
- Optimal locations for the new shallow aquifer wells will be selected based on performance of the existing wells, groundwater level monitoring data developed during project operation, and additional water supply development studies, as needed.
- Shallow groundwater demand in excess of the historical average shallow aquifer extraction rate (183 AFY at Phase 2 buildout and 489 AFY at Phase 3 buildout) will be offset by an equivalent volume of increased recharge relative to current conditions, such that the net groundwater extraction rate from the shallow aquifer does not increase above historical levels. This increased shallow aquifer recharge will be derived from a combination of the following sources:[1]
    - Discharge from Little Salado Creek and Marshall Drain will be captured and recharged at facilities constructed for the CLIBP. A long, linear stormwater retention/detention basin

[1] Mitigation Measure (MM) Water-04, described in Section 5.6.4, requires that a Recharge Enhancement Plan be prepared that describes how the Project will achieve sufficient recharge to fully offset any additional groundwater demand on the shallow aquifer imposed by the Project.

will be constructed on the north side of the Site by widening approximately 4,000 feet of Little Salado Creek and Marshall Drain from the current width of approximately 15 feet to over 250 feet, and modifying the streambed to increase its permeability (AECOM, 2016b). The basin will be designed for retention of 200 acre-feet (the estimated runoff volume of a 2-year storm event) and detention of an additional 180 acre-feet. Based on the available information, it is reasonable to expect that several hundred acre-feet per year of groundwater can be recharged to the shallow aquifer in these facilities compared to current conditions.[2]

- Developments within the CLIBP will be required to implement Low Impact Development (LID) standards that promote on-Site stormwater retention and recharge (AECOM, 2016c). Design Goal D-25 requires that all stormwater be retained on the individual lease holds (parcels) to be developed at the CLIBP. This will result in additional recharge relative to the current condition.[3]
- Developments within the CLIBP will be required to employ landscape planting strategies and xeriscape designs to decrease non-potable water demand. The non-potable water demand estimate presented in Table 2.2.1 is based on conservative default development assumptions in Stanislaus County (AECOM, 2016a; AECOM and VVH Consulting Engineers, 2016), and does not consider the implementation of xeriscape planting standards. It is reasonable to assume that landscaping associated with project buildout using these methods can result in a non-potable water demand reduction of several hundred acre-feet, which may be considered net *in lieu* recharge to the shallow aquifer.

The CLIBP potable water supply is assumed to be developed as follows:

- It is assumed that the new water supply wells will be installed into the confined aquifer underlying the Corcoran Clay at the approximate locations shown on Figure 2.1.1. The potable supply wells will be constructed to pump water from the full usable depth of this aquifer. On a preliminary

[2] For perspective, the Little Salado Creek watershed occupies an area of approximately 10.8 square miles and has an average annual discharge of approximately 874 AFY (AECOM, 2016b). The reported discharge in Marshall Drain ranged from 1,147 to 2,731 AFY between 2005 and 2011 (Summers Engineering, 2013), and includes discharge from Little Salado Creek and local agricultural drainage, minus any existing recharge. Recharge from streams is proportional to streambed conductance, which is the product of the streambed thickness and width, times its vertical hydraulic conductivity. The proposed construction of the project retention/detention basin will increase the streambed width by at least an order of magnitude, and modify the bed of the basin to increase its permeability. It is reasonable to assume that construction and maintenance of the basin can increase its conductance by approximately two orders of magnitude, increasing the recharge through the basin by approximately 100-fold relative to the existing condition.

[3] Based on a screening-level evaluation using the U.S. Environmental Protection Agency (EPA) National Stormwater Calculator (EPA, 2014) presented in in Appendix A, it is anticipated that application of LID elements in site-specific construction can capture and infiltrate up to approximately 200 AFY of stormwater relative to Project buildout without parcel-specific LID elements. A detailed analysis relative to current conditions has not been performed, so the amount of recharge compared to current conditions may be different; however, the analysis indicates that significant recharge can be achieved through the implementation of LID elements.

basis, screen intervals are assumed to extend from approximately 320 to 870 feet below ground surface (bgs).

- Groundwater extracted from the confined aquifer for potable use will be treated to meet applicable water quality standards.

2.3 Applicable Regulations

The Site is not located in an adjudicated basin or in a special act district that regulates the extraction of groundwater. The Project would be able to supply groundwater for beneficial use on the properties to be developed in the business park under an appropriative groundwater right. No new entitlements would be required.

Development of groundwater resources to support the Project must comply with the Stanislaus County Groundwater Ordinance adopted in November 2014 (Chapter 9.37 of the Stanislaus County Code), which codifies requirements, prohibitions, and exemptions for permitting new wells with the intent of supporting sustainable groundwater extraction. In addition, the Project will have to comply with the requirements of a Groundwater Sustainability Plan (GSP) that will be adopted for the area by 2020 under California's new Sustainable Groundwater Management Act (SGMA). Stanislaus County's Groundwater Ordinance is deliberately aligned with the requirements of SGMA. Under the Ordinance, unless otherwise exempt, an applicant that wishes to install a new groundwater well must first provide substantial evidence the well is not unsustainably extracting groundwater as defined in the Ordinance and in SGMA. The County has determined that the CLIBP is not exempt from these requirements. The Ordinance and SGMA define unsustainable extraction as causing undesirable results, which are defined as meaning one or more of the following:

a. Chronic lowering of groundwater levels indicating a significant and unreasonable depletion of supply if continued over the planning and implementation horizon. Overdraft during a period of drought is not sufficient to establish a chronic lowering of groundwater levels if extractions and recharge are managed as necessary to ensure that reductions in groundwater levels or storage during a period of drought are offset by increases in groundwater levels or storage during other periods.

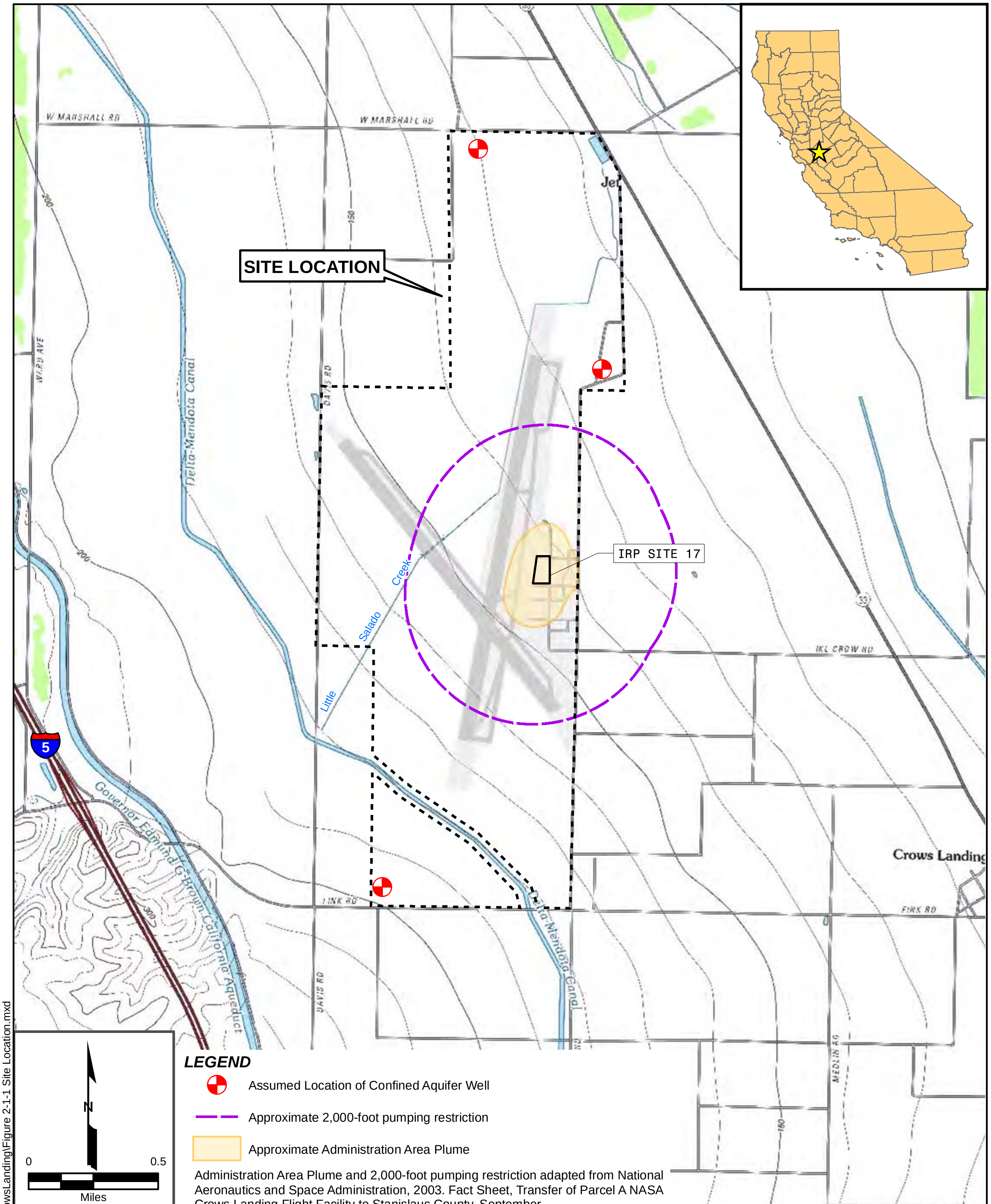
b. Significant and unreasonable reduction of groundwater storage.

c. Significant and unreasonable degraded water quality, including the migration of contaminant plumes that impair water supplies.

d. Significant and unreasonable land subsidence that substantially interferes with surface land uses.

e. Surface water depletions that have significant and unreasonable adverse impacts on beneficial uses of the surface water.

Prior to issuing a permit to construct a new groundwater supply well, the County must review information provided by the applicant and make a determination whether it constitutes substantial evidence that the proposed groundwater extraction will not cause or contribute to one or more of the above undesirable results. To that end, it should be noted that the undesirable results listed above are aligned with questions contained in Appendix G of the State CEQA Guidelines, which are evaluated in Section 5.0 of this report. As such, this report fulfills the substantial evidence requirement for demonstrating compliance with the sustainable groundwater management requirements in the Stanislaus County Groundwater Ordinance.



Path: J:\GIS\StanislausCounty\CrowsLanding\Figure 2-1-1 Site Location.mxd

JACOBSON | JAMES
& associates, inc

TITLE:

SITE MAP

DATE:

8/2/16

LOCATION:

CROWS LANDING INDUSTRIAL BUSINESS PARK
STANISLAUS COUNTY, CALIFORNIA

FIGURE:

2.1.1

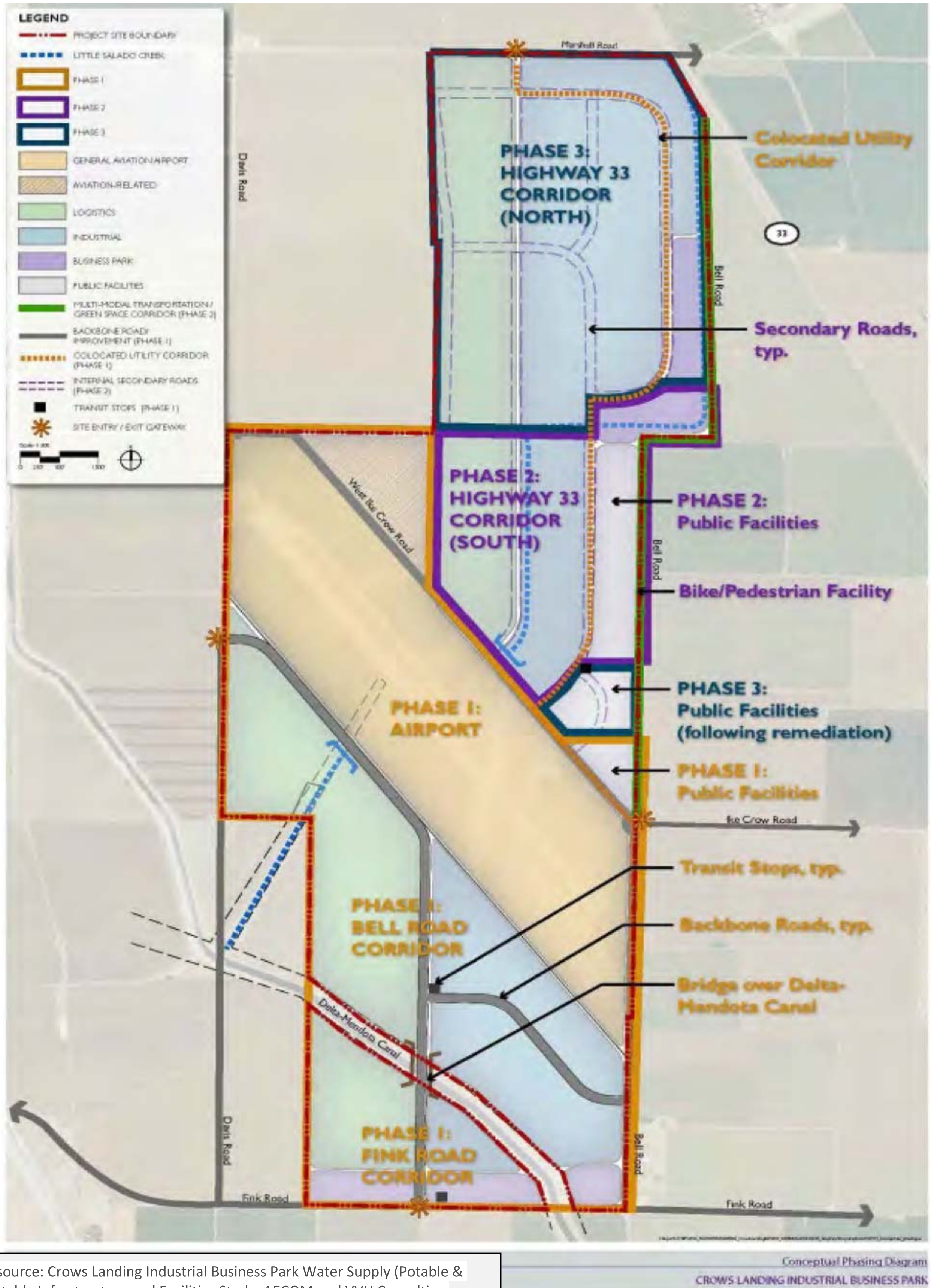


Figure source: Crows Landing Industrial Business Park Water Supply (Potable & Non-Potable Infrastructure and Facilities Study. AECOM and VVH Consulting Engineers, updated February 17, 2016.

JACOBSON | JAMES & associates, inc	TITLE: ***PROPOSED FACILITY LAYOUT***	DATE: 07/28/16
	LOCATION: *Crows Landing Industrial Business Park* *Stanislaus County, California*	FIGURE: **2.1.2**

3.0 PROJECT SETTING

3.1 Existing Site Conditions and Topography

The Site is located in a predominantly agricultural area of rural Stanislaus County. It is located east of Interstate 5, west of State Route 33, south of the City of Patterson, and approximately 1 mile west of the unincorporated community of Crows Landing. It is bounded on the east by Bell Road, on the south by Fink Road, on the west by Davis Road, and on the north by Marshall Road and State Route 33. The Delta-Mendota Canal traverses the southern portion of the Site in a northwest/southeast direction. The Site is occupied by abandoned runways, taxiways, buildings and other facilities associated with the former Crows Landing airfield, surrounded by approximately 1,200 acres of cultivated agricultural land. Paved and unpaved access roads traverse the Site

Physiographically, the Site is located on the San Joaquin Valley floor, approximately 1 to 2 miles east of the Diablo Range, and 4 to 6 miles west of the San Joaquin River. The western margin of the valley consists of low hills and dissected alluvial fans at the foot of the Diablo Range. A short distance to the east, elevations drop off into alluvial and flood plains associated with the San Joaquin River. The Delta-Mendota Canal and California Aqueduct run along the western margin of the valley. The Site slopes gently to the northeast from a high elevation of approximately 180 feet above mean sea level (amsl) near the southwest Site corner to approximately 110 feet amsl near the intersection of State Route 33 and Marshall Road.

3.2 Climate

The area has a "Mediterranean" climate characterized by hot, dry summers and short, wet winters, and averages over 260 sunny days per year. The average annual precipitation at the Modesto meteorological station is just over 13 inches per year, with 88 percent of the precipitation occurring between November and April (Turlock Irrigation District, 2012; Sperling's Best Places, 2016).

Much of California, including the Central Valley, has experienced unprecedented drought conditions over the last four years. As a result, water conservation measures have been mandated, delivery of surface water from the state and federal water systems has been curtailed, and reliance on groundwater resources for agricultural uses has increased.

3.3 Surface Hydrology

Drainage in the Site vicinity is generally toward the northeast, from streams draining the Diablo Range and along the natural slope of the valley floor toward the San Joaquin River. Drainage from the agricultural fields and airfield areas of the site is routed to Little Salado Creek, which traverses the Site in a northeasterly direction. Little Salado Creek is an ephemeral stream that drains the eastern slope of the Diablo Range, and discharges to Marshall Drain near the northeast corner of the Site. Marshall Drain transitions to an underground pipe near the intersection of Marshall Road and State Route 33. The average annual discharge on Little Salado Creek is estimated to be approximately 874 AFY (AECOM and VVH Consulting Engineers, 2016).

The dissected alluvial terrace deposits west of the Site at the base of the coast range generally do not contain shallow groundwater; however, due to their coarse grained nature, they are considered potentially important for groundwater recharge. When sufficient runoff occurs, it eventually drains to the San Joaquin River, approximately 4 to 6 miles east of the Site.

3.4 Hydrogeology

The Site is located in the Delta-Mendota Groundwater Subbasin (DMGS) of the San Joaquin Valley Groundwater Basin. Within Stanislaus County, the DMGS is bounded to the east by the San Joaquin River and to the west by low-permeability bedrock of the Coast Ranges that is associated with Tertiary and older marine formations. The subbasin extends southward from the northern boundary of Stanislaus County along the west side of San Joaquin Valley for approximately 80 miles, and crosses a total of four counties, encompassing an area of approximately 747,000 acres. The total estimate storage capacity of the DMGS is 30,400,000 acre feet to a depth of 300 feet, and 81,800,000 acre feet to the base of fresh groundwater (California Department of Water Resources [DWR], 2006).

Groundwater in the DMGS occurs in the Tulare Formation and overlying Quaternary and Holocene alluvium, terrace deposits and flood basin deposits. The Tulare Formation extends to a depth of over 1,000 feet, and includes beds, lenses, and tongues of clay, sand, and gravel that have been alternately deposited in oxidizing and reducing environments. It also includes a number of lacustrine clay units (DWR, 2013), the most prominent of which is known as the Corcoran Clay and acts as a regional aquitard that divides the basin fresh water deposits into an upper aquifer system that is unconfined to semi-confined, and a lower aquifer system that is confined (DWR, 2013). The Corcoran Clay is reported to occur at depths between approximately 200 and 250 feet near the Project Site, and extends from near the western margin of the subbasin to beneath the San Joaquin River. Groundwater production wells are completed in both the unconfined and confined aquifer systems; however, most high-capacity wells extend into the confined aquifer system. Domestic wells in the area are generally completed in the unconfined aquifer system.

As of 2006 (before the current drought), urban and agricultural groundwater extraction was estimated to be 508,000 AFY for the DMGS (DWR, 2006). An operational yield study by the City of Patterson estimated that the city could pump up to 12,000 AFY without significantly impacting the use of groundwater resources in the area surrounding Patterson's sphere of influence (RMC, 2016). The City of Newman pumped approximately 4,200 acre-feet of water in 2012 (Kenneth D. Schmidt and Associates [KDSA], 2013).

3.4.1 Groundwater Levels and Flow

The freshwater aquifers that are important to this study comprise approximately the upper 950 feet of sediments in this area. Groundwater levels are reported to range from approximately 30 to 50 feet bgs, and groundwater flow is generally toward the northeast, toward the San Joaquin River (DWR, 2016b). The reach of the San Joaquin River near the Site is hydraulically connected to the local shallow aquifer system

(State Water Resources Control Board [SWRCB], 2015); however, based on the depth to groundwater near the Site, it is unlikely that surface water resources and groundwater-dependent ecosystems (GDEs) in this area are connected to a regional groundwater table.

Groundwater elevation contour maps for the confined aquifer in the Site vicinity from 2011 to spring 2016 are provided as Appendix B. The contour maps show a groundwater ridge or mound persists opposite Little Salado, Salado, and Orestimba Creeks, which suggests recharge occurs along the mountain front. The contour maps show that in recent years, cones of depression have formed northwest and south of the Site, and locally influence the groundwater flow direction. The cones of depression appear most pronounced in the groundwater elevation contour maps from 2014 through 2016, particularly in the fall. This timing coincides with reductions of Central Valley Project (CVP) and State Water Project (SWP) surface water deliveries to local water providers in response to historic drought conditions (see Table 3.4.2). The cone of depression to the south is located northwest of Newman, near the northern portion of the Eastin Water District, which derives its water supply entirely from groundwater. A trend toward conversion of crop land to orchards in this area, as well as surrounding areas served by Del Puerto Water District (DPWD), was observed based on review of aerial imagery from the last 10 years (Google Earth, 2016). As such, this cone of depression may relate to an increase in pumping from the confined aquifer in response to increasing demand as the orchards matured, coupled with hardened demand that was not met from surface water deliveries.

The cone of depression to the northwest of the Site is consistent with reported groundwater pumping from the confined aquifer northwest of Patterson for irrigation purposes. Hydrogeologic conditions in this area are described in a report for the Arambel Business Park (KDSA, 2013). Groundwater pumping for irrigation from confined aquifer wells northwest of Patterson reportedly influence the groundwater flow direction (i.e., create drawdown in the confined aquifer). Most recharge in this area is associated with CVP surface water deliveries, as recharge from west side streams and rainfall is generally small. In 2010, more than half of the water applied for irrigation in this vicinity was from surface water deliveries, with the rest of the demand met from groundwater pumping. Curtailment of surface water deliveries in recent years due to drought conditions may have led to increased pumping from the confined aquifer to meet agricultural demand, while reducing a significant source of groundwater recharge. These conditions may explain the cone of depression observed northwest of the Site.

Groundwater hydrographs for several wells near the Site that are reported or assumed to be screened within the confined aquifer and for which long term hydrographs were retrieved from the DWR's California Statewide Groundwater Elevation Monitoring (CASGEM) website and are shown on Figure 3.4.1 (DWR, 2016d). Analysis of long terms hydrographs in the region south of the Site indicates that groundwater levels in the area were generally lowest in the 1940's and 1950's, increased during the 1960's and 1970's when surface water became available from the state and federal water projects, and decreased through the 1990's and 2000's, when surface water deliveries began to be curtailed for environmental reasons. Shorter term trends were identified related to periods of above or below normal precipitation. The two wells located south of the Site, near the cone of depression northwest of Newman, show a recent decreasing trend that may relate to current drought conditions and increased groundwater pumping to replace curtailment of surface water deliveries. It is noteworthy that current groundwater levels in the well with the longest period of record (State Well No. 06S08E29J001M) are approximately 40 feet above their historical low level in October 1952. Groundwater levels in State Well No.'s 07S08D14D001M and 06S08E34M001M are at their historical low levels; however, water level data are not available for these wells prior to October 1958 and March 1959, respectively.

The hydrographs for State Well No.'s 06S08E20D002M and 06S08E09E001M span the period from 2011 to the present. In general, these hydrographs suggest that groundwater levels near the Site recovery quickly after pumping ceases, as evidenced by relatively consistent water elevations by season (see State Well No. 06S08E09E001M on Figure 3.4.1). Water levels near the Site have overall been stable over the period of record (since 2011), which indicates recent pumping rates near the Site have been sustainable on an annual basis, even during the drought.

3.4.2 Aquifer Properties

DWR has estimated the average specific yield of the water-bearing sediments in the DMGS as 11.8 percent (DWR, 2006). The permeability of the shallow groundwater-bearing strata in the Site vicinity is reported by local drillers to be variable (Ward, personal communication, 2016). The rancher that currently farms the land at the Site uses three production wells (Wheeler, personal communication, 2016). Two of these wells are completed in the shallow aquifer system overlying the Corcoran Clay, to a depth of approximately 210 feet bgs. One of these shallow wells has not been a reliable groundwater producer, and the yield from this well has reportedly decreased over time. When it was originally rehabilitated by the current user and placed back into service, it reportedly produced groundwater at a rate of approximately 900 gallons per minute (gpm) at the beginning of the irrigation season, decreasing to approximately 450 gpm by the end of the irrigation season. However, the yield from this well has reportedly decreased from year to year, and in 2015, this well reportedly did not produce a significant amount of groundwater. The second shallow well is reliably pumped continually throughout the irrigation season; however, the well yield typically decreases from approximately 1,400 gpm at the beginning of the season to approximately 400 gpm at the end of the season. The third existing well at the Site is completed to a depth of approximately 495 feet bgs, with two screened intervals. This well has consistently produced groundwater at a rate of approximately 900 gpm throughout the irrigation season, suggesting that most or all of the groundwater pumped from this well is derived from the confined aquifer below the Corcoran Clay. The rancher that currently farms the land indicated that the water quality from this well is distinct from the other two shallow wells, and contains more boron. This observation would be consistent with most of the water from this well coming from the confined aquifer.

Estimated transmissivities are available for seven wells near Patterson to the north of the Site, and seven wells near Newman, southwest of the Site (KDSA, 2010 and 2013). These 14 wells are reportedly screened entire within the confined aquifer, or in the confined and shallow aquifer ("composite" wells). In addition, specific capacity tests for two nearby confined aquifer wells were evaluated by Stanislaus County

Department of Environmental Resources and the results provided to JJ&A. An evaluation of aquifer parameters based on these tests is presented in Table 3.4.1. The estimated hydraulic conductivity for the confined and composite aquifers ranged from 13 to 117 feet per day (ft/day), with a geometric mean of 45 ft/day and a 10^{th} percentile value of 17 ft/ day. By comparison, results from a 72-hour pumping test Patterson City Well No. 7 yielded an average hydraulic conductivity for the confined aquifer of 40 feet/day (KDSA, 2013).

The vertical hydraulic conductivity of the Corcoran Clay near the site is not known, but a reasonable range based on the literature is approximately 6.2 E-04 to 3.0 E-06 ft/day (USGS, 2009; USGS, 2004).

The storativity of the confined aquifer from the Patterson City Well No. 7 pumping test was 0.0003 (KDSA, 2013). This is similar to the results of a pumping test conducted by Kleinfelder at a similar location approximately 12 miles to the north, which was 0.0001 (Kleinfelder, 2016).

Table 3.4.1 Aquifer Properties Estimated from Specific Capacity Tests

Well	Screen Aquifer	Screen Interval Span (feet)	Reported Specific Capacity (gpm/ft)	Estimated Transmissivity (gpd/ft^2)	Estimated K for Screen Interval Span (ft/day)
Patterson City Well 2	Composite	190	42	71,400	50
Patterson City Well 4	Composite	225	19	32,300	19
Patterson City Well 5	Confined	175	42	84,000	64
Patterson City Well 6	Composite	130	15	25,500	26
Patterson City Well 7	Confined	267	21	42,000	21
Patterson City Well 8	Confined	140	59	118,000	113
Patterson City Well 11	Confined	220	45	90,000	55
Newman City Well 2	Composite	247	77	130,900	71
Newman City Well 3	Composite	270	65.1	110,670	55
Newman City Well 4	Composite	322	77.8	132,260	55
Newman City Well 13	Composite	315	92.1	156,570	66
Newman City Well 36	Composite	303	32.9	55,930	25
Newman City Well 42	Composite	301	64.2	109,140	48
Newman City Well 53	Composite	300	51.3	87,210	39
6S/8E-6Q (WCR#788583)	Confined	180	20.9	41,800	31
6S/8E-21R(WCR#82200)	Confined	190	9.4	18,800	13

3.4.3 Groundwater Quality

Generally, groundwater quality in the basin is suitable for most urban and agricultural uses, with primary constituents of concern consisting of total dissolved solids (TDS), nitrate, boron, chloride, and organic compounds (DWR, 2003). Areas of high TDS concentrations are primarily found in the western region of the valley, due to the recharge of streamflow originating from the marine sediments in the nearby Coast

Ranges, while high concentrations of boron are typically found in the valley trough as the results of salts, due to evaporation and poor drainage (DWR, 2003). Sulfate and boron concentrations vary in both the shallow and confined aquifers, with slightly higher boron concentrations in the confined aquifer; there is little difference in arsenic concentrations between the shallow and confined aquifers. Nitrate, nitrite, hexavalent chromium, and 1,2,3-trichloropropane have been detected at concentrations above the Maximum Contaminant Levels (MCL) in groundwater from the Crows Landing Community Services District area surrounding the Site (AECOM and VVH Consulting Engineers, 2016).

The Navy maintains a 2,000 foot pumping restriction at the Crows Landing Air Facility around a contamination plume known as the IRP Site 17 Administration Area Plume (see Figure 2.1.1) (AECOM and VVH Consulting Engineers, 2016). The contamination plume is the result of underground fuel storage tanks, used for the former facility, and includes benzene and other volatile organic compounds. The plume contaminants appear to be limited to the shallow aquifer, above the Corcoran Clay.

3.4.4 Groundwater Budget and Existing Groundwater Demand

Development of a complete groundwater budget and demand inventory is beyond the scope of this study; however, the following information is pertinent to this analysis. DWR has listed the DMGS as being in a state of overdraft, though groundwater levels in the vicinity of the Site are generally stable (Section 3.4.1). A study of groundwater level trends from 1993 to 2008 found that groundwater levels in northern portions of the DMGS were generally hydrologically balanced (AECOM, 2011). The study found minimal apparent net change in groundwater elevations, which were interpreted as equilibrium between use and recharge. However, consistent declines in groundwater levels in certain localized areas (including an area west of Newman), may be indicative of a developing local overdraft condition. This is consistent with groundwater elevation contours and hydrographs for the Site vicinity, as discussed in Section 3.4.1.

Land use overlying the DMGS near the Site is primarily agricultural, with local agricultural water demand served by surface-water deliveries from DPWD, supplemented by groundwater extraction. Municipal water demand for the Cities of Patterson and Newman, as well as the community of Crows Landing, is met using groundwater. Demand forecasts are available for the City of Patterson from the 2015 update to its Urban Water Management Plan (UWMP) (RMC, 2016). The demand is projected to increase from 6,376 AFY in 2020 to 11,801 AFY in 2040. Similar proportional increases in demand may also be expected in the communities of Newman and Crows Landing if they follow similar population and development trends. However, it is important to note that increased municipal demand would be expected to be offset by a corresponding decrease in agricultural demand associated with conversion of agricultural land to municipal use.

Groundwater demand for agricultural production at the Site has historically been met through a combination of groundwater pumping and surface deliveries from DPWD. Information regarding the total applied water volumes and groundwater pumpage for on-Site wells for the last five years was provided by the rancher that farms the property and is summarized in Table 3.4.2, below.

Table 3.4.2 Historical Site Groundwater Pumpage and Surface Water Deliveries

Year	Volume of Groundwater Extracted (acre-feet) [1]			Volume of Surface Water Delivered (acre-feet) [2]	Percent of CVP Contract Allotment Available [2]	Total Applied Water (acre-feet)
	Deep Well	Shallow Wells	Total			
2012	380	560	940	1,629	40%	2,569
2013	402	448	850	424	20%	1,274
2014	390	212	602	158	0%	760
2015	564	378	942	0	0%	942
Average	434	400	834	553	15%	1,386

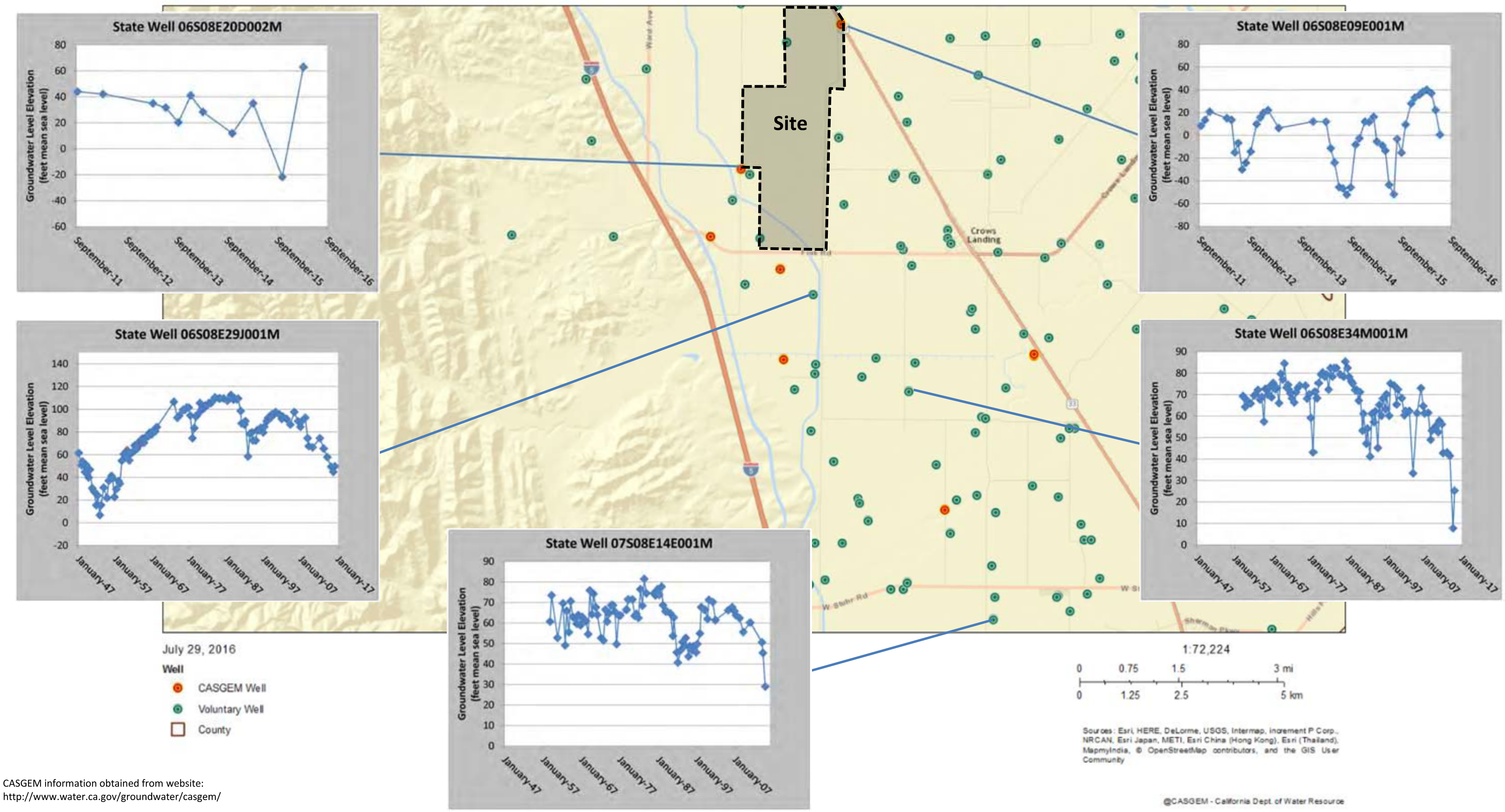
1. Based on information reported in AECOM, 2016 or data provided by Wheeler, 2016. Where confliciting data were provided, extraction volumes reported in AECOM, 2016 were utilized and divided among the wells in proportion to reported pumping rates.

2. Taken from Water Use Statements from Del Puerto Water District provided by Wheeler, 2016.

3.5 Subsidence

Land subsidence can occur when compressible clays are depressurized as a result of groundwater extraction, triggering water to flow from the clays into the surrounding aquifer, and ultimately consolidation of the clay under pressure from the overlying sediments. This can happen especially in confined aquifer conditions such as below the Corcoran Clay, where the head loss resulting from groundwater extraction is greater than in unconfined aquifers. The process of subsidence is reversible when granular aquifer materials compress and expand under changing pressure conditions, but irrecoverable when clay frameworks are compressed and reoriented. Irrecoverable subsidence results in decreased storage capacity within the aquifer. In general, most subsidence occurs when an aquifer is initially depressurized, but can continue for months, or even years, after clays slowly dewater and adjust to the new pressure regime. If groundwater levels subsequently recover, subsidence generally does not resume (or does not progress as rapidly), until groundwater levels fall below historical low levels.

DWR has included the DMGS on the list of critically overdrafted basins, largely due to overdraft and subsidence reported outside Stanislaus County to the south (DWR, 2016a); nevertheless DWR has designated the entire DMGS as having a high potential for future subsidence (DWR, 2016b). The Bureau of Reclamation, in cooperation with DWR, monitors a geodetic survey network of triangulated elevation monitoring of benchmarks in the area surrounding the San Joaquin River from Fresno to Patterson, including locations along the Delta-Mendota Canal (U.S. Bureau of Reclamation [USBOR], 2014). Survey data from this program indicate a subsidence rate of 0 to 0.15 feet (0 to 1.8 inches) per year from December 2011 to December 2015 near the Site, including areas surrounding Patterson and Newman (USBOR, 2016). More rapid short-term subsidence rates were reported from December 2012 through December 2013, ranging from 0.15 to 0.3 feet per year (USBOR, 2014). This is consistent with DWR's report of 1 to 2.5 inches of subsidence from 2005 to the present at continuous survey station P259, located near the northeast corner of the Site at the intersection of Marshall Road and State Highway 33 (DWR, 2016b).



CASGEM information obtained from website:
http://www.water.ca.gov/groundwater/casgem/

JACOBSON | JAMES
& associates, inc

Crows Landing Industrial Business Park
Stanislaus County, California

PROJECT NO.	DATE	DRAWN BY	APPR. BY
STANCO.001	7/28/16	JH	MT

FIGURE 3.4.1

Hydrographs for Confined Aquifer Wells Located Near the Site

4.0 EVALUATION OF HYDROGEOLOGIC EFFECTS

To evaluate the potential effects of the CLIBP on groundwater resources, an analytical groundwater modeling study was performed to assess the potential impacts of pumping on groundwater levels at the Site and in the surrounding area under a range of scenarios that bracket the current uncertainty regarding aquifer conditions. The analytical modeling study was based on the conceptual understanding described in Section 4.1, and implemented as described in Section 4.2. The results are presented in Section 4.3.

4.1 Conceptual Understanding

The modeling study is based on the following working conceptual understanding of groundwater occurrence and flow in the vicinity of the Site:

- Bedrock of the Diablo Range, located approximately 1 to 2 miles west of the Site, forms a no-flow boundary for the alluvial aquifers underlying the DMGS.
- In the Site area, groundwater occurs in a two-aquifer system, including an upper unconfined aquifer and a lower confined aquifer. These two aquifers are separated by the Corcoran Clay, a regionally extensive aquitard that occurs at a depth of approximately 250 feet bgs, with an average thickness of approximately 70 feet, based on data provided by Stanislaus County.
- The base of freshwater aquifers in this area is reported to occur at an elevation of approximately 800 feet below sea level (approximately 950 feet bgs) (Page, 1973). The confined aquifer system available for development by the CLIBP is therefore assumed to extend from approximately 320 to 870 feet bgs, for a total thickness of approximately 550 feet.
- Mountain front recharge occurs near the western edge of the subbasin, where streams draining the Diablo Range emerge onto small alluvial fans at the edge of the valley. The Corcoran Clay may be absent or discontinuous in this area (AECOM, 2011), so it is possible that some recharge percolates directly into the confined aquifer in this area.
- Regional groundwater flow is toward the northeast, away from the Diablo Range and toward the San Joaquin River, approximately 4 to 6 miles east of the Site (see Appendix B). This flow pattern has been locally disrupted by cones of depression located north and south of Site vicinity, which have expanded since 2013 during drought conditions.
- In the vicinity of the Site, groundwater levels have consistently recovered each year after the irrigation season, and a recurrent groundwater mound at the mountain front near Little Salado Creek and Salado Creek suggests a persistent inflow of recharge from this area restores groundwater levels and the prevalent flow direction in this area (see Figure 3.4.1 and Appendix B). This suggests that groundwater recharge and discharge are generally balanced in this area.
- Groundwater levels along the mountain front west of the Site are reported to be approximately 110 feet bgs near Crow Creek (southwest of the Site), and decreasing to approximately 30 feet bgs near

Del Puerto Creek (northwest of the Site),where a cone of depression appears to have formed during recent drought years (see Appendix B).

- Groundwater levels near the San Joaquin River are generally close to the elevation of the river, suggesting that this reach of the river is hydraulically connected with the shallow aquifer. Groundwater contours near the river suggest that shallow groundwater is discharging to the river, especially in the area to the southeast of the Site.
- Transmissivity data from municipal wells in Patterson and Newman that are screened within the confined aquifer indicate the lateral hydraulic conductivity ranges from 19 to 113 ft/day (see Table 3.4.1). Hydraulic conductivity calculations based on these data indicate a mean of 47 ft/day, a geometric mean of 41 ft/day, and a 10^{th} percentile of 17 ft/day. The hydraulic conductivity is assumed to be the same in the shallow and confined aquifers.
- Pumping test data from Patterson City Well No. 7 and an irrigation well located in a similar setting approximately 12 miles to the north indicate the storativity of the confined aquifer ranges from 0.0001 (Kleinfelder, 2016) to 0.0003 (KDSA, 2013). The storativity in the Corcoran Clay is assumed to be the same as for the confined aquifer. The storativity in the shallow aquifer near the Site is not known, but a reasonable value based on our experience is approximately 0.04.
- DWR (2006) estimated the specific yield for the DMGS to be 11.8; this value was used for the shallow and confined aquifers.
- The vertical hydraulic conductivity of the Corcoran Clay near the site is not known, but a reasonable range based on the literature is approximately 1.0 E-04 to 3.0 E-06 ft/day (USGS 2004 and 2009).

4.2 Analytical Drawdown Model

4.2.1 Approach

An analytical model was constructed to evaluate the reasonable range of drawdown that could occur from groundwater extraction related to development of the CLIBP. The model was constructed using the AnAqSim modeling code (Fitts Geosolutions, 2016), a three-dimensional (multi-layer) analytical element modeling code capable of simulating groundwater flow to wells under confined, unconfined, or semi-confined aquifer conditions. AnAqSim is able to simulate a variety of boundary conditions (e.g., no-flow, constant flux, variable flux, general head, and constant head), line or area sources and sinks (e.g., rivers and recharge), and flow barriers. AnAqSim can be used to simulate transient conditions as a result of pumping from single or multiple wells at constant or varying rates, and calculates the head and discharge as functions of location and time across a designated model grid or at designated points.

Four modeling scenarios were developed using a superposition approach to simulate drawdown under a reasonable range of conditions. Superposition or impact modeling is a robust modeling approach which focuses on evaluation of drawdown as opposed to actual hydraulic head, and allows the modeler to focus more on the evaluation of the changes introduced by a project, rather than the simulation of past or future groundwater levels (Reilly, Franke and Bennett, 1987). The use of superposition modeling in hydrogeologic literature is well established and this approach has been widely used to evaluate the impacts of water supply pumping.

For each of the modeling scenarios, a baseline model was constructed to simulate a set of aquifer conditions representing reasonable end point assumptions. The model was then run in transient mode with simulated pumping from the project wells, and resulting water level surface was subtracted from the baseline to evaluate the drawdown induced by the project at the end of Phase 1, Phase 2 and Phase 3 of the Project. The model inputs and supporting rationale are discussed below and summarized in Table 4.2.1. The model domain and boundaries are shown graphically in Figure 4.2.1, and model layering is shown in Figure 4.2.2.

Model Domain and Layering. For this evaluation, a model domain was established that measures approximately 75,000 by 50,000 feet that is approximately centered on the Site. The model domain was divided into two subdomains. The eastern subdomain includes three layers representing the shallow unconfined aquifer, the Corcoran Clay, and the lower confined aquifer. The western subdomain consists of a narrow strip on the west side of the model domain (the "forebay"), which was constructed as a single layer separated from the rest of the model domain by an inter-domain boundary; the forebay represents mountain-front sediments where the Corcoran Clay may or may not be present as a confining layer. The San Joaquin River was incorporated into the model with a direct connection to the shallow aquifer subdomain. Spatially-variable area sink/source polygons were constructed to model groundwater recharge around the San Joaquin River and groundwater extraction from the three assumed new confined aquifer wells at the CLIBP. This approach was selected because the software and domain configuration allow for modeling of drawdown in any of the subdomains (the focus is on the confined aquifer) at different phases of Project buildout with the ability to vary aquifer characteristics and boundary conditions that bracket the current uncertainty regarding aquifer conditions.

Boundary Conditions. General head boundaries were simulated on north, east, and south the east sides of the model domain. General head conditions were selected based on groundwater elevations from contour maps for the project vicinity (Appendix B). The western boundary of the model domain was simulated in two different ways to bracket the current uncertainty regarding the persistence of the Corcoran Clay in this area (see Figure 4.2.2):

- In Scenarios 1 and 2, the western boundary of the forebay was defined as a no-flow boundary along the mountain front, with surface recharge to the forebay. For these scenarios, the forebay subdomain was extended to a depth of 300 feet bgs, and water was allowed to flow laterally directly from the forebay into the Corcoran Clay and the lower confined aquifer (direct recharge condition).
- In Scenarios 3 and 4, the western boundary of the forebay was defined as a constant head boundary, with the assigned heads based on average historical groundwater elevations along the western margin of the basin over the last five years (Appendix B). For these scenarios, the depth of the forebay subdomain was identical to the shallow aquifer depth, and lateral groundwater flow was allowed from the forebay only into the shallow aquifer. Under these scenarios, the only path

by which mountain front recharge may enter the lower confined aquifer is via percolation through the Corcoran Clay (no direct recharge condition).

Line Sinks. The San Joaquin River was simulated as a line sink with direct connection to the shallow aquifer. The river stage was set based data from USGS gaging stations "SMN" (San Joaquin River above the Merced River near Newman) and SCL (San Joaquin River near Crows Landing) (DWR, 2016c).

Aquifer Characteristics. The aquifer was modeled as a 3-layer domain with the Corcoran Clay as a leaky confining layer. Aquifer transmissivity and storativity, and confining layer vertical hydraulic conductivity, were assigned a reasonable range of values based on the information discussed in Section 3, as summarized in Table 4.2.1, below. Assigned values for horizontal hydraulic conductivity ranged from a maximum of 40 ft/day (the value derived from the City of Patterson pumping test) to 17 ft/day (the 10th percentile hydraulic conductivity derived from the analysis of specific capacity data presented in Table 3.4.1).

Pumping. Pumping was simulated to occur from three wells installed as shown on Figure 4.2.1. Pumping was assumed to be equally distributed among the three wells. Pumping was modeled to occur only in the confined aquifer over a thickness of 550 feet, encompassing the sediments extending vertically from the base of the Corcoran Clay to approximately 80 feet above the reported base of fresh water. The total pumping for each project development phase was based upon the net increase in potable groundwater demand at the end of each buildout phase, compared with the pre-development condition, as summarized in Table 4.2.1, below.

Table 4.2.1 Analytical Model Input Parameters

Model Input Parameter		Input Data Value				Data Source
		Shallow Aquifer	Corcoran Clay	Confined Aquifer	Forebay	
Aquifer Thickness (feet)		250	70	550	250 to 400	Section 3.4
Storativity		0.04	0.0001 to 0.0003	0.0001 to 0.0003	0.004	Section 3.4.2
Specific Yield		11.8	0.0001 to 0.0003	11.8	11.8	Section 3.4.2
Hydraulic Conductivity, Horizontal (ft/day)		17 to 40	0.0003 to 0.001	17 to 40	17 to 40	Table 3.2.1
Hydraulic Conductivity, Vertical (ft/day)		1	0.000003 to 0.0001	1	1	Fetter, 1994
Net Pumping Rate (AFY)	Phase 1 (2017 to 2026)	0	0	739	NA	Table 2.2.1
	Phase 2 (2027 to 2036)	0	0	1,036	NA	Table 2.2.1
	Phase 3 (2037 to 2046)	0	0	1,496	NA	Table 2.2.1

4.2.2 Model Inputs

The analytic element model's input parameters are summarized in the Table 4.2.1 above. The model assumes all pumping is from the confined aquifer to meet the increased demand for potable water, and that there is no net increase in groundwater demand from the shallow aquifer.

4.2.3 Model Scenarios

As with any predictive modeling study, uncertainty in the model inputs will affect the reliability of the results. Therefore, four modeling scenarios were developed in order to address a reasonable range of possible outcomes, thus bracketing the likely effects of the Project. These scenarios are described in Table 4.2.2, below. For each scenario, drawdown is evaluated at the full buildout of each construction phase (i.e., after 10, 20, and 30 years).

4.2.2 Analytical Modeling Scenarios

Parameter	Scenario 1	Scenario 2	Scenario 3	Scenario 4
Direct Recharge to Confined Aquifer from Forebay	✓	✓		
No Direct Recharge to Confined Aquifer from Forebay			✓	✓
Best Case Aquifer Parameters[1]	✓		✓	
Worst Case Aquifer Parameters[2]		✓		✓

[1] Confined aquifer storativity of 0.0003 and horizontal hydraulic conductivity of 40 ft/day; Corcoran Clay storativity and specific yield of 0.0003, horizontal hydraulic conductivity of 0.001 ft/day, and vertical hydraulic conductivity of 0.0001 ft/day.

[2] Confined aquifer storativity of 0.0001 and horizontal hydraulic conductivity of 17 ft/day; Corcoran Clay storativity of 0.0001, specific yield of 0.000003, horizontal hydraulic conductivity of 0.0003 ft/day, and vertical hydraulic conductivity of 0.000003 ft/day.

4.2.4 Assumptions and Limitations

This section presents hydrogeologic assumptions that are incorporated in the analytical element model.

- The aquifer layers have a uniform lateral and vertical hydraulic conductivities, and uniform specific yield and storativity. This is a typical simplifying assumption inherent in many models, and is appropriate as long as the objective is to model the general distribution of impacts under average conditions.
- The potentiometric surface is approximated through the use of boundary conditions and is not calibrated. This is simplifying assumption used in many models that are designed to evaluate drawdown relative to a baseline condition using a superposition approach. The inherent limitation in this approach is that the model cannot be used to predict actual groundwater level elevations. In

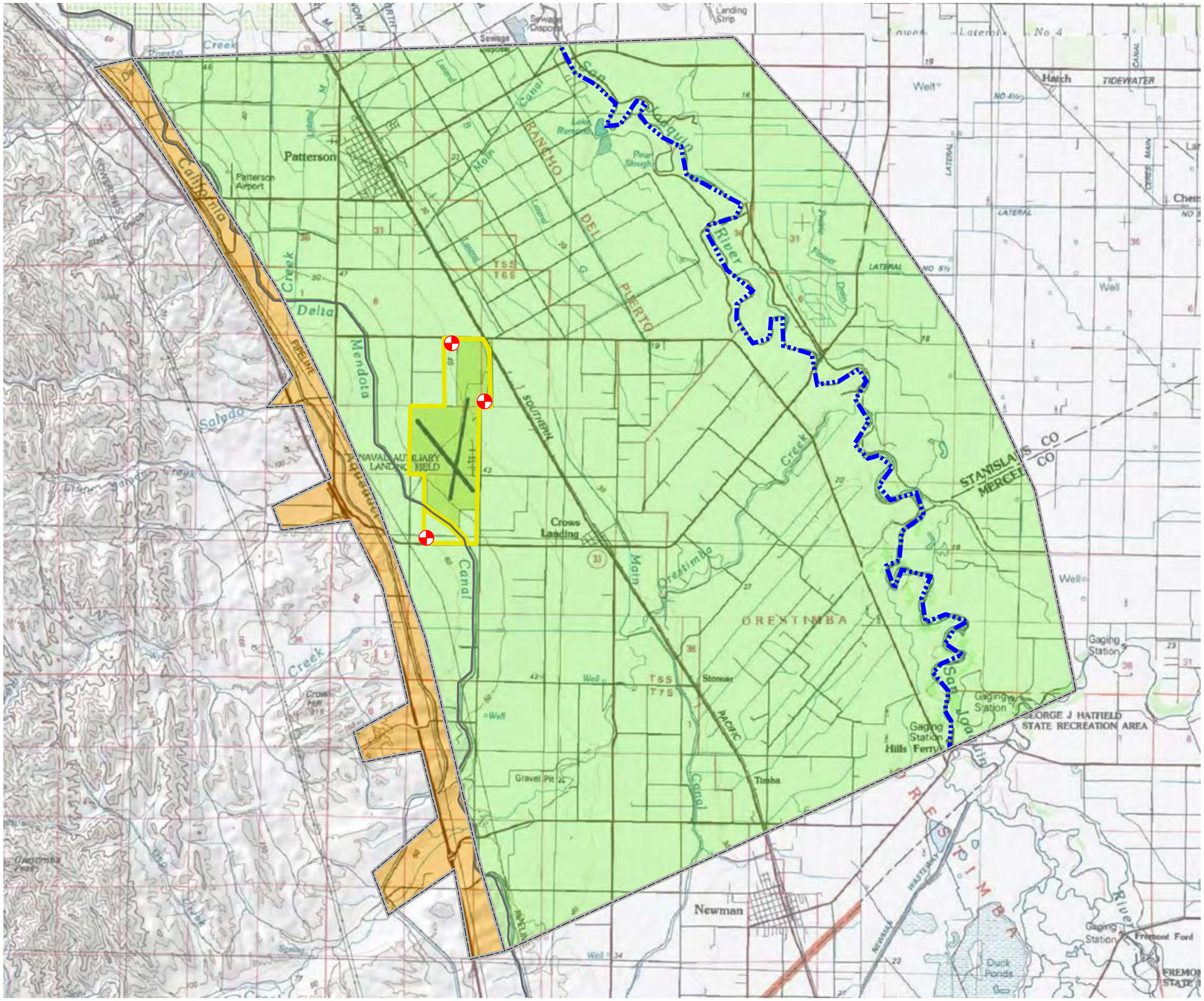
addition, the modeled drawdown may be considered an approximation. The impact of these limitations is lessened through the use of range of boundary and aquifer conditions.

- Water is released from storage in the aquifers instantaneously, the pumping well is screened in, and receives water from, the full thickness of the aquifer, and the well is 100 percent efficient.
- Areal recharge and pumping discharge (with exception of the Project) are assumed to be balanced and are therefore neglected in the simulation. This assumption is supported by the generally stable groundwater levels in the Site vicinity.
- Mountain front recharge, underflow in, underflow out, and river discharge are balanced and simulated using boundary conditions, line sinks and areal flux in the forebay subdomain.

4.3 Results

The distribution of drawdown predicted for each of the four scenarios is shown at the buildout of Project Phase 1, 2 and 3 on Figures 4.3.1, 4.3.2, and 4.3.3, respectively, and key findings are summarized in Table 4.3.1. Predicted drawdown in the confined aquifer is greatest under Scenario 4 and least under Scenario 1. Predicted drawdown is more sensitive to the modeled difference in aquifer parameters than to the different recharge conditions that were evaluated. Key findings from the predictive modeling are summarized below:

- Drawdown is predicted to stabilize quickly for each stress period, generally within a year.
- The maximum predicted drawdown in the confined aquifer ranges from:
  - 2 to 7 feet at completion of Phase 1 buildout;
  - 3 to 10 feet at completion of Phase 2 buildout; and,
  - 4 to 14 feet at completion of Phase 3 buildout
- The maximum predicted drawdown in the confined aquifer beneath the Delta-Mendota Canal ranges from:
  - 1 to 6 feet at completion of Phase 1 buildout;
  - 2 to 9 feet at completion of Phase 2 buildout; and,
  - 3 to 13 feet at completion of Phase 3 buildout
- The predicted drawdown in the confined aquifer at completion of Phase 3 buildout ranges from 2 to 7 feet near the city of Patterson and from approximately 1 to 4 feet beneath the city of Newman. This suggests that drawdown related to Project pumping would contribute slightly to the cones of depression northwest and south of the Site, but the Project-related drawdown will be in the range of 1 to 10 percent of the total drawdown observed in these areas to date (on the order of 50 to 100 feet based on fall 2015 data; see Appendix B).
- Predicted drawdown in the shallow aquifer from new pumping in the confined aquifer will be negligible.



LEGEND

- Assumed Location of Confined Aquifer Well
- Site location
- San Joaquin River Line Sink
- East Subdomain (Shallow Aquifer, Corcoran Clay and Confined Aquifer)
- West Subdomain (Forebay)

N

0 1.5
Miles

JACOBSON | JAMES
& associates, inc

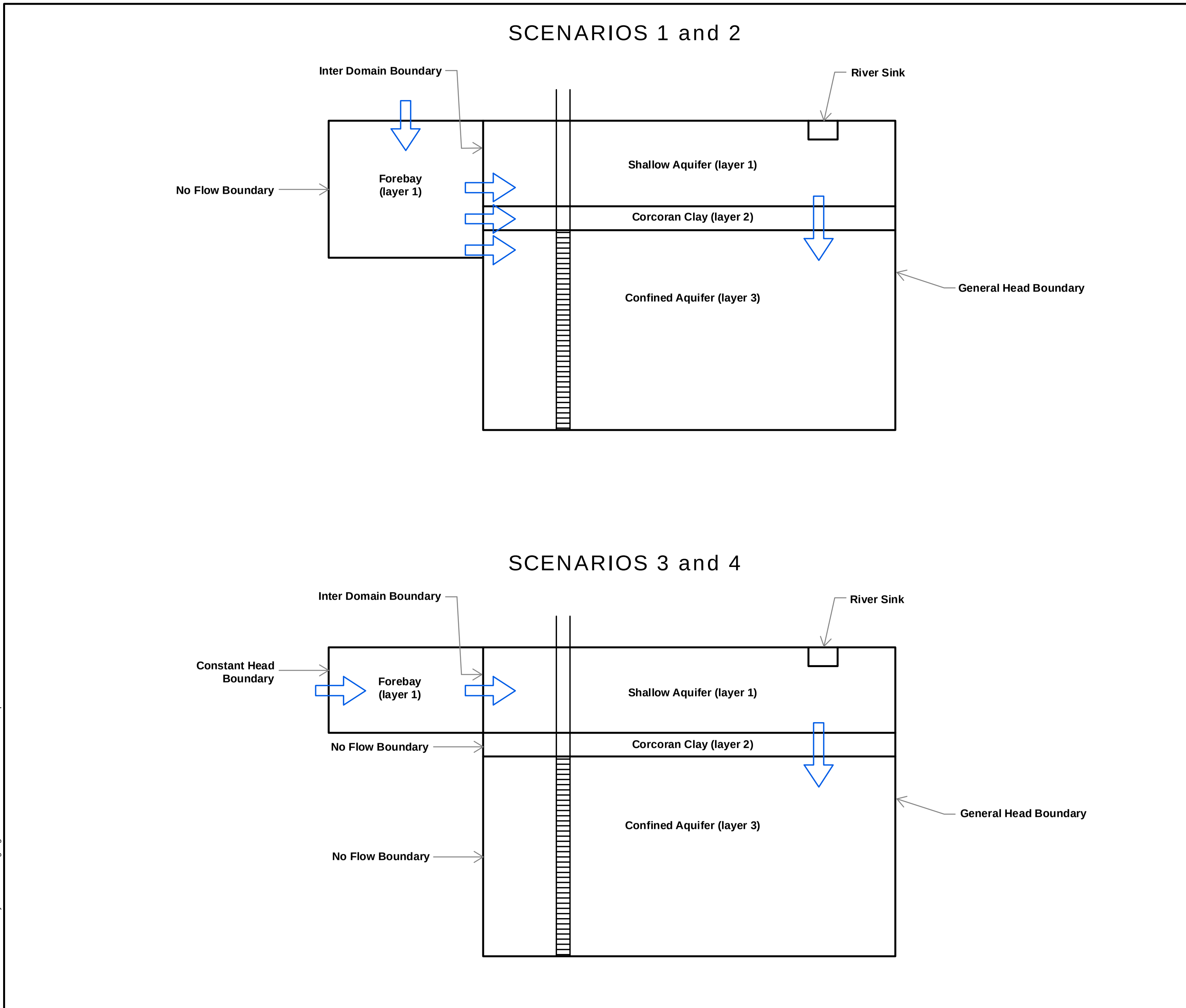
TITLE

MODEL DOMAIN BOUNDARIES AND FEATURES

CROWS LANDING INDUSTRIAL BUSINESS PARK
STANISLAUS COUNTY, CALIFORNIA

DRAWN BY	APPROVED BY	DATE	FIGURE
DPG	JB	8/2/16	4.2.1

Path: J:\GIS\StanislausCounty\CrowsLanding\Figure 4-2-1 Model Domain and Features.mxd



JACOBSON | JAMES & associates, inc

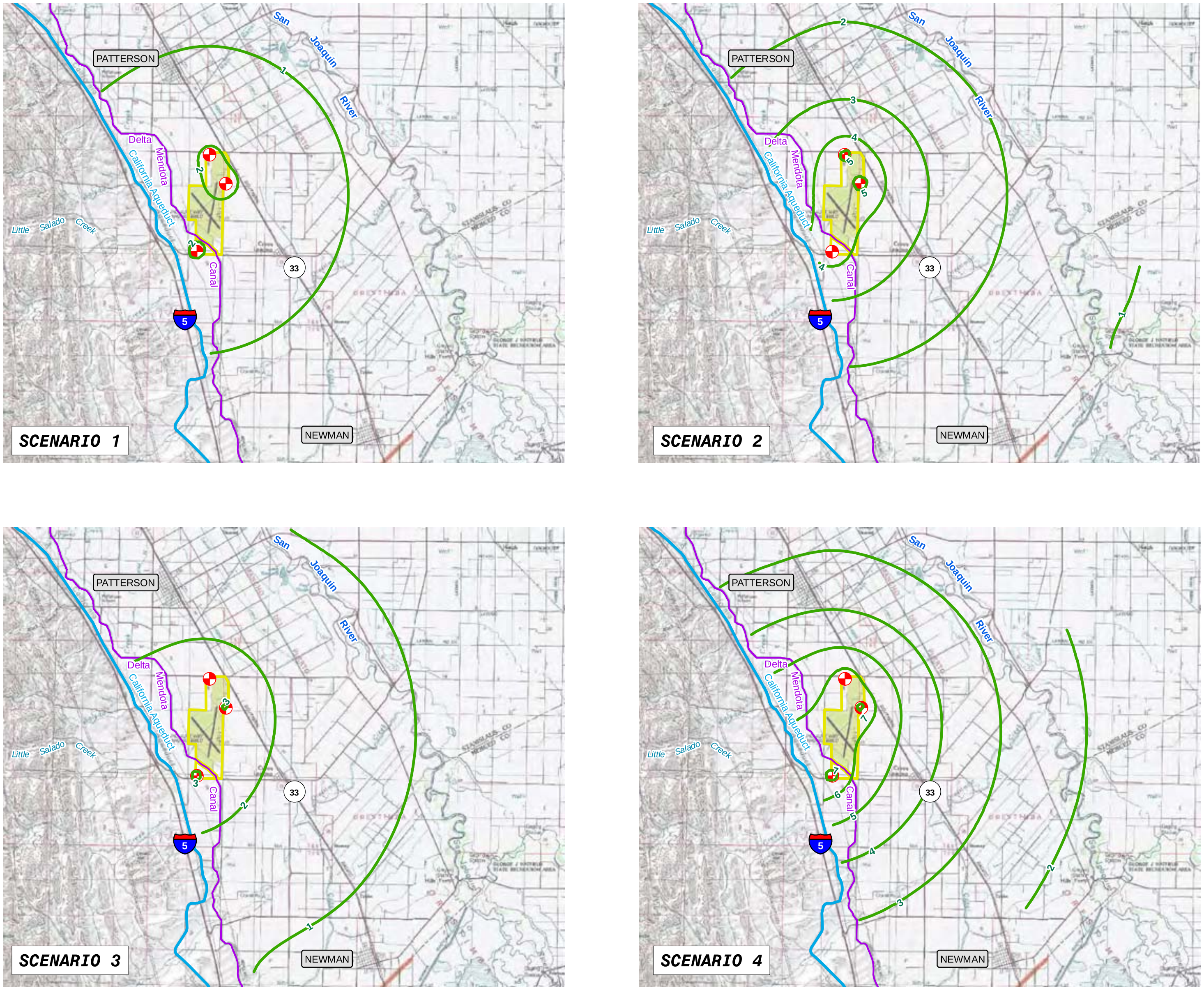
TITLE

CONCEPTUAL ILLUSTRATION OF MODEL CONSTRUCTION

CROWS LANDING INDUSTRIAL BUSINESS PARK STANISLAUS COUNTY, CALIFORNIA

DRAWN BY	APPROVED BY	DATE	FIGURE
DPG	JB	8/1/16	4.2.2

Path: J:\GIS\StanislausCounty\CrowsLanding\Figure 4-2-2 Model Construction Concept.mxd



LEGEND

Assumed Location of Confined Aquifer Well

Contour for Predicted Groundwater Drawdown (1-foot intervals)

Site Boundary

N

0 3 Miles

JACOBSON | JAMES
& associates, inc

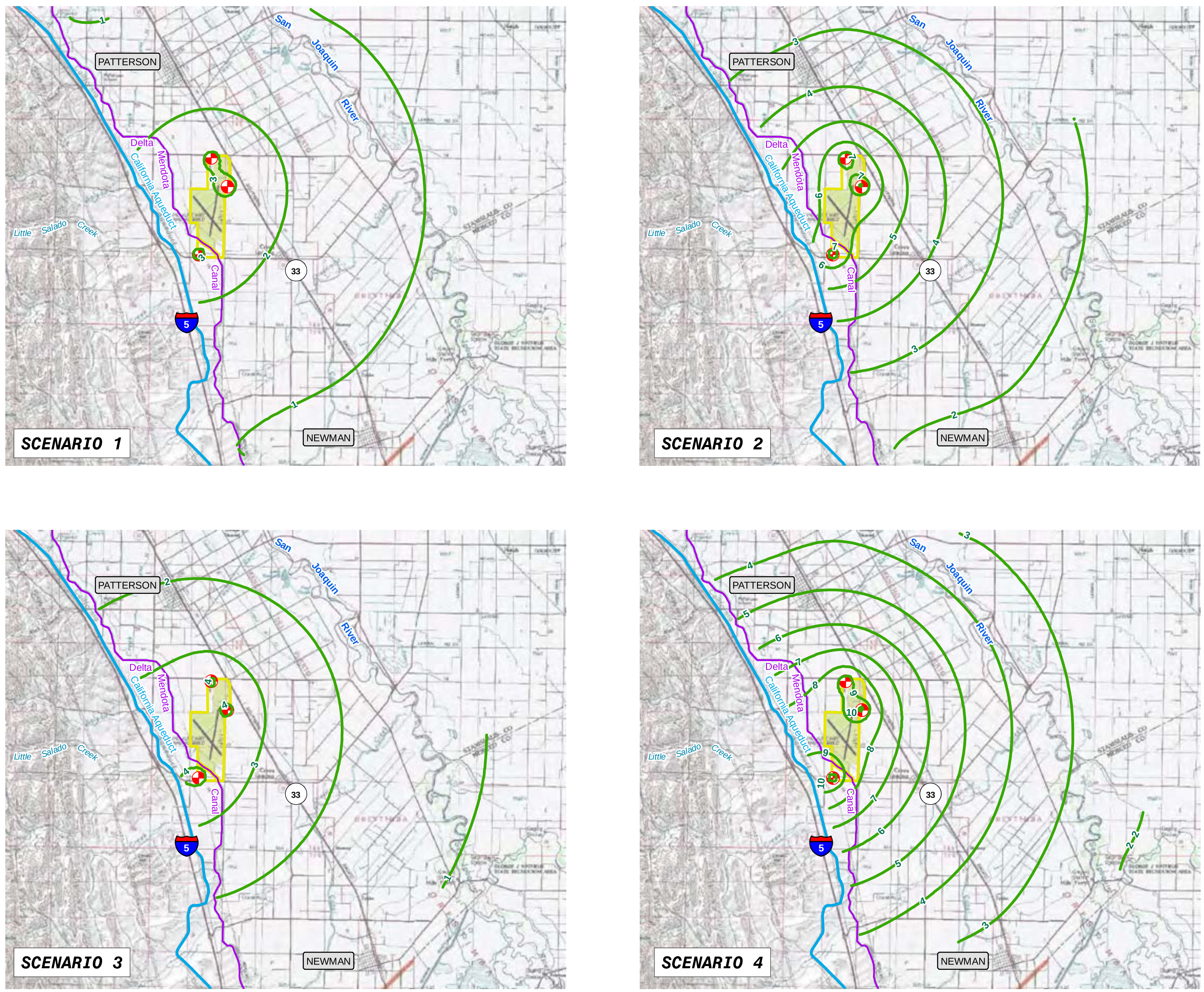
TITLE

PREDICTED DRAWDOWN IN THE CONFINED AQUIFER FOR SCENARIOS 1 THROUGH 4 END OF PHASE 1

CROWS LANDING INDUSTRIAL BUSINESS PARK STANISLAUS COUNTY, CALIFORNIA

DRAWN BY	APPROVED BY	DATE	FIGURE
DPG	JB	8/1/16	4.3.1

Path: J:\GIS\StanislausCounty\CrowsLanding\Figure 4-3-1 Phase 1 Scenarios.mxd



LEGEND

Assumed Location of Confined Aquifer Well

Contour for Predicted Groundwater Drawdown (1-foot intervals)

Site Boundary

JACOBSON | JAMES
& associates, inc

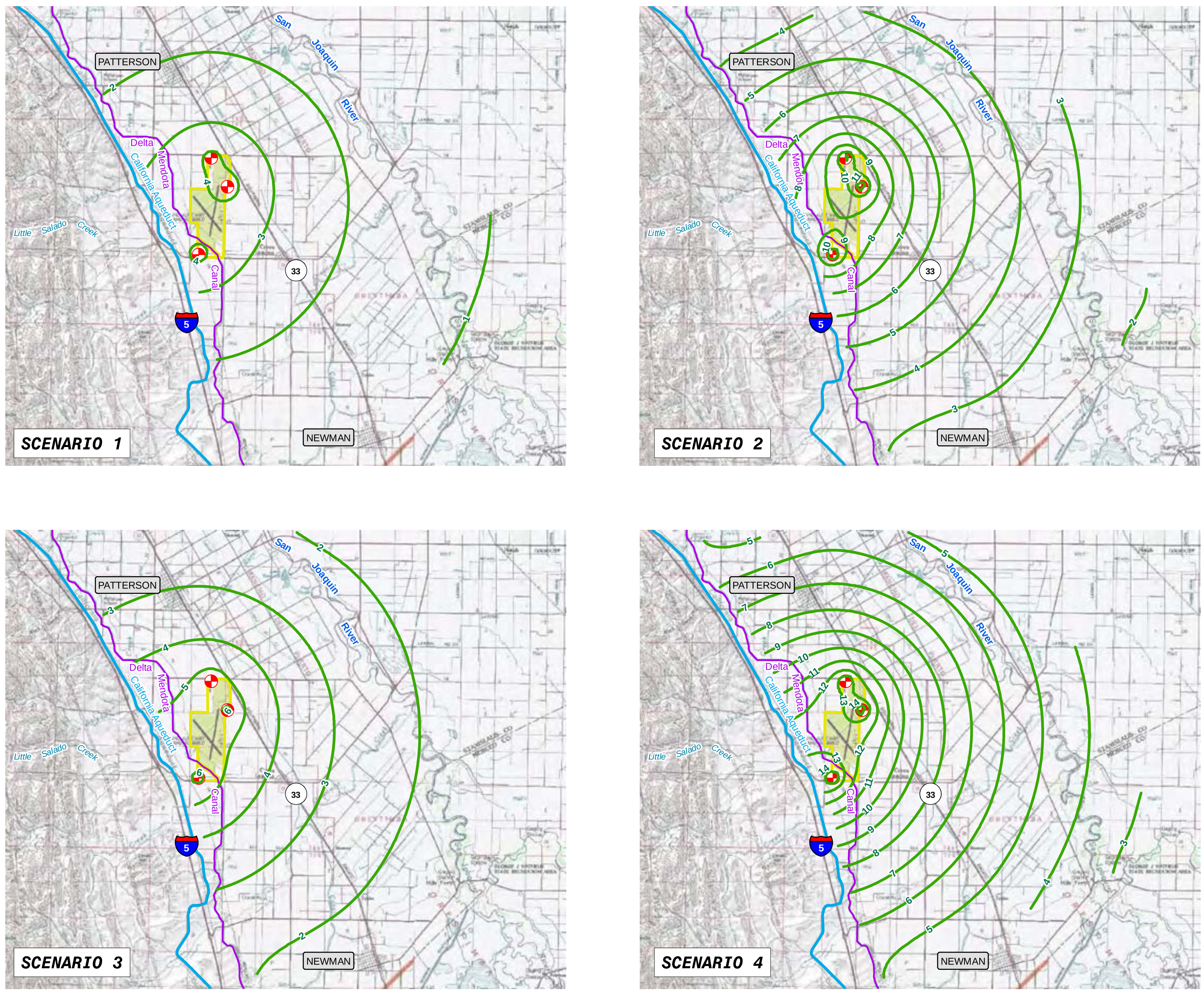
TITLE

PREDICTED DRAWDOWN IN THE CONFINED AQUIFER FOR SCENARIOS 1 THROUGH 4 END OF PHASE 2

CROWS LANDING INDUSTRIAL BUSINESS PARK STANISLAUS COUNTY, CALIFORNIA

DRAWN BY	APPROVED BY	DATE	FIGURE
DPG	JB	8/1/16	4.3.2

Path: J:\GIS\StanislausCounty\CrowsLanding\Figure 4-3-2 Phase 2 Scenarios.mxd



LEGEND

- Assumed Location of Confined Aquifer Well
- Contour for Predicted Groundwater Drawdown (1-foot intervals)
- Site Boundary

JACOBSON | JAMES
& associates, inc

TITLE

PREDICTED DRAWDOWN IN THE CONFINED AQUIFER FOR SCENARIOS 1 THROUGH 4 END OF PHASE 3

CROWS LANDING INDUSTRIAL BUSINESS PARK STANISLAUS COUNTY, CALIFORNIA

DRAWN BY	APPROVED BY	DATE	FIGURE
DPG	JB	8/1/16	4.3.3

Path: J:\GIS\StanislausCounty\CrowsLanding\Figure 4-3-3 Phase 3 Scenarios.mxd

5.0 IMPACT EVALUATION

This section presents an evaluation of the potential environmental impacts of the Project associated with groundwater resources. The impact evaluation is provided in the form of reasoned evaluations in answer to each of the applicable significance questions contained in Appendix G of the CEQA Guidelines, listed below. The questions are grouped by topic based on the "undesirable results" defined in the County Groundwater Ordinance and the California Water Code. As such, the evaluation also provides substantial evidence whether or not the proposed new wells to be installed for the Project comply with the prohibition against unsustainable extraction contained in the County Groundwater Ordinance. An additional section is added to discuss water supplies and entitlements, which are a topic under CEQA that is not included in the Groundwater Ordinance.

5.1 Groundwater-Dependent Ecosystems

Question IV(a): Would the project have a substantial adverse effect, either directly or through habitat modifications, on any species identified as a candidate, sensitive, or special status species in local or regional plans, policies, or regulations, or by the California Department of Fish and Game or U.S. Fish and Wildlife Service?

Question IV(b): Would the project have a substantial adverse effect on any riparian habitat or other sensitive natural community identified in local or regional plans, policies, regulations, or by the CDFG or USFWS?

Question IV(c): Would the project have a substantial adverse effect on a federally protected wetlands as defined by Section 404 of the Clean Water Act (including marsh, vernal pool, coastal, etc.) through direct removal, filling, hydrological interruption, or other means?

Groundwater near the site occurs at depths of at least 30 feet or more beneath the ground surface, so wetlands identified in the Site vicinity are not connected to the regional water table. Further east, wetlands and riparian vegetation near the San Joaquin River may be groundwater connected; however, pumping from the confined aquifer is predicted to produce negligible drawdown in this area. The project will not result in any net increase in groundwater demand from the shallow aquifer, and it is unlikely that localized drawdown around shallow aquifer pumping wells will extend as far as the San Joaquin River. As such, impacts to GDEs will be less than significant. A groundwater monitoring program will be implemented to assess project drawdown in the shallow and confined aquifer near the Site, and will be used to assess changes to the shallow aquifer well field operation to avoid excessive drawdown in any particular area (see Section 5.6). This program will further reduce the less than significant impacts to GDEs.

5.2 Water Quality

Question IX(a): Would the project violate any water quality standards or waste discharge requirements?

Question IX(f): Would the project otherwise substantially degrade water quality?

The Project includes operation of existing and new groundwater wells in both the shallow and confined aquifers beneath the Site. New wells completed in the confined aquifer will be completed above the base of fresh water and separated from the existing hydrocarbon plume in the shallow aquifer by the Corcoran Clay. Therefore, Project pumping from the confined aquifer will not draw from areas where water is known to have low quality, and will not interfere with shallow aquifer remediation efforts. Pumping from the shallow aquifer to meet non-potable Project water demand will occur outside of the 2,000-foot pumping restriction around the IRP Site 17 contamination plume to avoid capture of contaminated water or interference with remediation efforts. No degradation of irrigation water has been reported over time, which indicates that infiltration of applied groundwater does not substantially degrade groundwater quality, and poor quality water is not being drawn into the area. New wells installed for the Project will not be cross screened across the Corcoran Clay, and so will not create a conduit between zones of varying water quality. The existing cross screened irrigation well will be actively pumped as part of the project, and therefore will not serve as a conduit for water exchange between the shallow and confined aquifers. Based on these considerations, no significant impacts are anticipated.

5.3 Subsidence

Question VI(c): Would the project be located on a geologic unit or soil that is unstable, or that would become unstable as a result of the project, and potentially result in on or off-site landslide, lateral spreading, subsidence, liquefaction, or collapse?

DWR has designated the entire DMGS as having a high potential for future subsidence, and between 1 and 2.5 inches of subsidence have been reported since 2005 at continuous monitoring station P259 along State Route 33 near the northeast corner of the Site (DWR, 2016b). The DWR and Bureau of Reclamation have undertaken a joint subsidence monitoring program in support of the San Joaquin River Restoration Program that includes a geodetic control network of monitoring stations that spans the Site (USBOR, 2014). Surveying conducted in support of this program indicates that the average subsidence rate near the Site has been in the range of 0 to 01.5 feet per year between December 2011 and December 2015 (USBOR. 2016). Surveys conducted between December 2012 and December 2013 indicate slightly accelerated short term subsidence rates during that time period between 0.15 and 0.3 feet per year (USBOR, 2014).

As discussed in Section 3.5, subsidence in the San Joaquin Valley has occurred mainly when compressible clays are dewatered as a result of drawdown in the confined aquifer system beneath the Corcoran Clay to below historical low levels. Long term hydrographs are not available for any of the wells at the Site; however, as discussed in Section 3.4.1 and shown on Figure 3.4.1, several wells with long terms hydrograph data are located in the region south of the Site near the City of Newman (DWR, 2016d). Current groundwater levels in the well with the longest period of record (State Well No. 06S08E29J001M) are approximately 40 feet above their historical low level in October 1952. Conversely, groundwater levels in State Well No.'s 07S08D14D001M and 06S08E34M001M are at their historical low levels; however, water level data are not available for these wells prior to October 1958 and March 1959, respectively, so it is not known whether the current low groundwater level elevations at these wells represents the historical low.

Based on the above, it is possible that drawdown induced by the Project near the Delta-Mendota Canal (3 to 13 feet at the end of Phase 3 buildout) could lower groundwater levels to near or below historical low levels. Some subsidence could be induced as a result; however, given the limited amount of drawdown that is predicted and that less than 2 inches of subsidence has been reported near the Site to date, the likelihood of subsidence that substantially interferes with surface land uses and infrastructure is judged to be small. Nevertheless, Mitigation Measure (MM) Water-01 is proposed to monitor for active subsidence and make adjustments to the groundwater extraction program, if needed (see Section 5.6). With implementation of MM Water-01, impacts will be less than significant.

5.4 Chronic Drawdown and Diminution of Supply

Question IX(b): Would the project substantially deplete groundwater supplies or interfere substantially with groundwater recharge such that there would be a net deficit in aquifer volume or a lowering of the local groundwater table (e.g., the production rate of pre-existing nearby wells would drop to a level which would not support existing land uses or planned uses for which permits have been granted)?

Operation of the potable water production wells for the Projects will result in groundwater level drawdown in the confined aquifer in the region around the Site, and will result in interference drawdown to existing supply wells completed in this aquifer. Regional drawdown, if it represents a substantial fraction of the overall available drawdown, or groundwater in storage, in an aquifer system, can result in less water supplies being available for future supply, insufficient availability of groundwater during dry periods, or a general increase in groundwater supply development costs. Interference drawdown is a more localized effect that can decrease well yield and, in extreme cases, cause wells to go dry. The wells potentially most vulnerable to interference drawdown are domestic wells, which are generally shallower than municipal, industrial and irrigation wells that are completed to greater depths and have greater pumping capacities. In the Site vicinity, domestic wells tend to be completed in the shallow aquifer; whereas, higher capacity production wells are completed in either the shallow or the confined aquifer (or both).

The maximum predicted Project-induced drawdown in the confined aquifer is approximately 13 feet. This is less than 10 percent of the available drawdown above the top of the confined aquifer, and is unlikely to result in a significant depletion in regional supplies. For perspective, urban and agricultural groundwater extraction was estimated to be 508,000 AFY for the DMGS (DWR, 2006). An operational yield study by the

City of Patterson estimated that the city could pump up to 12,000 AFY without significantly impacting the use of groundwater resources in the area surrounding Patterson's sphere of influence (RMC, 2016). The City of Newman pumped approximately 4,200 acre-feet of water in 2012 (KDSA, 2013). A drawdown of less than 20 feet would not be expected to result in a significant diminution in the yield in a production well, as it typically represents less than 10 percent of the available drawdown. Drawdown in the shallow aquifer from pumping in the confined aquifer is expected to be negligible. The project will not result in any net increase in groundwater demand from the shallow aquifer; however, if shallow wells located near the Site boundary are pumped excessively, nearby off-site domestic wells could experience drawdown in excess of 5 feet, which could potentially result in a significant diminution in yield in a very shallow well. MM Water-02 is proposed to place new shallow wells at least 250 feet from the nearest Site boundary. In addition, MM Water-03 is proposed to implement a groundwater level monitoring program, and adjust well field operation if drawdown in excess of 5 feet is observed near an existing domestic well. (See Section 5.6 for a description of these mitigation measures.)

Development of the Project will include retention of stormwater such that off Site stormwater flows do not increase above pre-development flows. The majority of this retention will occur as a result of water infiltration in the retention basins to be constructed on the northeast side of the CLIBP. In addition, the Project will require implementation of LID performance standards for stormwater capture and recharge at each developed parcel in order to maintain the existing groundwater balance in the shallow aquifer.

Based on the above information, with implementation of MM Water-02 and MM Water-03, Project impacts to groundwater supplies, aquifer volume, and lowering of the groundwater table will be less than significant.

Question XVIII(b): Does the project have impacts that are individually limited, but cumulatively considerable? ("Cumulatively considerable" means that the incremental effects of a project are considerable when viewed in connection with the effects of past projects, the effects of other current projects, and the effects of probable future projects.)

Predictive modeling indicates that drawdown associated with Project pumping from the confined aquifer will contribute incrementally to the cones of depression observed to the northwest and south of the Site. The Project-related drawdown at the off-Site cones of depression is predicted to range from approximately 1 to 6 feet at completion of Phase 3 buildout. This represents only about 1 to 10 percent of drawdown in the off Site cones of depression, which was on the order of 50 to 100 feet in fall 2015, and appears to be associated with increased extraction during recent drought conditions and curtailment of surface water deliveries. Long-term well hydrographs indicate that water levels have historically rebounded relatively quickly when stresses are relieved (i.e., when drought conditions end or demand is met by surface water deliveries). Subsidence and other undesirable results have not been reported in the vicinity of these cones of depression.

Municipal groundwater demand by the City of Patterson is projected to increase from 6,376 AFY in 2020 to 11,801 AFY in 2040 (RMC, 2016). Proportionally similar increases in urban demand may be expected by the City of Newman and the community of Crows Landing, assuming they experience similar urban growth.

These increases in urban demand will be offset by decreased agricultural demand as land use is changed from agricultural to urban to accommodate the population growth on which the water demand forecasts are built. In addition, these communities will be required to comply with a GSP adopted under SGMA to assure the sustainable management of local groundwater supplies by 2040. The communities of Patterson and Newman are currently considering becoming Groundwater Sustainability Agencies that will implement and enforce the GSP within their jurisdiction.

Based on these considerations, the groundwater resources impacts associated with the Project will be less than cumulatively considerable.

5.5 Water Supply and Entitlements

Question XVII(d): Would the project have sufficient water supplies available to serve the project from existing entitlements and resources, or are new or expanded entitlements needed?

Based on the above analyses, adequate groundwater supplies are available for Project use in the shallow and confined aquifers beneath the Site without causing or contributing to undesirable results as defined in the County Groundwater Ordinance, SGMA, and the California Water Code. As such, the proposed groundwater extraction would comply with these regulations. In addition, the Site is not located in an adjudicated basin, or in a special act district that regulates the extraction of groundwater. The Project would be able to supply groundwater for beneficial use on the properties to be developed in the business park under an appropriative groundwater right. No new entitlements would be required, and the Project would therefore have no impact.

5.6 Proposed Mitigation Measures

This section identifies mitigation measures to reduce potentially significant impacts associated with the Project to less than significant levels.

5.6.1 MM Water-01 – Subsidence Monitoring

The objective of MM Water-01 is to prevent subsidence associated with the Project. The Project shall include installation and semi-annual monitoring of three subsidence monuments at the Site. The exact construction, placement, and monitoring methodology shall be defined in a subsidence monitoring plan to be prepared for County approval before Project implementation. It is advised that one monument be placed near the Delta-Mendota Canal and/or the California Aqueduct, for which subsidence may be of particular concern. The monitoring entity shall report the subsidence monitoring activities and findings semi-annually to Stanislaus County for each year in July and January. If subsidence in excess of 2 inches is measured at a monument, an investigation shall be undertaken to determine the source of the subsidence and whether changes need to be made to the water supply pumping program to arrest further subsidence that could be damaging to infrastructure.

5.6.2 MM Water-02 – Well Setbacks

The objective of MM Water-02 is to prevent interference drawdown to off-Site wells. Any new shallow groundwater extraction well shall be placed at least 250 feet inside of the nearest Site boundary to minimize potential drawdown effects on shallow aquifer wells located on nearby properties. A well permit application shall be prepared by the applicant for County approval to identify the new well(s) purpose, location(s), and construction details before the wells are constructed.

5.6.3 MM Water-03 – Groundwater Level Monitoring

The objective of MM Water-03 is to assess and verify the amount of drawdown induced by Project pumping, and to prevent potential interference drawdown to shallow off-Site wells. A groundwater monitoring plan that outlines the monitoring wells network and procedures for the groundwater level monitoring program shall be prepared by the applicant for County approval before Project implementation. Groundwater levels shall be measured monthly to the nearest 0.1 foot bgs in the shallow and confined aquifers at the locations identified in the groundwater monitoring plan, and the length of time in days since the well was last operated shall also be noted. Groundwater level monitoring shall commence prior to Project implementation to establish Site baseline conditions. The extent and frequency of the monitoring program shall be evaluated every five years. The groundwater monitoring plan shall identify adjustments to be made to well field operation if Project-induced drawdown in excess of 5 feet is observed in the shallow aquifer near an existing domestic well, or if drawdown in the confined aquifer exceeds predicted levels. The monitoring entity shall report the groundwater monitoring activities and findings to Stanislaus County for each year by January 31 of the following year.

5.6.4 MM Water-04 – Recharge Enhancement Plan

The objective of MM Water-04 is to prepare a plan that describes how the Project will enhance groundwater recharge, such that any increase in Project groundwater demand on the shallow aquifer will be fully offset. The plan shall be prepared by the applicant for County approval before Project implementation. After County approval, the plan shall be implemented, including submittal of annual reports to the County by January 31 of the following year that document the amount of groundwater extracted from the shallow aquifer and the amount of recharge achieved. The plan must account for and offset any increase in the net groundwater demand, including increases resulting from development of the Project non-potable water supply and cessation of agricultural pumping and irrigation. The enhanced recharge is expected to be derived from recharge of water in the Project stormwater retention/detention basin, implementation of LID design standards for developed of parcels in the CLIBP that increase stormwater retention and recharge, and decreased non-potable water demand through the use of xeriscape landscape designs. The plan shall include design details and describe maintenance activities, and shall include supporting calculations or modeling to demonstrate that its implementation will result in sufficient recharge.

6.0 REFERENCES

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APPENDIX A

EVALUATION OF POTENTIAL STORMWATER CAPTURE EFFICIENCY FROM LOW IMPACT DEVELOPMENT STANDARDS

Appendix A: Evaluation of Potential Stormwater Capture Efficiency from Low Impact Development Standards

A screening level desktop study was performed to evaluate potential stormwater capture efficiency from Low Impact Development (LID) standards using the U.S. Environmental Protection Agency's (EPA) National Stormwater Calculator (SWC) software[1]. The study was performed as a superposition model to evaluate potential increases in capture and infiltration of surface runoff with LID elements compared with a baseline condition (buildout of the Project with no LID elements, but with required stormwater retention using a retention pond in the northeast portion of the Site). Attachment A-1 shows the SWC summary reports for both the baseline and LID implementation conditions. The basis for key assumptions and inputs are summarized below:

- The calculated storm water capture applies to additional capture that may be achieved at individual development sites through the implementation of LID elements, such as retention ponds, permeable pavements, street planters, vegetated swales, and disconnection. It is assumed that the stormwater retention basin to be constructed in the northeast portion of the Site will have sufficient infiltration capacity to maintain pre-development recharge rates.
- The site area was defined as 1 acre so that runoff calculations could be scaled appropriately based on the size of development by Project phase.
- The soil was assigned "moderately high" runoff potential (clay loam type) based on soil survey data accessed by the SWC.
- The soil was assigned a drainage rate of 0.6 feet per day based on the mean saturated hydraulic conductivity at the Site[2].
- The topography was assigned a flat (2%) slope.
- Precipitation was assigned as 11.53 inches per year based on average rainfall data at Newman from 1970 to 2006 (as accessed by the SWC).
- Evaporation was assigned as 0.22 inches per day based on data at Newman from 1970 to 2006 (as accessed by the SWC).
- The SWC default climate change scenario was applied for the near term scenario (2020 through 2049).
- Land cover (at buildout) was estimated to be 75% impervious surface based on visual review of typical recent commercial projects in the County, with the remaining 25% assigned as "lawn' to simulate landscaping
- Conceptual LID elements[3] were assigned as follows:

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[1] EPA, 2016. National Stormwater Calculator Desktop Application. Version 1.1.0.2.

[2] University of California Davis and U.S. Department of Agriculture Natural Resources Conservation Service, 2016. California Soil Properties Soil Properties App. http://casoilresource.lawr.ucdavis.edu/ca-soil-properties/. Accessed July 31.

- The baseline condition did not include any LID elements.
- The LID implementation condition assumed the Project impervious surfaces consisted of: 20% permeable pavement; 10% infiltration basins; 10% disconnection (directing runoff from impervious areas such as roofs or parking lots onto pervious surfaces rather than into storm drains); and 2% street planters.

Based on the inputs described above, the SWC estimated that 2.89 inches (0.24 foot) per year of runoff per acre would be captured for local infiltration with LID implementation compared with the baseline condition with no local LID elements (Project detention basin only). The volume of additional runoff that could be captured with LID implementation at buildout of Phases 1, 2, and 3 is estimated to be 146, 178, and 228 AFY, respectively, as summarized in the table below.

Estimated Additional Annual Surface Runoff Capture Compared with the Baseline (No LID) Condition

Timeframe	Additional Surface Runoff Capture (AFY)
Additional Capture by Buildout Phase	
Phase 1 (810 acres developed)	146
Phase 2 (177 acres developed)	32
Phase 3 (274 acres developed)	49
Cumulative Additional Capture at Phased Buildout	
Phase 1 (810 acres developed)	146
Phase 2 (987 acres developed)	178
Phase 3 (1,261 acres developed)	228

[3] Specific LID elements would be determined during Project design.

National Stormwater Calculator Report

Site Description

Crows Landing Industrial Business Park

Parameter	Current Scenario	Baseline Scenario
Site Area (acres)	1	1
Hydrologic Soil Group	C	C
Hydraulic Conductivity (in/hr)	0.6	0.6
Surface Slope (%)	2	2
Precip. Data Source	NEWMAN	NEWMAN
Evap. Data Source	NEWMAN	NEWMAN
Climate Change Scenario	None	None
% Forest	0	0
% Meadow	0	0
% Lawn	25	25
% Desert	0	0
% Impervious	75	75
Years Analyzed	20	20
Ignore Consecutive Wet Days	False	False
Wet Day Threshold (inches)	0.10	0.10

LID Control	Current Scenario	Baseline Scenario
Disconnection	10 / 100	0
Rain Harvesting	0	0
Rain Gardens	0	0
Green Roofs	0	0
Street Planters	2 / 6	0
Infiltration Basins	10 / 5	0
Porous Pavement	20 / 100	0

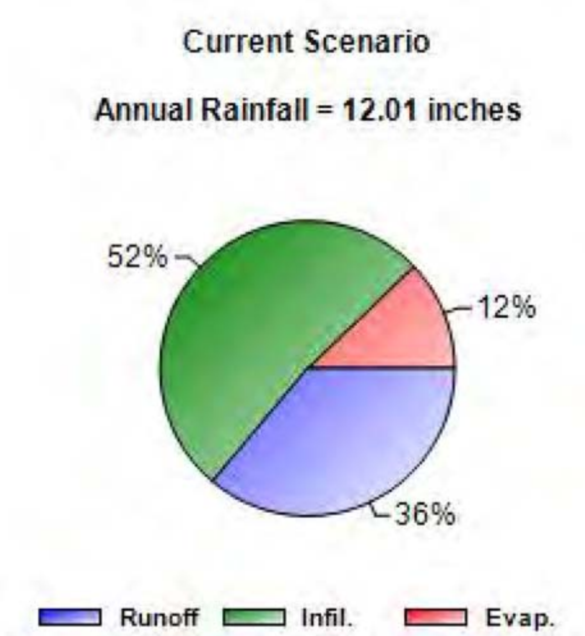
% of impervious area treated / % of treated area used for LID

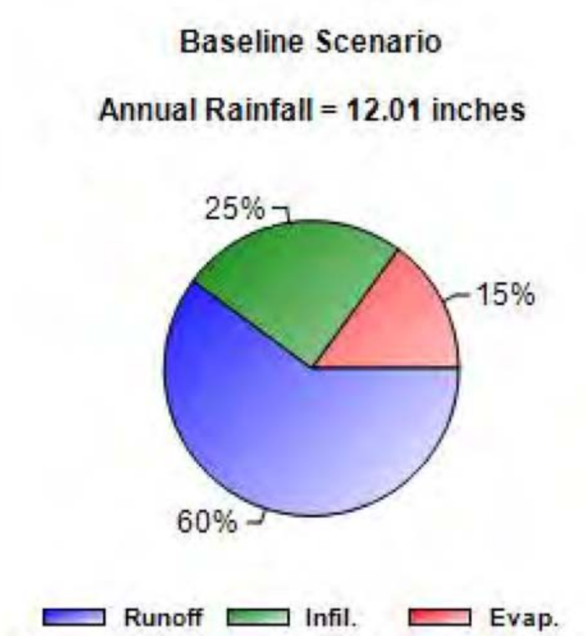
National Stormwater Calculator Report

Summary Results

Crows Landing Industrial Business Park

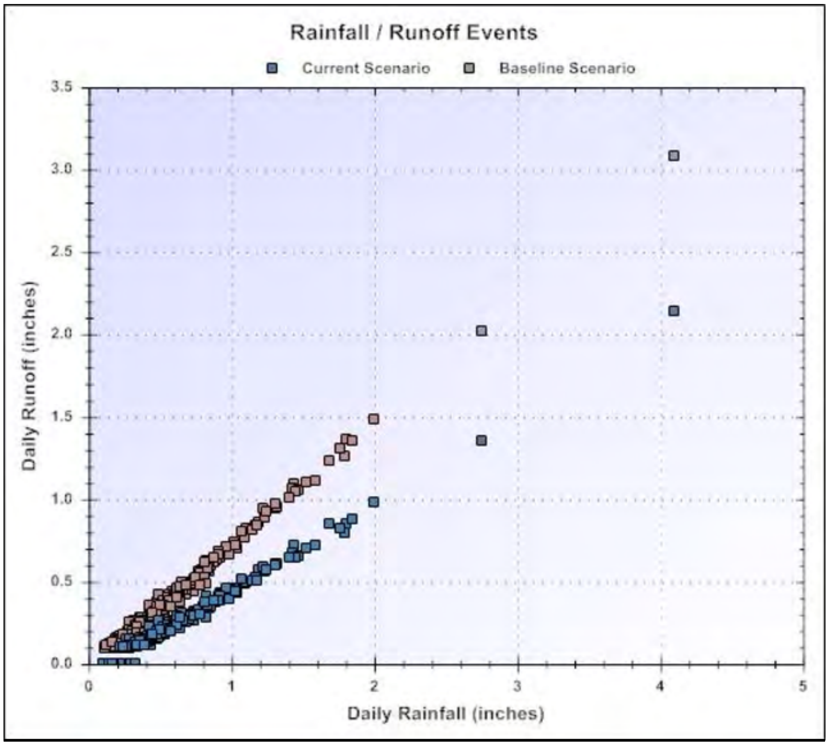
Statistic	Current Scenario	Baseline Scenario
Average Annual Rainfall (inches)	12.01	12.01
Average Annual Runoff (inches)	4.35	7.24
Days per Year With Rainfall	29.68	29.63
Days per Year with Runoff	13.79	19.64
Percent of Wet Days Retained	53.54	33.73
Smallest Rainfall w/ Runoff (inches)	0.22	0.10
Largest Rainfall w/o Runoff (inches)	0.33	0.23
Max. Rainfall Retained (inches)	1.96	1.02

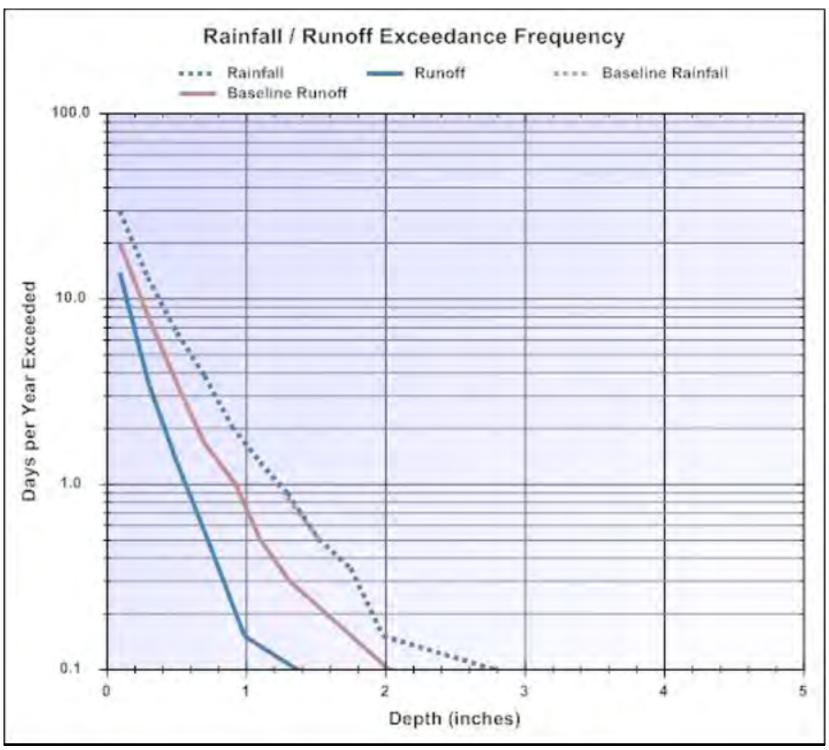




National Stormwater Calculator Report

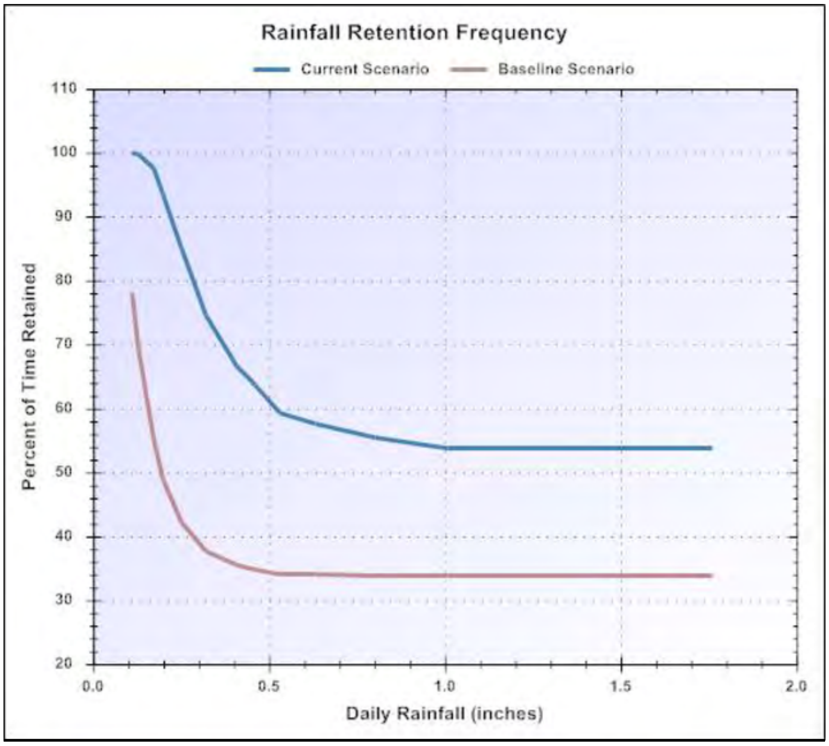
Crows Landing Industrial Business Park

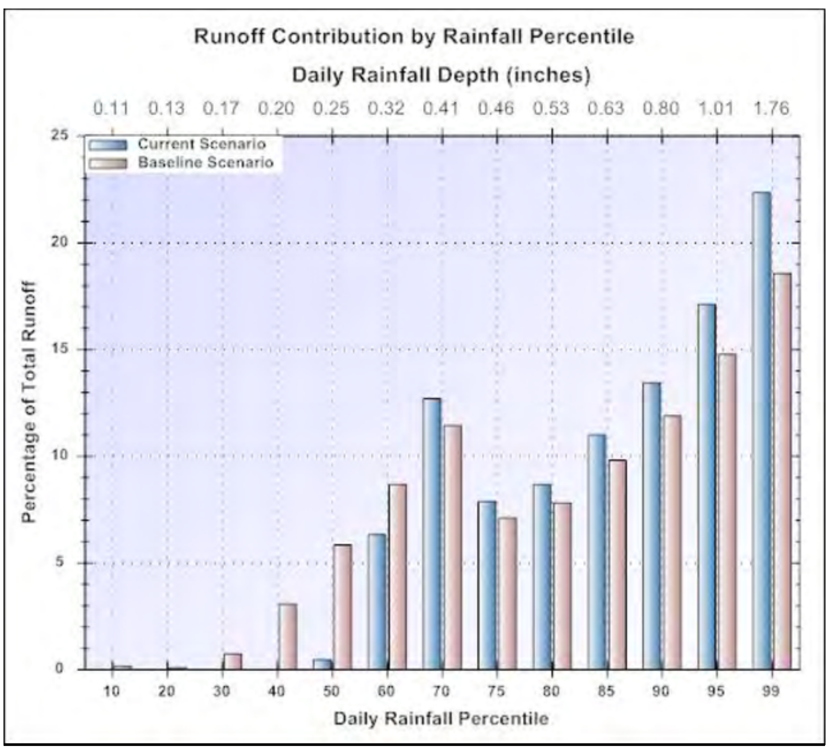




National Stormwater Calculator Report

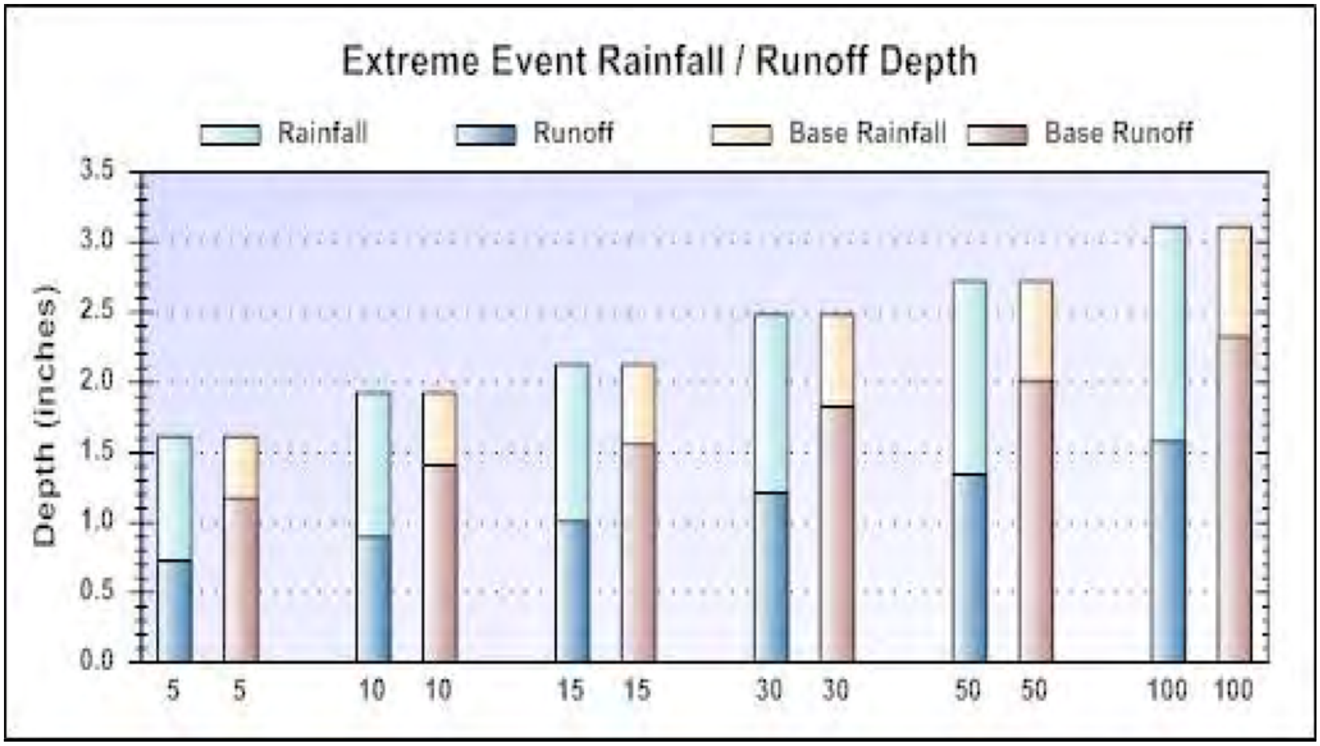
Crows Landing Industrial Business Park

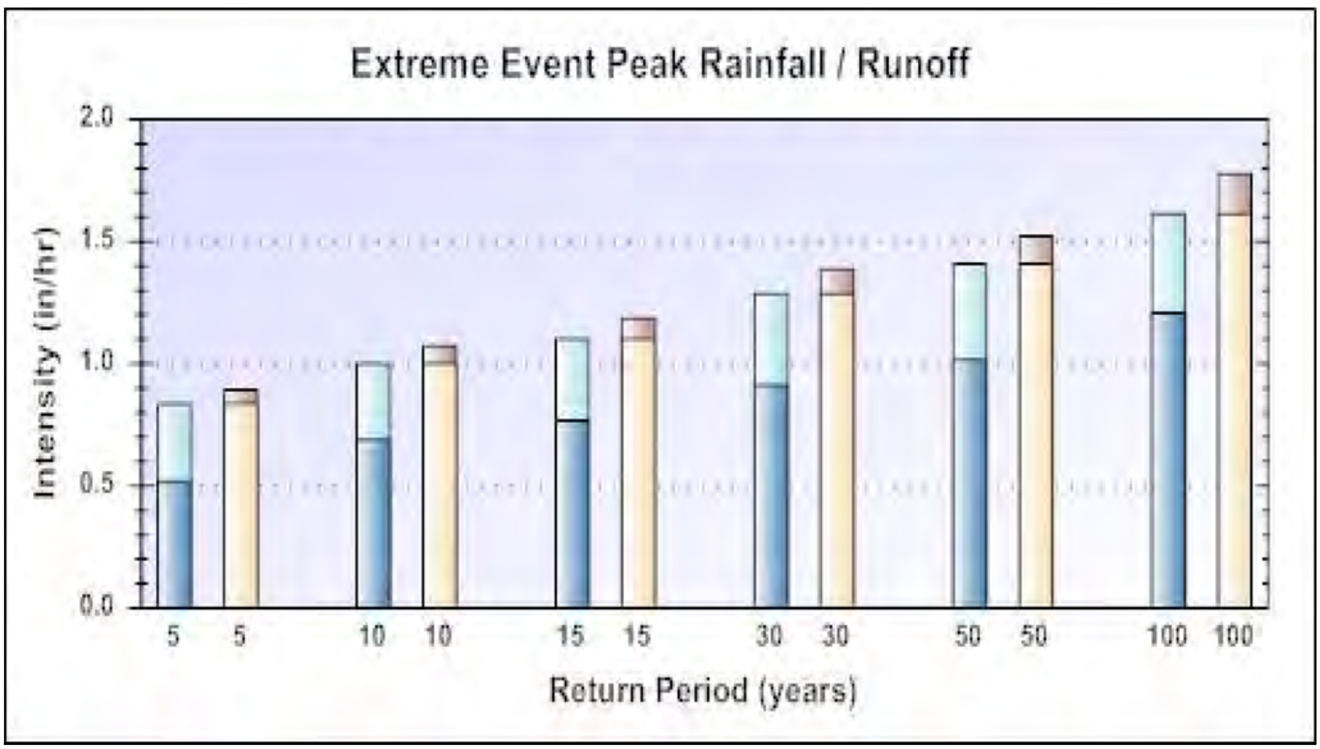




National Stormwater Calculator Report

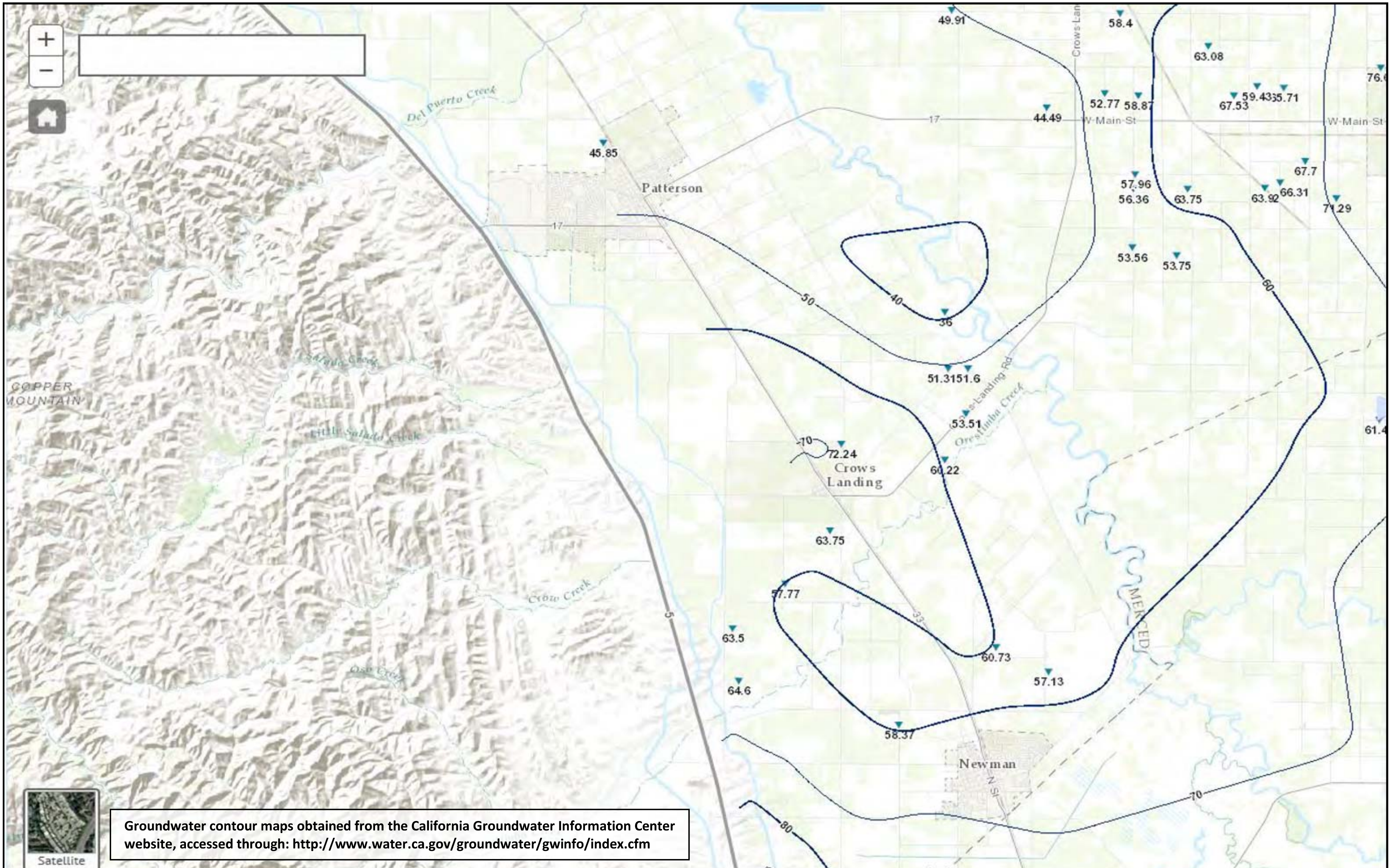
Crows Landing Industrial Business Park





APPENDIX B

GROUNDWATER ELEVATION CONTOUR MAPS FROM THE DWR GROUNDWATER INFORMATION CENTER INTERACTIVE MAPPING APPLICATION



Groundwater contour maps obtained from the California Groundwater Information Center website, accessed through: http://www.water.ca.gov/groundwater/gwinfo/index.cfm

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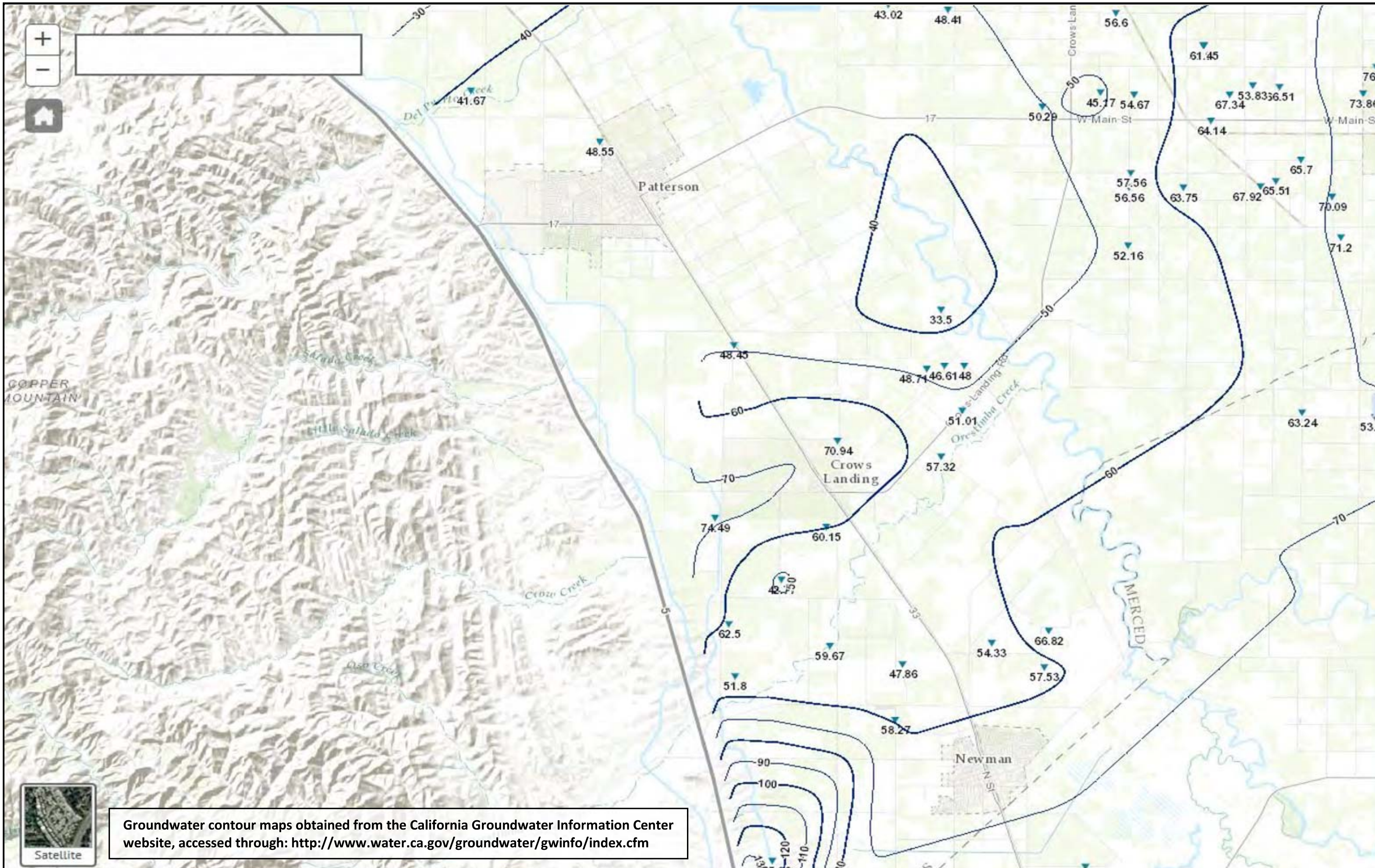
& associates, inc

Crows Landing Industrial Business Park
Stanislaus County, California

PROJECT NO.	DATE	DRAWN BY	APPR. BY
STANCO.001	7/28/16	TC	JB

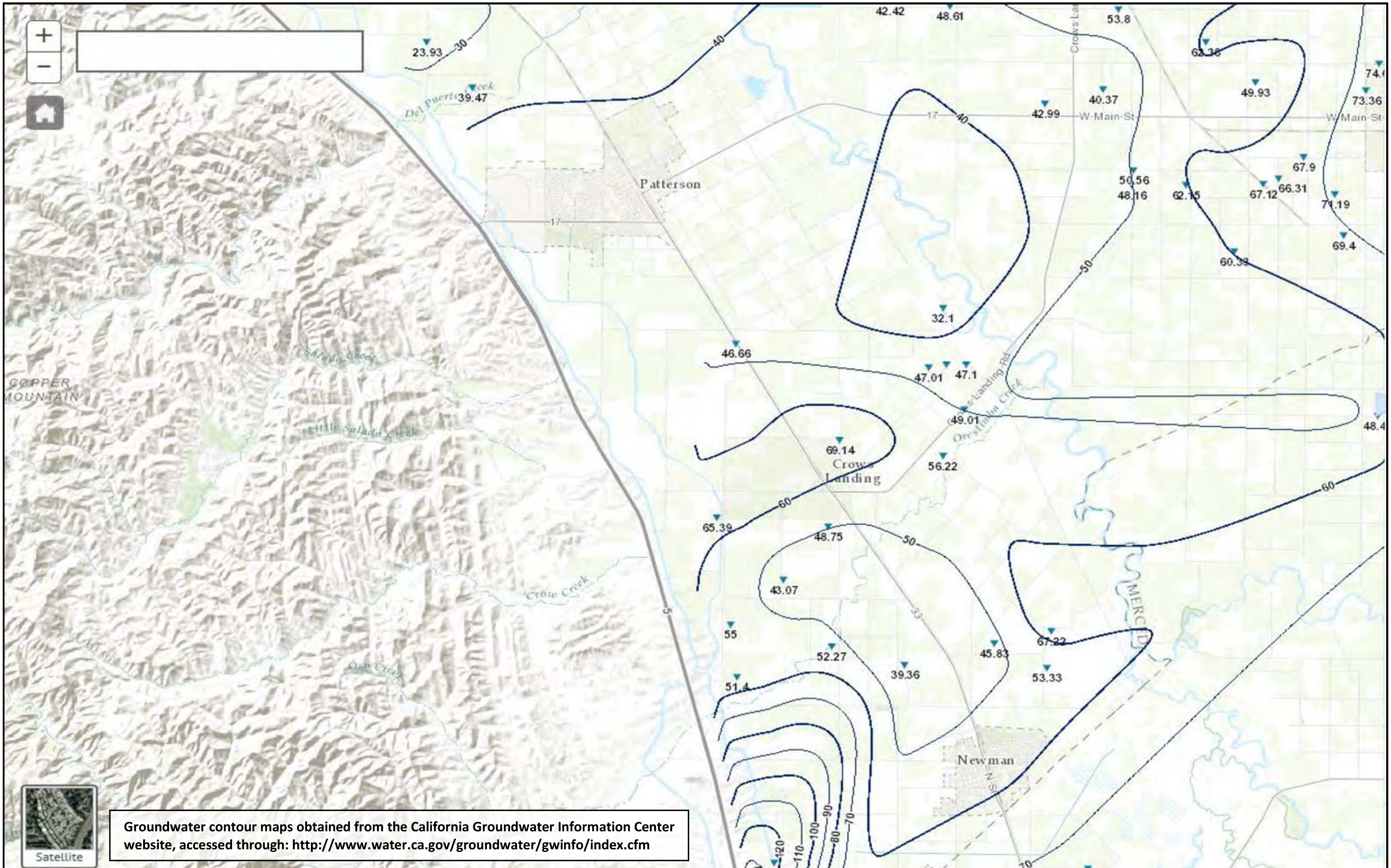
FIGURE B-1

Groundwater Elevation Map for the Confined Aquifer, Spring 2011



JACOBSON | JAMES & associates, inc	Crows Landing Industrial Business Park Stanislaus County, California		FIGURE B-2 **Groundwater Elevation Map for the Confined Aquifer, Spring 2012**

PROJECT NO.	DATE	DRAWN BY	APPR. BY
STANCO.001	7/28/16	TC	JB



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Stanislaus County, California

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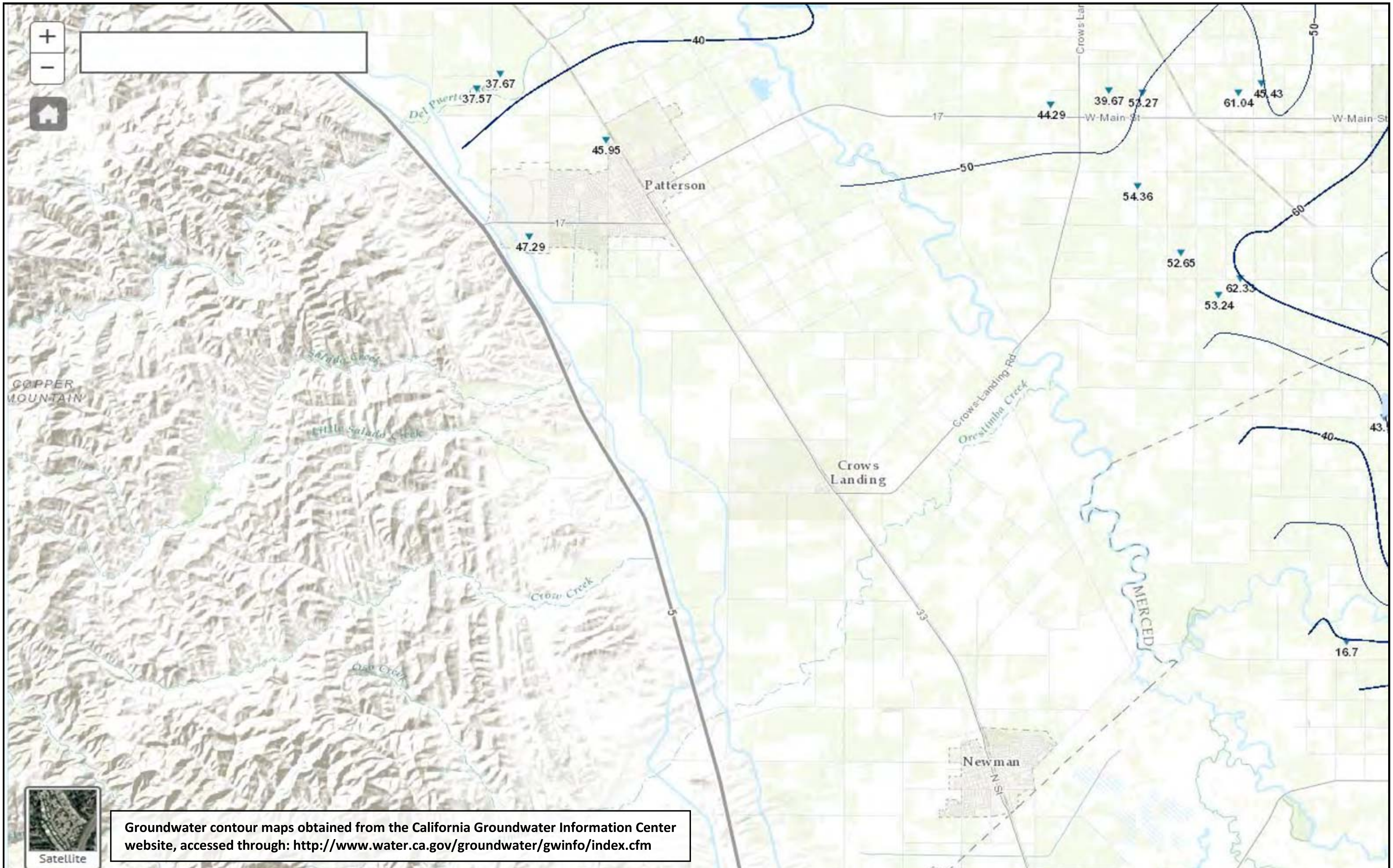
STANCO.001	7/28/16	TC	JB

FIGURE B-3

Groundwater Elevation Map for the Confined Aquifer, Spring 2013



Groundwater contour maps obtained from the California Groundwater Information Center website, accessed through: http://www.water.ca.gov/groundwater/gwinfo/index.cfm

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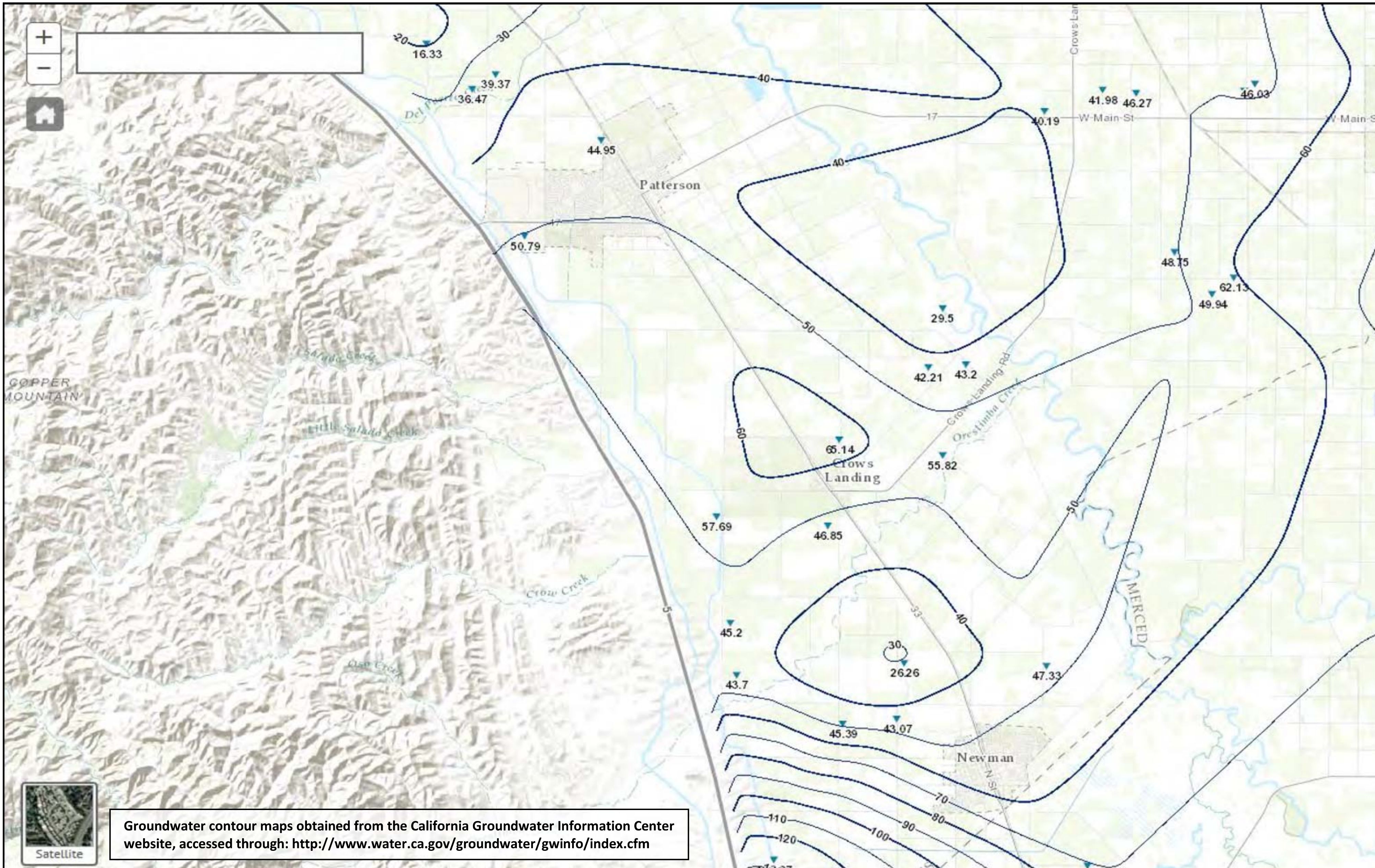
FIGURE B-4

Groundwater Elevation Map for the Confined Aquifer, Fall 2013



Groundwater contour maps obtained from the California Groundwater Information Center website, accessed through: http://www.water.ca.gov/groundwater/gwinfo/index.cfm

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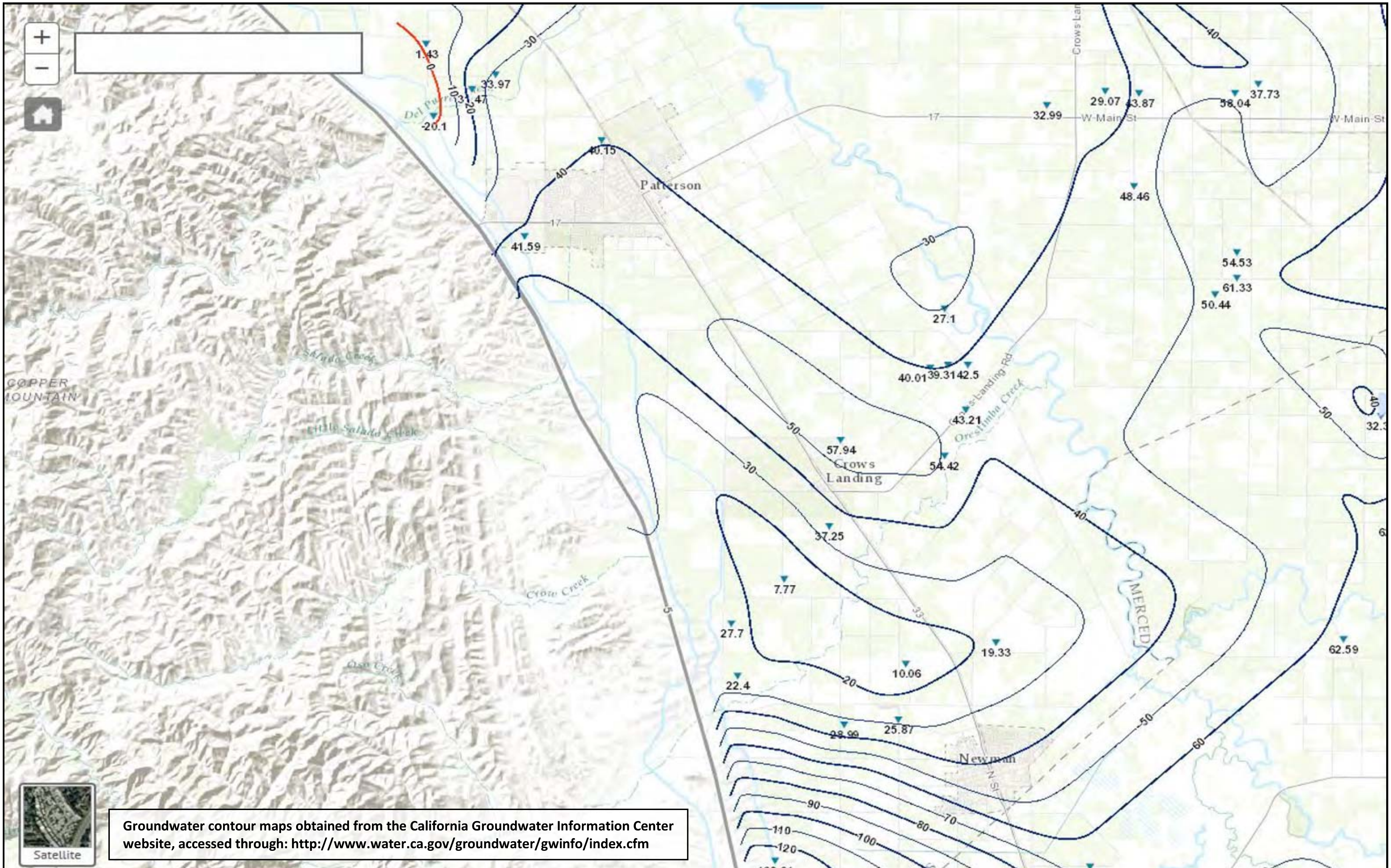
STANCO.001	7/28/16	TC	JB

FIGURE B-5

Groundwater Elevation Map for the Confined Aquifer, Spring 2014



Groundwater contour maps obtained from the California Groundwater Information Center website, accessed through: http://www.water.ca.gov/groundwater/gwinfo/index.cfm

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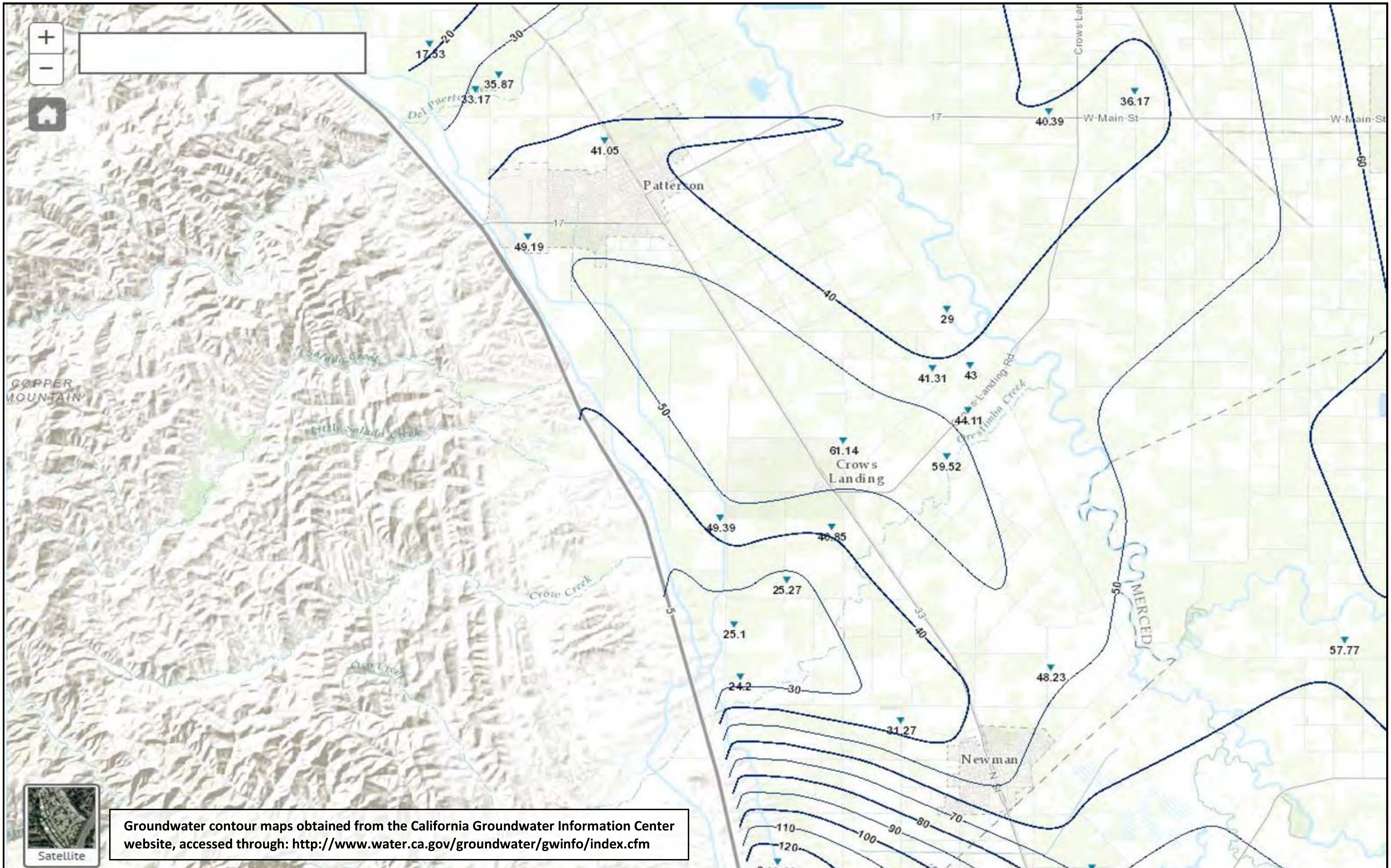
PROJECT NO.	DATE	DRAWN BY	APPR. BY
STANCO.001	7/28/16	TC	JB

FIGURE B-6

Groundwater Elevation Map for the Confined Aquifer, Fall 2014



Groundwater contour maps obtained from the California Groundwater Information Center website, accessed through: http://www.water.ca.gov/groundwater/gwinfo/index.cfm

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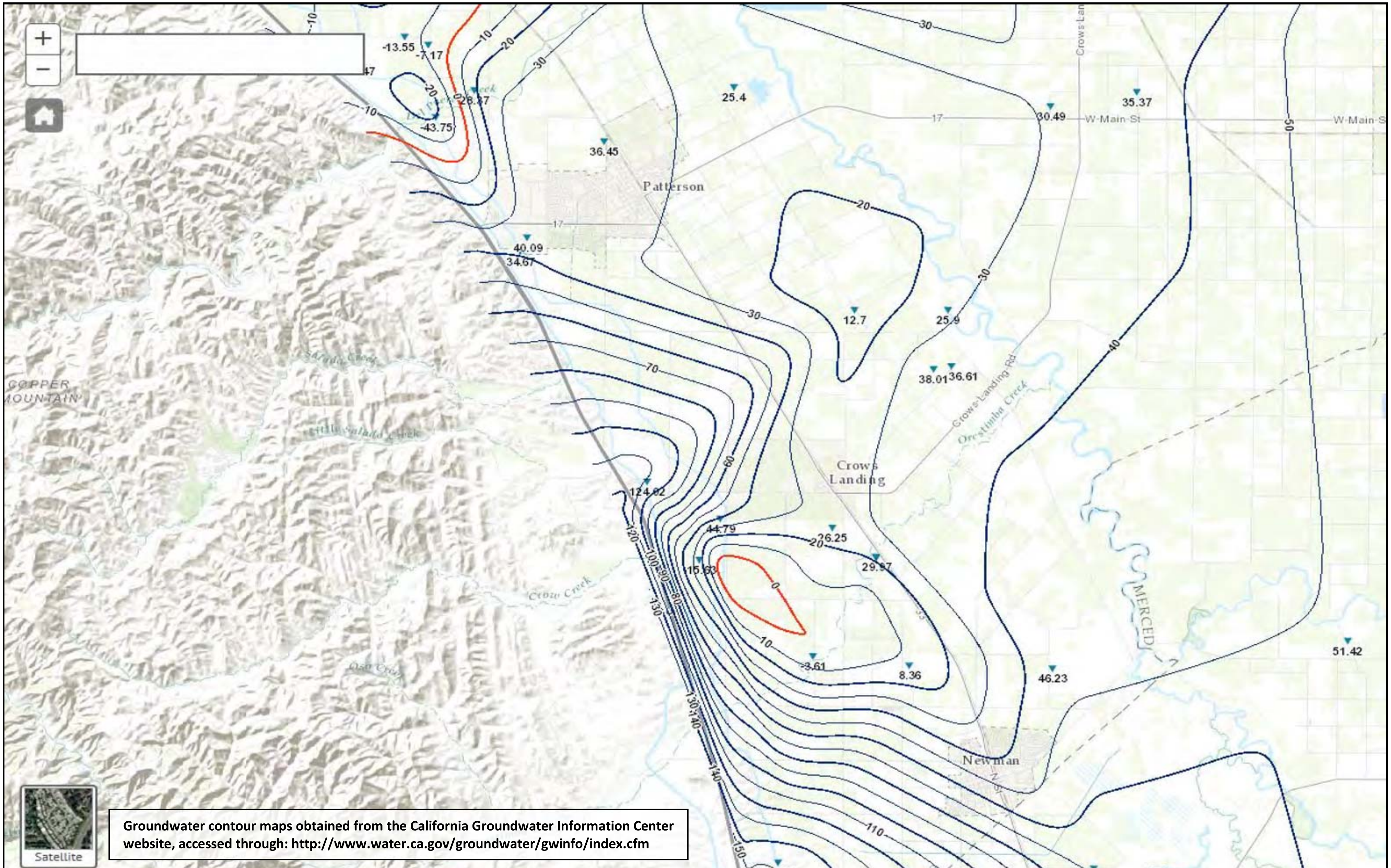
Stanislaus County, California

PROJECT NO.	DATE	DRAWN BY	APPR. BY
STANCO.001	7/28/16	TC	JB

FIGURE B-7

Groundwater Elevation Map for the Confined Aquifer, Spring 2015



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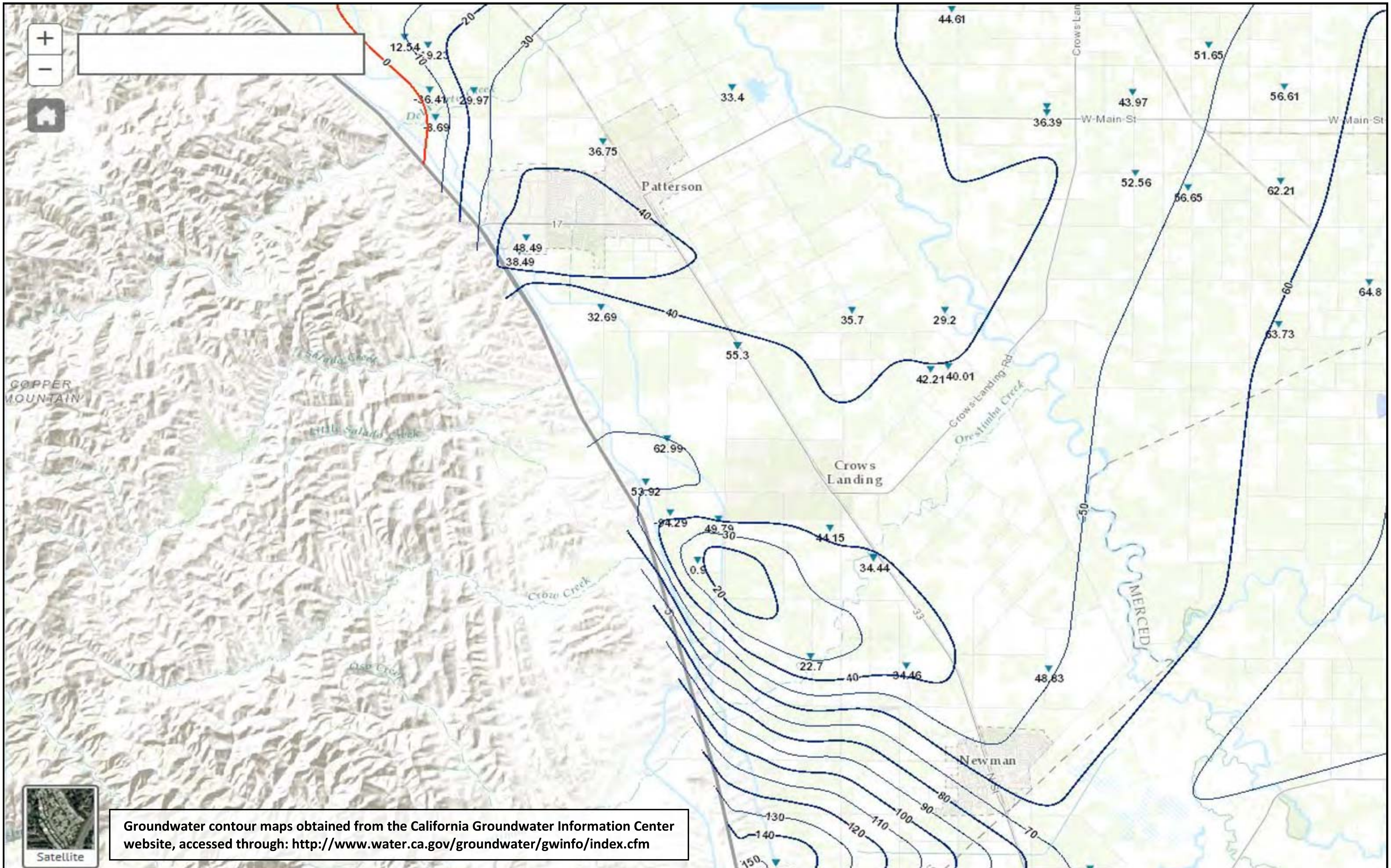
Crows Landing Industrial Business Park
Stanislaus County, California

PROJECT NO.	DATE	DRAWN BY	APPR. BY
STANCO.001	7/28/16	TC	JB

FIGURE B-8

Groundwater Elevation Map for the Confined Aquifer, Fall 2015



Groundwater contour maps obtained from the California Groundwater Information Center website, accessed through: http://www.water.ca.gov/groundwater/gwinfo/index.cfm

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Stanislaus County, California

PROJECT NO.	DATE	DRAWN BY	APPR. BY
STANCO.001	7/28/16	TC	JB

FIGURE B-9

Groundwater Elevation Map for the Confined Aquifer, Spring 2016

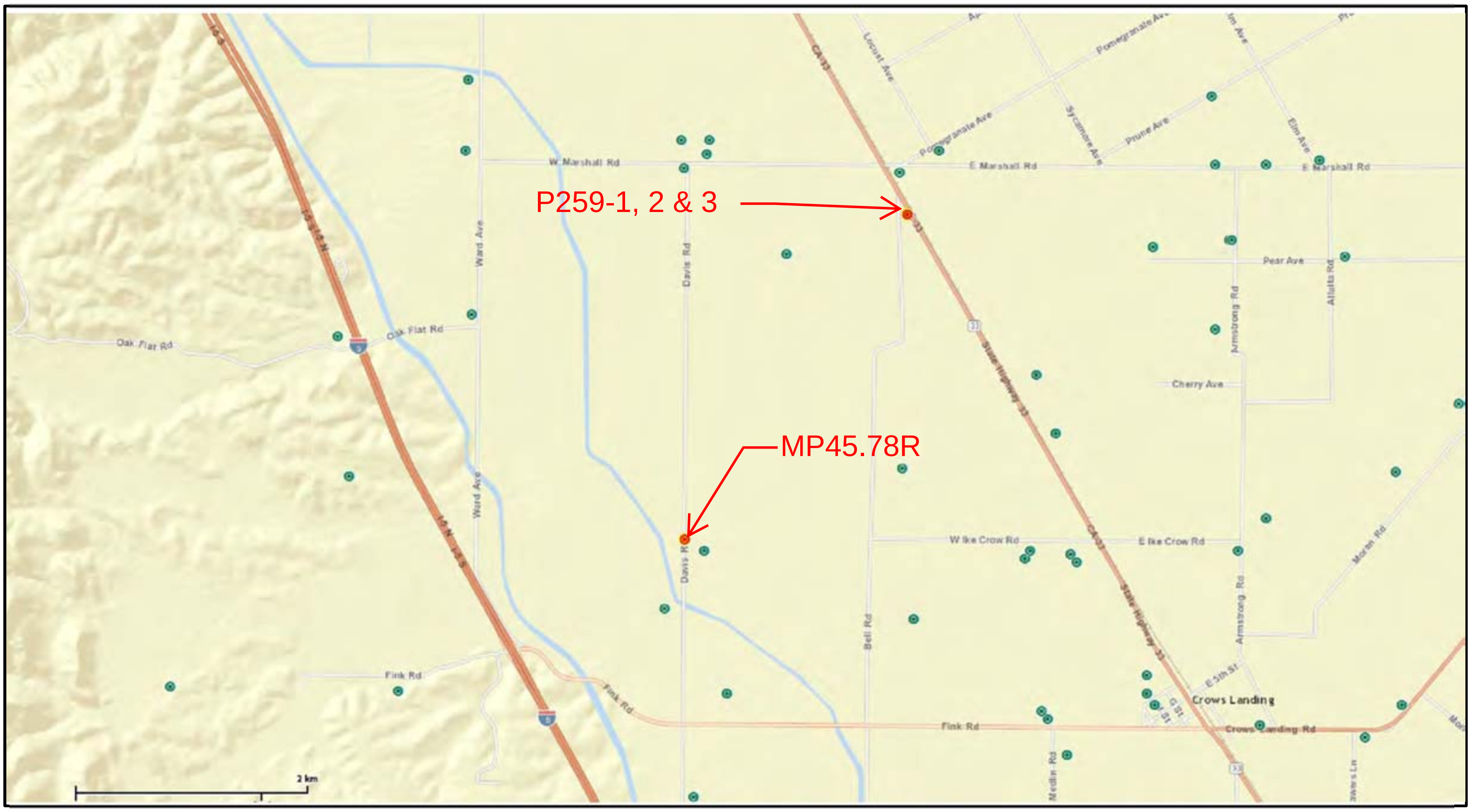
APPENDIX B

CASGEM Well Data and Model Export Data

CASGEM WELL DATA

CASGEM Wells

Crows Landing Industrial Business Park



N

Well

County

Groundwater Basin

Township

Section

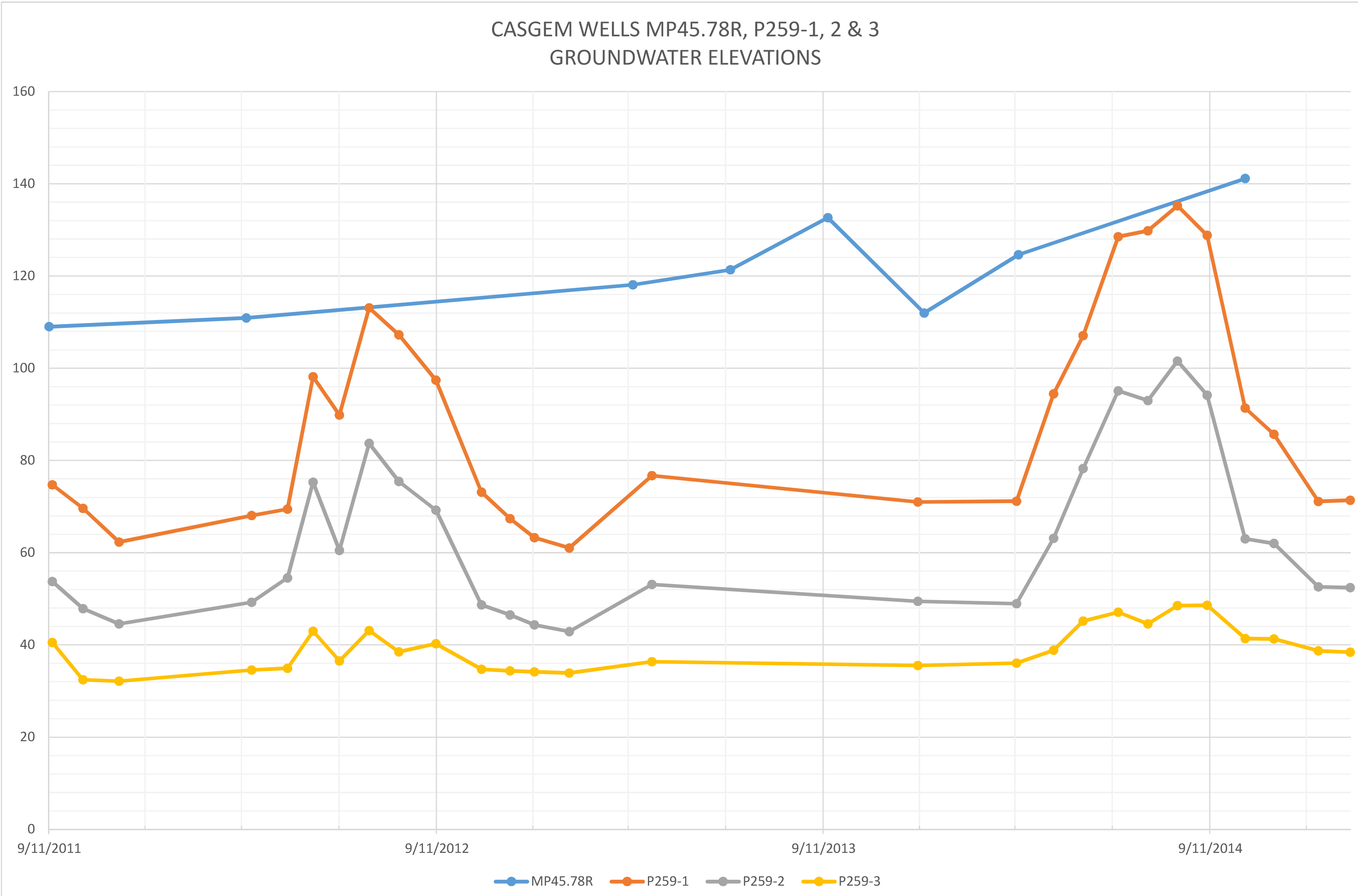
DWR Region Office

Hydrologic Region

Basin Unmonitored

Designated

Print Date:Wed Feb 11 2015 11:13:51 AM


CASGEM WELLS MP45.78R, P259-1, 2 & 3
GROUNDWATER ELEVATIONS
160
140
120
100
80
60
40
20
0
9/11/2011
9/11/2012
9/11/2013
9/11/2014
MP45.78R
P259-1
P259-2
P259-3

CASGEM ID	Local Well Number	Date	Military Time (PST)	Reading @RP	Reading @WS	RP to WS	RP Elevation	GS Elevation	WSE	GS to WS	Measurement Method	Measurement Accuracy	Collecting/ Co-op Agency	Voluntary or CASGEM Measurement
374061N1211212W001	MP45.78R	9/11/2011	00:00	109.500	0.000	109.500	153.500	153.000	44.000	109.000	ST - Steel tape measurement	0.1 Ft	San Luis & Delta-Mendota Water Authority	CASGEM
374061N1211212W001	MP45.78R	9/11/2011	00:00	109.500	0.000	109.500	153.500	153.000	44.000	109.000	ST - Steel tape measurement	0.1 Ft	San Luis & Delta-Mendota Water Authority	CASGEM
374061N1211212W001	MP45.78R	9/11/2011	00:00	109.500	0.000	109.500	153.500	153.000	44.000	109.000	ST - Steel tape measurement	0.1 Ft	San Luis & Delta-Mendota Water Authority	CASGEM
374061N1211212W001	MP45.78R	3/15/2012	00:00	111.400	0.000	111.400	153.500	153.000	42.100	110.900	ST - Steel tape measurement	0.1 Ft	San Luis & Delta-Mendota Water Authority	CASGEM
374061N1211212W001	MP45.78R	3/15/2013	00:00	118.590	0.000	118.590	153.500	153.000	34.910	118.090	ST - Steel tape measurement	0.1 Ft	San Luis & Delta-Mendota Water Authority	CASGEM
374061N1211212W001	MP45.78R	6/15/2013	00:00	121.920	0.000	121.920	153.580	153.000	31.660	121.340	ST - Steel tape measurement	0.1 Ft	San Luis & Delta-Mendota Water Authority	CASGEM
374061N1211212W001	MP45.78R	9/15/2013	00:00	133.240	0.000	133.240	153.580	153.000	20.340	132.660	ST - Steel tape measurement	0.1 Ft	San Luis & Delta-Mendota Water Authority	CASGEM
374061N1211212W001	MP45.78R	12/15/2013	00:00	112.570	0.000	112.570	153.580	153.000	41.010	111.990	ST - Steel tape measurement	0.1 Ft	San Luis & Delta-Mendota Water Authority	CASGEM
374061N1211212W001	MP45.78R	3/14/2014	00:00	125.220	0.000	125.220	153.580	153.000	28.360	124.640	ST - Steel tape measurement	0.1 Ft	San Luis & Delta-Mendota Water Authority	CASGEM
374061N1211212W001	MP45.78R	10/14/2014	00:00	141.730	0.000	141.730	153.580	153.000	11.850	141.150	ST - Steel tape measurement	0.1 Ft	San Luis & Delta-Mendota Water Authority	CASGEM
374316N1210994W001	P259-1	9/14/2011	07:50	73.900	0.000	73.900	82.180	83.000	8.280	74.720	ST - Steel tape measurement	0.01 Ft	San Luis & Delta-Mendota Water Authority	CASGEM
374316N1210994W001	P259-1	3/20/2012	07:45	100.000	32.750	67.250	82.180	83.000	14.930	68.070	ST - Steel tape measurement	0.01 Ft	San Luis & Delta-Mendota Water Authority	CASGEM
374316N1210994W001	P259-1	4/23/2012	07:32	100.000	31.380	68.620	82.180	83.000	13.560	69.440	ST - Steel tape measurement	0.01 Ft	San Luis & Delta-Mendota Water Authority	CASGEM
374316N1210994W001	P259-1	5/17/2012	09:20	100.000	2.670	97.330	82.180	83.000	-15.150	98.150	ST - Steel tape measurement	0.01 Ft	San Luis & Delta-Mendota Water Authority	CASGEM
374316N1210994W001	P259-1	10/13/2011	00:00	100.000	31.200	68.800	82.180	83.000	13.380	69.620	ST - Steel tape measurement	0.01 Ft	San Luis & Delta-Mendota Water Authority	CASGEM
374316N1210994W001	P259-1	11/16/2011	00:00	100.000	38.500	61.500	82.180	83.000	20.680	62.320	ST - Steel tape measurement	0.01 Ft	San Luis & Delta-Mendota Water Authority	CASGEM
374316N1210994W001	P259-1	6/11/2012	07:50	120.000	30.950	89.050	82.180	83.000	-6.870	89.870	ST - Steel tape measurement	0.01 Ft	San Luis & Delta-Mendota Water Authority	CASGEM
374316N1210994W001	P259-1	7/9/2012	08:37	120.000	7.710	112.290	82.180	83.000	-30.110	113.110	ST - Steel tape measurement	0.01 Ft	San Luis & Delta-Mendota Water Authority	CASGEM
374316N1210994W001	P259-1	8/6/2012	08:10	125.000	18.580	106.420	82.180	83.000	-24.240	107.240	ST - Steel tape measurement	0.01 Ft	San Luis & Delta-Mendota Water Authority	CASGEM
374316N1210994W001	P259-1	9/10/2012	07:50	125.000	28.410	96.590	82.180	83.000	-14.410	97.410	ST - Steel tape measurement	0.01 Ft	San Luis & Delta-Mendota Water Authority	CASGEM
374316N1210994W001	P259-1	10/23/2012	07:49	100.000	27.680	72.320	82.180	83.000	9.860	73.140	ST - Steel tape measurement	0.01 Ft	San Luis & Delta-Mendota Water Authority	CASGEM
374316N1210994W001	P259-1	11/19/2012	07:51	100.000	33.430	66.570	82.180	83.000	15.610	67.390	ST - Steel tape measurement	0.01 Ft	San Luis & Delta-Mendota Water Authority	CASGEM
374316N1210994W001	P259-1	12/12/2012	07:49	100.000	37.570	62.430	82.180	83.000	19.750	63.250	ST - Steel tape measurement	0.01 Ft	San Luis & Delta-Mendota Water Authority	CASGEM
374316N1210994W001	P259-1	1/14/2013	08:42	100.000	39.780	60.220	82.180	83.000	21.960	61.040	ST - Steel tape measurement	0.01 Ft	San Luis & Delta-Mendota Water Authority	CASGEM
374316N1210994W001	P259-1	4/2/2013	07:50	100.000	24.110	75.890	82.180	83.000	6.290	76.710	ST - Steel tape measurement	0.01 Ft	San Luis & Delta-Mendota Water Authority	CASGEM
374316N1210994W001	P259-1	3/12/2014	08:19	100.000	29.640	70.360	82.180	83.000	11.820	71.180	ST - Steel tape measurement	0.01 Ft	San Luis & Delta-Mendota Water Authority	CASGEM
374316N1210994W001	P259-1	12/9/2013	08:20	100.000	29.810	70.190	82.180	83.000	11.990	71.010	ST - Steel tape measurement	0.01 Ft	San Luis & Delta-Mendota Water Authority	CASGEM
374316N1210994W001	P259-1	4/16/2014	07:55	100.000	6.380	93.620	82.180	83.000	-11.440	94.440	ST - Steel tape measurement	0.01 Ft	San Luis & Delta-Mendota Water Authority	CASGEM
374316N1210994W001	P259-1	5/14/2014	08:25	125.000	18.720	106.280	82.180	83.000	-24.100	107.100	ST - Steel tape measurement	0.01 Ft	San Luis & Delta-Mendota Water Authority	CASGEM
374316N1210994W001	P259-1	6/16/2014	07:53	130.000	2.280	127.720	82.180	83.000	-45.540	128.540	ST - Steel tape measurement	0.01 Ft	San Luis & Delta-Mendota Water Authority	CASGEM
374316N1210994W001	P259-1	7/14/2014	07:45	150.000	21.010	128.990	82.180	83.000	-46.810	129.810	ST - Steel tape measurement	0.01 Ft	San Luis & Delta-Mendota Water Authority	CASGEM
374316N1210994W001	P259-1	8/11/2014	07:55	150.000	15.570	134.430	82.180	83.000	-52.250	135.250	ST - Steel tape measurement	0.01 Ft	San Luis & Delta-Mendota Water Authority	CASGEM
374316N1210994W001	P259-1	9/8/2014	07:54	150.000	21.980	128.020	82.180	83.000	-45.840	128.840	ST - Steel tape measurement	0.01 Ft	San Luis & Delta-Mendota Water Authority	CASGEM
374316N1210994W001	P259-1	10/14/2014	07:50	150.000	59.470	90.530	82.180	83.000	-8.350	91.350	ST - Steel tape measurement	0.01 Ft	San Luis & Delta-Mendota Water Authority	CASGEM
374316N1210994W001	P259-1	11/10/2014	07:05	145.000	60.130	84.870	82.180	83.000	-2.690	85.690	ST - Steel tape measurement	0.01 Ft	San Luis & Delta-Mendota Water Authority	CASGEM
374316N1210994W001	P259-1	12/22/2014	07:48	140.000	69.720	70.280	82.180	83.000	11.900	71.100	ST - Steel tape measurement	0.01 Ft	San Luis & Delta-Mendota Water Authority	CASGEM

374316N1210994W001	P259-1	1/21/2015	08:03	125.000	54.460	70.540	82.180	83.000	11.640	71.360	ST - Steel tape measurement	0.01 Ft	San Luis & Delta-Mendota Water Authority	CASGEM
374316N1210994W002	P259-2	9/14/2011	08:03	52.900	0.000	52.900	82.130	83.000	29.230	53.770	ST - Steel tape measurement	0.01 Ft	San Luis & Delta-Mendota Water Authority	CASGEM
374316N1210994W002	P259-2	3/20/2012	07:53	70.000	21.620	48.380	82.130	83.000	33.750	49.250	ST - Steel tape measurement	0.01 Ft	San Luis & Delta-Mendota Water Authority	CASGEM
374316N1210994W002	P259-2	4/23/2012	07:44	70.000	16.350	53.650	82.130	83.000	28.480	54.520	ST - Steel tape measurement	0.01 Ft	San Luis & Delta-Mendota Water Authority	CASGEM
374316N1210994W002	P259-2	5/17/2012	09:26	80.000	5.590	74.410	82.130	83.000	7.720	75.280	ST - Steel tape measurement	0.01 Ft	San Luis & Delta-Mendota Water Authority	CASGEM
374316N1210994W002	P259-2	10/13/2011	08:01	70.000	23.000	47.000	82.130	83.000	35.130	47.870	ST - Steel tape measurement	0.01 Ft	San Luis & Delta-Mendota Water Authority	CASGEM
374316N1210994W002	P259-2	11/16/2011	07:47	70.000	26.300	43.700	82.130	83.000	38.430	44.570	ST - Steel tape measurement	0.01 Ft	San Luis & Delta-Mendota Water Authority	CASGEM
374316N1210994W002	P259-2	6/11/2012	08:03	80.000	20.370	59.630	82.130	83.000	22.500	60.500	ST - Steel tape measurement	0.01 Ft	San Luis & Delta-Mendota Water Authority	CASGEM
374316N1210994W002	P259-2	7/9/2012	08:48	100.000	17.150	82.850	82.130	83.000	-0.720	83.720	ST - Steel tape measurement	0.01 Ft	San Luis & Delta-Mendota Water Authority	CASGEM
374316N1210994W002	P259-2	8/6/2012	08:20	100.000	25.390	74.610	82.130	83.000	7.520	75.480	ST - Steel tape measurement	0.01 Ft	San Luis & Delta-Mendota Water Authority	CASGEM
374316N1210994W002	P259-2	9/10/2012	07:59	100.000	31.640	68.360	82.130	83.000	13.770	69.230	ST - Steel tape measurement	0.01 Ft	San Luis & Delta-Mendota Water Authority	CASGEM
374316N1210994W002	P259-2	10/23/2012	07:56	100.000	52.190	47.810	82.130	83.000	34.320	48.680	ST - Steel tape measurement	0.01 Ft	San Luis & Delta-Mendota Water Authority	CASGEM
374316N1210994W002	P259-2	11/19/2012	08:05	100.000	54.390	45.610	82.130	83.000	36.520	46.480	ST - Steel tape measurement	0.01 Ft	San Luis & Delta-Mendota Water Authority	CASGEM
374316N1210994W002	P259-2	12/12/2012	08:01	100.000	56.530	43.470	82.130	83.000	38.660	44.340	ST - Steel tape measurement	0.01 Ft	San Luis & Delta-Mendota Water Authority	CASGEM
374316N1210994W002	P259-2	1/14/2013	08:49	100.000	57.970	42.030	82.130	83.000	40.100	42.900	ST - Steel tape measurement	0.01 Ft	San Luis & Delta-Mendota Water Authority	CASGEM
374316N1210994W002	P259-2	4/2/2013	08:01	100.000	47.760	52.240	82.130	83.000	29.890	53.110	ST - Steel tape measurement	0.01 Ft	San Luis & Delta-Mendota Water Authority	CASGEM
374316N1210994W002	P259-2	3/12/2014	08:28	100.000	51.920	48.080	82.130	83.000	34.050	48.950	ST - Steel tape measurement	0.01 Ft	San Luis & Delta-Mendota Water Authority	CASGEM
374316N1210994W002	P259-2	12/9/2013	08:29	100.000	51.410	48.590	82.130	83.000	33.540	49.460	ST - Steel tape measurement	0.01 Ft	San Luis & Delta-Mendota Water Authority	CASGEM
374316N1210994W002	P259-2	4/16/2014	08:06	100.000	37.740	62.260	82.130	83.000	19.870	63.130	ST - Steel tape measurement	0.01 Ft	San Luis & Delta-Mendota Water Authority	CASGEM
374316N1210994W002	P259-2	5/14/2014	08:38	100.000	22.600	77.400	82.130	83.000	4.730	78.270	ST - Steel tape measurement	0.01 Ft	San Luis & Delta-Mendota Water Authority	CASGEM
374316N1210994W002	P259-2	6/16/2014	08:00	100.000	5.770	94.230	82.130	83.000	-12.100	95.100	ST - Steel tape measurement	0.01 Ft	San Luis & Delta-Mendota Water Authority	CASGEM
374316N1210994W002	P259-2	7/14/2014	07:59	125.000	32.870	92.130	82.130	83.000	-10.000	93.000	ST - Steel tape measurement	0.01 Ft	San Luis & Delta-Mendota Water Authority	CASGEM
374316N1210994W002	P259-2	8/11/2014	08:06	125.000	24.310	100.690	82.130	83.000	-18.560	101.560	ST - Steel tape measurement	0.01 Ft	San Luis & Delta-Mendota Water Authority	CASGEM
374316N1210994W002	P259-2	9/8/2014	08:09	125.000	31.720	93.280	82.130	83.000	-11.150	94.150	ST - Steel tape measurement	0.01 Ft	San Luis & Delta-Mendota Water Authority	CASGEM
374316N1210994W002	P259-2	10/14/2014	08:01	100.000	37.880	62.120	82.130	83.000	20.010	62.990	ST - Steel tape measurement	0.01 Ft	San Luis & Delta-Mendota Water Authority	CASGEM
374316N1210994W002	P259-2	11/10/2014	08:06	100.000	38.880	61.120	82.130	83.000	21.010	61.990	ST - Steel tape measurement	0.01 Ft	San Luis & Delta-Mendota Water Authority	CASGEM
374316N1210994W002	P259-2	12/22/2014	08:05	100.000	48.270	51.730	82.130	83.000	30.400	52.600	ST - Steel tape measurement	0.01 Ft	San Luis & Delta-Mendota Water Authority	CASGEM
374316N1210994W002	P259-2	1/21/2015	08:14	100.000	48.440	51.560	82.130	83.000	30.570	52.430	ST - Steel tape measurement	0.01 Ft	San Luis & Delta-Mendota Water Authority	CASGEM
374316N1210994W003	P259-3	9/14/2011	08:11	39.700	0.000	39.700	82.170	83.000	42.470	40.530	ST - Steel tape measurement	0.01 Ft	San Luis & Delta-Mendota Water Authority	CASGEM
374316N1210994W003	P259-3	3/20/2012	08:01	50.000	16.280	33.720	82.170	83.000	48.450	34.550	ST - Steel tape measurement	0.01 Ft	San Luis & Delta-Mendota Water Authority	CASGEM
374316N1210994W003	P259-3	4/23/2012	07:51	50.000	15.910	34.090	82.170	83.000	48.080	34.920	ST - Steel tape measurement	0.01 Ft	San Luis & Delta-Mendota Water Authority	CASGEM
374316N1210994W003	P259-3	5/17/2012	09:33	60.000	17.860	42.140	82.170	83.000	40.030	42.970	ST - Steel tape measurement	0.01 Ft	San Luis & Delta-Mendota Water Authority	CASGEM
374316N1210994W003	P259-3	10/13/2011	08:08	50.000	18.400	31.600	82.170	83.000	50.570	32.430	ST - Steel tape measurement	0.01 Ft	San Luis & Delta-Mendota Water Authority	CASGEM
374316N1210994W003	P259-3	11/16/2011	00:00	50.000	18.700	31.300	82.170	83.000	50.870	32.130	ST - Steel tape measurement	0.01 Ft	San Luis & Delta-Mendota Water Authority	CASGEM
374316N1210994W003	P259-3	6/11/2012	08:11	60.000	24.310	35.690	82.170	83.000	46.480	36.520	ST - Steel tape measurement	0.01 Ft	San Luis & Delta-Mendota Water Authority	CASGEM
374316N1210994W003	P259-3	7/9/2012	08:57	80.000	37.710	42.290	82.170	83.000	39.880	43.120	ST - Steel tape measurement	0.01 Ft	San Luis & Delta-Mendota Water Authority	CASGEM
374316N1210994W003	P259-3	8/6/2012	08:28	80.000	42.330	37.670	82.170	83.000	44.500	38.500	ST - Steel tape measurement	0.01 Ft	San Luis & Delta-Mendota Water Authority	CASGEM

374316N1210994W003	P259-3	9/10/2012	08:05	80.000	40.580	39.420	82.170	83.000	42.750	40.250	ST - Steel tape measurement	0.01 Ft	San Luis & Delta-Mendota Water Authority	CASGEM
374316N1210994W003	P259-3	10/23/2012	08:05	80.000	46.120	33.880	82.170	83.000	48.290	34.710	ST - Steel tape measurement	0.01 Ft	San Luis & Delta-Mendota Water Authority	CASGEM
374316N1210994W003	P259-3	11/19/2012	08:14	80.000	46.470	33.530	82.170	83.000	48.640	34.360	ST - Steel tape measurement	0.01 Ft	San Luis & Delta-Mendota Water Authority	CASGEM
374316N1210994W003	P259-3	12/12/2012	08:11	80.000	46.680	33.320	82.170	83.000	48.850	34.150	ST - Steel tape measurement	0.01 Ft	San Luis & Delta-Mendota Water Authority	CASGEM
374316N1210994W003	P259-3	1/14/2013	08:57	80.000	46.940	33.060	82.170	83.000	49.110	33.890	ST - Steel tape measurement	0.01 Ft	San Luis & Delta-Mendota Water Authority	CASGEM
374316N1210994W003	P259-3	4/2/2013	08:10	80.000	44.490	35.510	82.170	83.000	46.660	36.340	ST - Steel tape measurement	0.01 Ft	San Luis & Delta-Mendota Water Authority	CASGEM
374316N1210994W003	P259-3	3/12/2014	08:39	100.000	64.760	35.240	82.170	83.000	46.930	36.070	ST - Steel tape measurement	0.01 Ft	San Luis & Delta-Mendota Water Authority	CASGEM
374316N1210994W003	P259-3	12/9/2013	08:38	100.000	65.300	34.700	82.170	83.000	47.470	35.530	ST - Steel tape measurement	0.01 Ft	San Luis & Delta-Mendota Water Authority	CASGEM
374316N1210994W003	P259-3	4/16/2014	08:14	100.000	62.000	38.000	82.170	83.000	44.170	38.830	ST - Steel tape measurement	0.01 Ft	San Luis & Delta-Mendota Water Authority	CASGEM
374316N1210994W003	P259-3	5/14/2014	08:46	100.000	55.650	44.350	82.170	83.000	37.820	45.180	ST - Steel tape measurement	0.01 Ft	San Luis & Delta-Mendota Water Authority	CASGEM
374316N1210994W003	P259-3	6/16/2014	08:10	100.000	53.750	46.250	82.170	83.000	35.920	47.080	ST - Steel tape measurement	0.01 Ft	San Luis & Delta-Mendota Water Authority	CASGEM
374316N1210994W003	P259-3	7/14/2014	08:12	100.000	56.250	43.750	82.170	83.000	38.420	44.580	ST - Steel tape measurement	0.01 Ft	San Luis & Delta-Mendota Water Authority	CASGEM
374316N1210994W003	P259-3	8/11/2014	08:13	100.000	52.300	47.700	82.170	83.000	34.470	48.530	ST - Steel tape measurement	0.01 Ft	San Luis & Delta-Mendota Water Authority	CASGEM
374316N1210994W003	P259-3	9/8/2014	08:16	100.000	52.220	47.780	82.170	83.000	34.390	48.610	ST - Steel tape measurement	0.01 Ft	San Luis & Delta-Mendota Water Authority	CASGEM
374316N1210994W003	P259-3	10/14/2014	08:15	100.000	59.450	40.550	82.170	83.000	41.620	41.380	ST - Steel tape measurement	0.01 Ft	San Luis & Delta-Mendota Water Authority	CASGEM
374316N1210994W003	P259-3	11/10/2014	08:12	100.000	59.520	40.480	82.170	83.000	41.690	41.310	ST - Steel tape measurement	0.01 Ft	San Luis & Delta-Mendota Water Authority	CASGEM
374316N1210994W003	P259-3	12/22/2014	08:16	100.000	62.120	37.880	82.170	83.000	44.290	38.710	ST - Steel tape measurement	0.01 Ft	San Luis & Delta-Mendota Water Authority	CASGEM
374316N1210994W003	P259-3	1/21/2015	08:24	100.000	62.390	37.610	82.170	83.000	44.560	38.440	ST - Steel tape measurement	0.01 Ft	San Luis & Delta-Mendota Water Authority	CASGEM

MODEL EXPORT DATA

Phase 1 Potable Water Average Day Demand Junctions

ID	Demand (gpm)	Elevation (ft)	Head (ft)	Pressure (psi)
J27	11.91	193	304.21	48.19
J17	25.66	188	303.66	50.11
J18	8.97	179	302.65	53.58
J74	5.5	178	302.58	53.98
J62	10.06	178	302.63	54
J20	16.03	176	302.59	54.85
J19	11.76	173	302.59	56.15
J26	13.61	173	302.59	56.15
J76	4.51	171	302.58	57.01
J25	7.86	171	302.59	57.02
J72	13.2	171	302.62	57.03
J24	11.74	169	302.59	57.89
J66	14.56	169	302.61	57.89
J70	11.42	169	302.63	57.9
J64	7.13	169	302.67	57.92
J78	1.34	165	302.57	59.61
J30	13.58	165	302.58	59.61
J22	12.11	164	302.58	60.05
J80	13.61	164	302.59	60.05
J23	42.4	161	302.6	61.35
J60	15.42	157	302.58	63.08
J58	34.72	156	302.58	63.51
J56	30.67	156	302.59	63.52
J21	14.76	154	302.59	64.38
J50	23.86	146	302.29	67.72
J7	2.99	146	302.29	67.72
J82	44.81	145	302.32	68.16
J54	33.8	144	302.29	68.59

Phase 1 Potable Water Maximum Day Demand Junctions

ID	Demand (gpm)	Elevation (ft)	Head (ft)	Pressure (psi)
J27	23.83	193	302.13	47.28
J17	51.38	188	300.15	48.59
J18	17.96	179	296.48	50.91
J74	11	178	296.26	51.24
J62	20.15	178	296.43	51.32
J20	32.1	176	296.3	52.13
J19	23.55	173	296.29	53.42
J26	27.25	173	296.29	53.42
J76	9.03	171	296.24	54.27
J25	15.74	171	296.3	54.29
J72	26.43	171	296.39	54.33
J24	23.51	169	296.3	55.16
J66	29.15	169	296.34	55.18
J70	22.88	169	296.41	55.21
J64	14.28	169	296.56	55.27
J78	2.69	165	296.23	56.86
J30	27.19	165	296.25	56.87
J22	24.25	164	296.25	57.31
J80	27.26	164	296.3	57.33
J23	84.91	161	296.31	58.63
J60	30.88	157	296.26	60.34
J58	69.53	156	296.25	60.77
J56	61.42	156	296.27	60.78
J21	29.55	154	296.27	61.64
J50	47.78	146	295.18	64.64
J7	5.99	146	295.19	64.64
J82	89.62	145	295.29	65.12
J54	67.68	144	295.19	65.51

Phase 1 Potable Water Peak Hour Demand Junctions

ID	Demand (gpm)	Elevation (ft)	Head (ft)	Pressure (psi)
J18	35.89	179	274.29	41.29
J74	21.99	178	273.49	41.38
J62	40.28	178	274.1	41.64
J20	64.15	176	273.62	42.3
J17	102.7	188	287.5	43.12
J19	47.07	173	273.57	43.58
J26	54.47	173	273.58	43.58
J27	47.65	193	294.64	44.04
J76	18.06	171	273.42	44.38
J25	31.46	171	273.62	44.47
J72	52.84	171	273.93	44.6
J24	46.99	169	273.62	45.33
J66	58.27	169	273.78	45.4
J70	45.72	169	274.03	45.51
J64	28.54	169	274.57	45.75
J78	5.38	165	273.36	46.95
J30	54.34	165	273.45	46.99
J22	48.47	164	273.46	47.43
J80	54.49	164	273.62	47.5
J23	169.71	161	273.68	48.82
J60	61.72	157	273.49	50.48
J58	138.97	156	273.45	50.89
J56	122.76	156	273.52	50.92
J21	59.07	154	273.51	51.78
J50	95.51	146	269.59	53.55
J7	11.97	146	269.61	53.56
J82	179.23	145	269.98	54.15
J54	135.28	144	269.62	54.43

Phase 1 Potable Water Average Day Demand Pipes

ID	From Node	To Node	Length (ft)	Diameter (in)	Roughness	Flow (gpm)	Velocity (ft/s)	Headloss (ft)	HL/1000 (ft/k-ft)
P217	RES9010	J27	1,330.64	12	130	458.01	1.3	0.79	0.6
P183	J17	J27	962.49	12.00	130	-446.10	1.27	0.55	0.57
P181	J64	J17	1,946.12	12.00	130	-420.44	1.19	0.99	0.51
P161	J64	J70	445.7	12	130	168.21	0.48	0.04	0.09
P153	J66	J64	1,021.01	12.00	130	-132.22	0.38	0.06	0.06
P145	J18	J64	486.16	12.00	130	-112.87	0.32	0.02	0.04
P233	J78	J82	4,057.96	12	100	105.46	0.3	0.26	0.06
P143	J62	J18	375.06	12.00	130	-103.91	0.29	0.01	0.04
P141	J20	J62	1,161.91	12	130	-93.84	0.27	0.04	0.03
P163	J70	J72	329.98	12	130	79.05	0.22	0.01	0.02
P171	J70	J80	1,410.69	12	130	77.74	0.22	0.03	0.02
P165	J72	J26	1,651.26	12	130	65.85	0.19	0.03	0.02
P155	J66	J23	525.40	12	130	62.45	0.18	0.01	0.01
P179	J78	J80	1,259.60	12	130	-64.57	0.18	0.02	0.02
P235	J82	J54	1,206.50	12	100	60.65	0.17	0.03	0.02
P151	J56	J66	1,692.40	12	130	-55.21	0.16	0.02	0.01
P173	J26	J74	619.75	12	130	52.24	0.15	0.01	0.01
P175	J74	J76	598.96	12	130	46.75	0.13	0.01	0.01
P137	J20	J19	561.79	12	130	41.09	0.12	0	0.01
P177	J76	J78	677.06	12	130	42.23	0.12	0	0.01
P135	J21	J20	1,604.30	12	130	-36.73	0.1	0.01	0.01
P147	J58	J60	803.00	12	130	-31.08	0.09	0	0
P223	J22	J19	2,367.35	12	130	-29.33	0.08	0.01	0
P149	J60	J56	604.43	12	130	-24.54	0.07	0	0
P93	J50	J54	818.7	12	130	-23.86	0.07	0	0
P133	J60	J21	428.80	12	130	-21.97	0.06	0	0
P225	J23	J24	2,298.86	12	130	20.05	0.06	0	0
P131	J58	J22	1,453.22	12	130	-8.99	0.03	0	0
P129	J30	J58	1,932.13	12	130	-5.36	0.02	0	0
P167	J24	J25	309.06	12	130	8.31	0.02	0	0
P221	J30	J22	2,589.55	12.00	130	-8.22	0.02	0	0
P91	J7	J54	714.38	12	130	-2.99	0.01	0	0
P169	J25	J80	411.15	12	130	0.45	0	0	0

Phase 1 Potable Water Maximum Day Demand Pipes

ID	From Node	To Node	Length (ft)	Diameter (in)	Roughness	Flow (gpm)	Velocity (ft/s)	Headloss (ft)	HL/1000 (ft/k-ft)
P217	RES9010	J27	1,330.64	12	130	916.98	2.6	2.87	2.16
P183	J17	J27	962.49	12	130	-893.16	2.53	1.98	2.06
P181	J64	J17	1,946.12	12	130	-841.78	2.39	3.59	1.84
P161	J64	J70	445.70	12	130	336.77	0.96	0.15	0.34
P153	J66	J64	1,021.01	12	130	-264.73	0.75	0.22	0.22
P145	J18	J64	486.16	12	130	-226.01	0.64	0.08	0.16
P233	J78	J82	4,057.96	12.00	100	211.07	0.6	0.94	0.23
P143	J62	J18	375.06	12.00	130	-208.05	0.59	0.05	0.14
P141	J20	J62	1,161.91	12	130	-187.9	0.53	0.13	0.11
P163	J70	J72	329.98	12	130	158.26	0.45	0.03	0.08
P171	J70	J80	1,410.69	12	130	155.63	0.44	0.11	0.08
P165	J72	J26	1,651.26	12.00	130	131.82	0.37	0.1	0.06
P179	J78	J80	1,259.60	12	130	-129.22	0.37	0.07	0.06
P155	J66	J23	525.4	12	130	125.01	0.35	0.03	0.05
P235	J82	J54	1,206.50	12	100	121.45	0.34	0.1	0.08
P151	J56	J66	1,692.40	12	130	-110.57	0.31	0.07	0.04
P173	J26	J74	619.75	12	130	104.57	0.3	0.02	0.04
P175	J74	J76	598.96	12	130	93.57	0.27	0.02	0.03
P177	J76	J78	677.06	12	130	84.54	0.24	0.02	0.03
P137	J20	J19	561.79	12	130	82.27	0.23	0.01	0.02
P135	J21	J20	1,604.30	12	130	-73.53	0.21	0.03	0.02
P147	J58	J60	803	12	130	-62.25	0.18	0.01	0.01
P223	J22	J19	2,367.35	12	130	-58.72	0.17	0.03	0.01
P149	J60	J56	604.43	12.00	130	-49.15	0.14	0.01	0.01
P93	J50	J54	818.70	12.00	130	-47.78	0.14	0.01	0.01
P133	J60	J21	428.80	12	130	-43.98	0.12	0	0.01
P225	J23	J24	2,298.86	12.00	130	40.10	0.11	0.02	0.01
P131	J58	J22	1,453.22	12	130	-18.01	0.05	0	0
P167	J24	J25	309.06	12	130	16.59	0.05	0	0
P221	J30	J22	2,589.55	12	130	-16.46	0.05	0	0
P129	J30	J58	1,932.13	12	130	-10.73	0.03	0	0
P91	J7	J54	714.38	12	130	-5.99	0.02	0	0
P169	J25	J80	411.15	12	130	0.85	0	0	0

Phase 1 Potable Water Peak Hour Demand Pipes

ID	From Node	To Node	Length (ft)	Diameter (in)	Rough-ness	Flow (gpm)	Velocity (ft/s)	Headloss (ft)	HL/1000 (ft/k-ft)
P217	RES9010	J27	1,330.64	12	130	1,832.99	5.2	10.36	7.79
P183	J17	J27	962.49	12	130	-1,785.34	5.06	7.14	7.41
P181	J64	J17	1,946.12	12	130	-1,682.63	4.77	12.93	6.64
P161	J64	J70	445.70	12	130	673.19	1.91	0.54	1.22
P153	J66	J64	1,021.01	12	130	-529.16	1.5	0.8	0.78
P145	J18	J64	486.16	12	130	-451.75	1.28	0.28	0.58
P233	J78	J82	4,057.96	12.00	100	421.98	1.2	3.38	0.83
P143	J62	J18	375.06	12.00	130	-415.86	1.18	0.19	0.5
P141	J20	J62	1,161.91	12	130	-375.57	1.07	0.48	0.41
P163	J70	J72	329.98	12	130	316.36	0.9	0.1	0.3
P171	J70	J80	1,410.69	12	130	311.1	0.88	0.41	0.29
P165	J72	J26	1,651.26	12.00	130	263.53	0.75	0.35	0.21
P179	J78	J80	1,259.60	12	130	-258.36	0.73	0.26	0.21
P155	J66	J23	525.4	12	130	249.9	0.71	0.1	0.19
P235	J82	J54	1,206.50	12	100	242.75	0.69	0.36	0.3
P151	J56	J66	1,692.40	12	130	-220.99	0.63	0.26	0.15
P173	J26	J74	619.75	12	130	209.05	0.59	0.09	0.14
P175	J74	J76	598.96	12	130	187.06	0.53	0.07	0.11
P177	J76	J78	677.06	12	130	169	0.48	0.06	0.09
P137	J20	J19	561.79	12	130	164.44	0.47	0.05	0.09
P135	J21	J20	1,604.30	12	130	-146.98	0.42	0.12	0.07
P147	J58	J60	803	12	130	-124.41	0.35	0.04	0.05
P223	J22	J19	2,367.35	12	130	-117.37	0.33	0.11	0.05
P149	J60	J56	604.43	12.00	130	-98.22	0.28	0.02	0.03
P93	J50	J54	818.70	12.00	130	-95.51	0.27	0.03	0.03
P133	J60	J21	428.80	12	130	-87.92	0.25	0.01	0.03
P225	J23	J24	2,298.86	12.00	130	80.19	0.23	0.05	0.02
P131	J58	J22	1,453.22	12	130	-36	0.1	0.01	0.01
P167	J24	J25	309.06	12	130	33.2	0.09	0	0
P221	J30	J22	2,589.55	12	130	-32.91	0.09	0.01	0
P129	J30	J58	1,932.13	12	130	-21.44	0.06	0	0
P91	J7	J54	714.38	12	130	-11.97	0.03	0	0
P169	J25	J80	411.15	12	130	1.74	0	0	0

Phases 1 and 2 Potable Water Average Day Demand Junctions

ID	Demand (gpm)	Elevation (ft)	Head (ft)	Pressure (psi)
J27	12.39	193	304.13	48.15
J17	23.74	188	303.54	50.06
J18	7.82	179	302.42	53.48
J74	5.04	178	302.34	53.88
J62	10.34	178	302.4	53.9
J20	13.91	176	302.36	54.75
J26	11.78	173	302.35	56.05
J19	10.76	173	302.36	56.05
J76	4.7	171	302.33	56.91
J25	6.9	171	302.36	56.92
J72	11.34	171	302.39	56.93
J24	10.51	169	302.36	57.78
J66	12.64	169	302.38	57.79
J70	9.92	169	302.4	57.8
J64	6.22	169	302.44	57.82
J78	1.32	165	302.33	59.5
J30	36.85	165	302.34	59.51
J22	11.51	164	302.35	59.95
J80	11.82	164	302.36	59.95
J23	36.83	161	302.37	61.26
J60	12.49	157	302.35	62.98
J58	33.05	156	302.35	63.41
J56	26.62	156	302.36	63.42
J21	12.82	154	302.35	64.28
J11	25.84	151	301.82	65.35
J9	19.72	150	301.82	65.79
J10	42.17	148	301.82	66.65
J48	34.72	147	301.82	67.08
J50	16.85	146	301.83	67.52
J7	3.42	146	301.84	67.52
J82	46.62	145	301.9	67.98
J54	29.47	144	301.84	68.39
J40	11.57	142	301.82	69.25
J6	18.44	141	301.86	69.7
J42	8.3	137	301.83	71.42
J5	10.2	131	301.87	74.04
J44	5.99	131	301.87	74.04
J46	2.06	131	301.9	74.05
J32	2.03	130	301.9	74.48
J3	12.37	130	301.9	74.48
J1	2.28	124	301.9	77.08
J2	12.67	124	301.9	77.08

Phases 1 and 2 Potable Water Maximum Day Demand Junctions

ID	Demand (gpm)	Elevation (ft)	Head (ft)	Pressure (psi)
J27	24.79	193	303.29	47.79
J17	47.48	188	302.13	49.45
J18	15.64	179	300.02	52.44
J62	20.67	178	299.97	52.85
J74	10.09	178	299.99	52.86
J20	27.82	176	299.86	53.67
J19	21.52	173	299.84	54.96
J26	23.56	173	300	55.03
J25	13.8	171	299.98	55.89
J76	9.39	171	299.99	55.89
J72	22.68	171	300.02	55.91
J66	25.27	169	299.96	56.74
J24	21.03	169	299.97	56.75
J70	19.83	169	300.03	56.78
J64	12.44	169	300.09	56.8
J30	73.69	165	299.79	58.4
J78	2.64	165	299.99	58.49
J22	23.02	164	299.8	58.84
J80	23.63	164	299.99	58.92
J23	73.66	161	299.96	60.21
J60	24.98	157	299.83	61.89
J58	66.11	156	299.8	62.31
J56	53.24	156	299.85	62.33
J21	25.65	154	299.83	63.19
J11	51.67	151	300.08	64.6
J9	39.43	150	300.13	65.05
J10	84.33	148	300.03	65.88
J48	69.43	147	300.02	66.31
J7	6.83	146	299.98	66.72
J50	33.7	146	299.99	66.73
J82	93.24	145	299.98	67.15
J54	58.93	144	299.98	67.59
J40	23.14	142	300.13	68.52
J6	36.88	141	300.63	69.17
J42	16.61	137	300.25	70.74
J5	20.39	131	300.64	73.51
J44	11.97	131	300.65	73.51
J46	4.11	131	300.94	73.64
J32	4.07	130	300.65	73.94
J3	24.73	130	300.89	74.05
J1	4.55	124	300.68	76.55
J2	25.35	124	300.78	76.6

Phases 1 and 2 Potable Water Peak Hour Demand Junctions

ID	Demand (gpm)	Elevation (ft)	Head (ft)	Pressure (psi)
J27	49.55	193	300.23	46.46
J17	94.92	188	297.04	47.24
J18	31.26	179	291.28	48.65
J62	41.33	178	291.12	49.02
J74	20.16	178	291.4	49.14
J20	55.62	176	290.72	49.71
J19	43.03	173	290.67	50.98
J26	47.11	173	291.4	51.3
J25	27.58	171	291.3	52.12
J76	18.77	171	291.4	52.17
J72	45.35	171	291.42	52.18
J66	50.53	169	291.14	52.92
J24	42.04	169	291.27	52.98
J70	39.64	169	291.43	53.05
J64	24.88	169	291.53	53.09
J30	147.32	165	290.49	54.37
J78	5.28	165	291.41	54.77
J22	46.02	164	290.54	54.83
J80	47.24	164	291.35	55.18
J23	147.26	161	291.14	56.39
J60	49.94	157	290.64	57.9
J58	132.16	156	290.53	58.29
J56	106.44	156	290.7	58.37
J21	51.27	154	290.64	59.21
J11	103.3	151	292.87	61.47
J9	78.84	150	293.11	62.01
J10	168.61	148	292.61	62.66
J48	138.81	147	292.52	63.05
J7	13.66	146	292.06	63.29
J50	67.38	146	292.22	63.36
J82	186.36	145	291.96	63.68
J54	117.81	144	292.06	64.15
J40	46.25	142	293.14	65.49
J6	73.74	141	295.62	67
J42	33.2	137	293.73	67.91
J5	40.77	131	295.65	71.34
J44	23.93	131	295.67	71.35
J32	8.13	130	295.56	71.74
J46	8.23	131	297.04	71.95
J3	49.45	130	296.8	72.27
J1	9.11	124	295.73	74.41
J2	50.67	124	296.25	74.64

Phases 1 and 2 Potable Water Average Day Demand Pipes

ID	From Node	To Node	Length (ft)	Diameter (in)	Roughness	Flow (gpm)	Velocity (ft/s)	Headloss (ft)	HL/1000 (ft/k-ft)
P217	RES9010	J27	1,330.64	12	130	479.72	1.36	0.87	0.65
P183	J17	J27	962.49	12	130	-467.32	1.33	0.6	0.62
P181	J64	J17	1,946.12	12	130	-443.58	1.26	1.09	0.56
P161	J64	J70	445.70	12	130	181.70	0.52	0.05	0.11
P219	RES9008	J46	665.41	12	100	166.30	0.47	0.1	0.15
P233	J78	J82	4,057.96	12	100	138.4	0.39	0.43	0.11
P153	J66	J64	1,021.01	12	130	-138.16	0.39	0.07	0.06
P59	J46	J44	515.4	12	130	136.29	0.39	0.03	0.06
P145	J18	J64	486.16	12	130	-117.5	0.33	0.02	0.05
P143	J62	J18	375.06	12	130	-109.68	0.31	0.02	0.04
P61	J44	J42	958.43	12	130	101.66	0.29	0.04	0.04
P141	J20	J62	1,161.91	12	130	-99.34	0.28	0.04	0.04
P235	J82	J54	1,206.50	12	100	90.38	0.26	0.06	0.05
P163	J70	J72	329.98	12	130	87.33	0.25	0.01	0.03
P179	J78	J80	1,259.60	12	130	-85.25	0.24	0.03	0.03
P171	J70	J80	1,410.69	12	130	84.45	0.24	0.04	0.03
P165	J72	J26	1,651.26	12	130	75.99	0.22	0.04	0.02
P155	J66	J23	525.40	12	130	66.85	0.19	0.01	0.02
P173	J26	J74	619.75	12	130	64.21	0.18	0.01	0.02
P175	J74	J76	598.96	12	130	59.16	0.17	0.01	0.01
P151	J56	J66	1,692.40	12	130	-58.67	0.17	0.02	0.01
P93	J50	J54	818.70	12	130	-57.5	0.16	0.01	0.01
P177	J76	J78	677.06	12	130	54.47	0.15	0.01	0.01
P67	J40	J42	1,035.34	12	130	-53.45	0.15	0.01	0.01
P137	J20	J19	561.79	12	130	46.61	0.13	0	0.01
P147	J58	J60	803.00	12	130	-45.56	0.13	0.01	0.01
P73	J40	J11	559.91	12	130	41.88	0.12	0	0.01
P87	J48	J50	980.29	12	130	-40.65	0.12	0.01	0.01
P69	J42	J9	1,285.03	12	130	39.91	0.11	0.01	0.01
P135	J21	J20	1,604.30	12	130	-38.83	0.11	0.01	0.01
P223	J22	J19	2,367.35	12	130	-35.85	0.1	0.01	0.01
P149	J60	J56	604.43	12	130	-32.05	0.09	0	0
P225	J23	J24	2,298.86	12	130	30.02	0.09	0.01	0
P63	J44	J5	574.96	12	130	28.64	0.08	0	0
P57	J3	J46	704.66	12	130	-27.96	0.08	0	0
P75	J10	J48	1,660.98	12	130	-26.12	0.07	0	0
P133	J60	J21	428.80	12	130	-26.00	0.07	0	0
P71	J9	J48	1,871.25	12	130	20.19	0.06	0	0
P167	J24	J25	309.06	12	130	19.51	0.06	0	0
P129	J30	J58	1,932.13	12	130	-19.36	0.05	0	0
P79	J5	J6	1,278.10	12	130	18.44	0.05	0	0
P221	J30	J22	2,589.55	12	130	-17.49	0.05	0	0
P77	J11	J10	981.29	12	130	16.05	0.05	0	0
P55	J2	J3	2,071.12	12	130	-15.59	0.04	0	0
P169	J25	J80	411.15	12	130	12.61	0.04	0	0
P131	J58	J22	1,453.22	12	130	-6.85	0.02	0	0
P91	J7	J54	714.38	12	130	-3.42	0.01	0	0
P53	J1	J2	2,835.47	12	130	-2.91	0.01	0	0
P237	J82	J32	13,724.69	12	100	1.4	0	0	0
P99	J1	J32	995.3	12	130	0.64	0	0	0

Phases 1 and 2 Potable Water Maximum Day Demand Pipes

ID	From Node	To Node	Length (ft)	Diameter (in)	Roughness	Flow (gpm)	Velocity (ft/s)	Headloss (ft)	HL/1000 (ft/k-ft)
P217	RES9010	J27	1,330.64	12	130	693.42	1.97	1.71	1.29
P183	J17	J27	962.49	12	130	-668.63	1.9	1.16	1.2
P181	J64	J17	1,946.12	12	130	-621.15	1.76	2.04	1.05
P219	RES9008	J46	665.41	12	100	598.58	1.7	1.06	1.59
P59	J46	J44	515.4	12	130	445.26	1.26	0.29	0.57
P61	J44	J42	958.43	12	130	376.01	1.07	0.4	0.41
P145	J18	J64	486.16	12	130	-211.69	0.6	0.07	0.14
P161	J64	J70	445.7	12	130	199.43	0.57	0.06	0.13
P153	J66	J64	1,021.01	12	130	-197.59	0.56	0.13	0.13
P143	J62	J18	375.06	12	130	-196.05	0.56	0.05	0.12
P67	J40	J42	1,035.34	12	130	-190.89	0.54	0.12	0.12
P141	J20	J62	1,161.91	12	130	-175.38	0.5	0.12	0.1
P69	J42	J9	1,285.03	12	130	168.51	0.48	0.12	0.09
P73	J40	J11	559.91	12	130	167.76	0.48	0.05	0.09
P57	J3	J46	704.66	12	130	-149.21	0.42	0.05	0.07
P151	J56	J66	1,692.40	12	130	-140.65	0.4	0.11	0.07
P71	J9	J48	1,871.25	12	130	129.08	0.37	0.11	0.06
P55	J2	J3	2,071.12	12	130	-124.47	0.35	0.11	0.05
P77	J11	J10	981.29	12	130	116.09	0.33	0.05	0.05
P53	J1	J2	2,835.47	12	130	-99.12	0.28	0.1	0.04
P147	J58	J60	803	12	130	-98.21	0.28	0.03	0.03
P99	J1	J32	995.30	12	130	94.57	0.27	0.03	0.03
P171	J70	J80	1,410.69	12	130	92.53	0.26	0.04	0.03
P87	J48	J50	980.29	12	130	91.41	0.26	0.03	0.03
P237	J82	J32	13,724.69	12	100	-90.50	0.26	0.66	0.05
P149	J60	J56	604.43	12	130	-87.41	0.25	0.02	0.03
P163	J70	J72	329.98	12	130	87.07	0.25	0.01	0.03
P137	J20	J19	561.79	12	130	86.12	0.24	0.02	0.03
P169	J25	J80	411.15	12	130	-76.81	0.22	0.01	0.02
P223	J22	J19	2,367.35	12	130	-64.6	0.18	0.04	0.02
P165	J72	J26	1,651.26	12	130	64.39	0.18	0.03	0.02
P167	J24	J25	309.06	12	130	-63.01	0.18	0	0.02
P135	J21	J20	1,604.30	12	130	-61.44	0.17	0.02	0.01
P93	J50	J54	818.70	12	130	57.7	0.16	0.01	0.01
P63	J44	J5	574.96	12	130	57.28	0.16	0.01	0.01
P225	J23	J24	2,298.86	12	130	-41.98	0.12	0.02	0.01
P173	J26	J74	619.75	12	130	40.83	0.12	0	0.01
P129	J30	J58	1,932.13	12	130	-39.43	0.11	0.01	0.01
P79	J5	J6	1,278.10	12	130	36.88	0.1	0.01	0.01
P133	J60	J21	428.80	12	130	-35.79	0.1	0	0.01
P221	J30	J22	2,589.55	12	130	-34.26	0.1	0.01	0
P75	J10	J48	1,660.98	12	130	31.75	0.09	0.01	0
P155	J66	J23	525.4	12	130	31.67	0.09	0	0
P175	J74	J76	598.96	12	130	30.74	0.09	0	0
P177	J76	J78	677.06	12	130	21.35	0.06	0	0
P233	J78	J82	4,057.96	12	100	10.8	0.03	0	0
P235	J82	J54	1,206.50	12	100	8.06	0.02	0	0
P179	J78	J80	1,259.60	12	130	7.91	0.02	0	0
P131	J58	J22	1,453.22	12	130	-7.32	0.02	0	0
P91	J7	J54	714.38	12	130	-6.83	0.02	0	0

Phases 1 and 2 Potable Water Peak Hour Demand Pipes

ID	From Node	To Node	Length (ft)	Diameter (in)	Roughness	Flow (gpm)	Velocity (ft/s)	Headloss (ft)	HL/1000 (ft/k-ft)
P219	RES9008	J46	665.41	12	100	1,376.91	3.91	4.96	7.45
P217	RES9010	J27	1,330.64	12	130	1,206.06	3.42	4.77	3.59
P183	J17	J27	962.49	12	130	-1,156.51	3.28	3.19	3.32
P181	J64	J17	1,946.12	12	130	-1,061.59	3.01	5.51	2.83
P59	J46	J44	515.4	12	130	1,025.42	2.91	1.37	2.66
P61	J44	J42	958.43	12	130	886.97	2.52	1.95	2.03
P67	J40	J42	1,035.34	12	130	-447.6	1.27	0.59	0.57
P145	J18	J64	486.16	12	130	-414.38	1.18	0.24	0.5
P69	J42	J9	1,285.03	12	130	406.18	1.15	0.61	0.48
P73	J40	J11	559.91	12	130	401.34	1.14	0.26	0.47
P143	J62	J18	375.06	12	130	-383.12	1.09	0.16	0.43
P153	J66	J64	1,021.01	12	130	-359.04	1.02	0.39	0.38
P57	J3	J46	704.66	12	130	-343.27	0.97	0.25	0.35
P141	J20	J62	1,161.91	12	130	-341.78	0.97	0.4	0.35
P71	J9	J48	1,871.25	12	130	327.34	0.93	0.6	0.32
P87	J48	J50	980.29	12	130	317.97	0.9	0.3	0.3
P77	J11	J10	981.29	12	130	298.04	0.85	0.26	0.27
P55	J2	J3	2,071.12	12	130	-293.82	0.83	0.54	0.26
P151	J56	J66	1,692.40	12	130	-290.03	0.82	0.43	0.26
P161	J64	J70	445.7	12	130	263.3	0.75	0.1	0.21
P93	J50	J54	818.7	12	130	250.59	0.71	0.16	0.2
P53	J1	J2	2,835.47	12	130	-243.15	0.69	0.52	0.18
P99	J1	J32	995.30	12	130	234.04	0.66	0.17	0.17
P237	J82	J32	13,724.69	12	100	-225.91	0.64	3.6	0.26
P147	J58	J60	803	12	130	-198.87	0.56	0.1	0.13
P169	J25	J80	411.15	12	130	-198.40	0.56	0.05	0.13
P149	J60	J56	604.43	12	130	-183.59	0.52	0.07	0.11
P167	J24	J25	309.06	12	130	-170.82	0.48	0.03	0.1
P137	J20	J19	561.79	12	130	169.66	0.48	0.05	0.09
P233	J78	J82	4,057.96	12	100	-158.66	0.45	0.55	0.14
P75	J10	J48	1,660.98	12	130	129.43	0.37	0.1	0.06
P171	J70	J80	1,410.69	12	130	129.31	0.37	0.08	0.06
P225	J23	J24	2,298.86	12	130	-128.78	0.37	0.13	0.06
P223	J22	J19	2,367.35	12	130	-126.64	0.36	0.13	0.06
P235	J82	J54	1,206.50	12	100	-119.12	0.34	0.1	0.08
P135	J21	J20	1,604.30	12	130	-116.50	0.33	0.08	0.05
P179	J78	J80	1,259.60	12	130	116.33	0.33	0.06	0.05
P63	J44	J5	574.96	12	130	114.51	0.32	0.03	0.05
P163	J70	J72	329.98	12	130	94.34	0.27	0.01	0.03
P129	J30	J58	1,932.13	12	130	-79	0.22	0.04	0.02
P79	J5	J6	1,278.10	12	130	73.74	0.21	0.03	0.02
P221	J30	J22	2,589.55	12	130	-68.32	0.19	0.05	0.02
P133	J60	J21	428.8	12	130	-65.22	0.19	0.01	0.02
P165	J72	J26	1,651.26	12	130	49	0.14	0.02	0.01
P177	J76	J78	677.06	12	130	-37.05	0.11	0	0.01
P155	J66	J23	525.40	12	130	18.48	0.05	0	0
P175	J74	J76	598.96	12	130	-18.27	0.05	0	0
P91	J7	J54	714.38	12	130	-13.66	0.04	0	0
P131	J58	J22	1,453.22	12	130	-12.3	0.03	0	0
P173	J26	J74	619.75	12	130	1.89	0.01	0	0

Phases 1,2 and 3 Potable Water Average Day Demand Junctions

ID	Demand (gpm)	Elevation (ft)	Head (ft)	Pressure (psi)
J1	9.5	124	302.14	77.19
J10	46.73	148	302.04	66.74
J11	30.81	151	302.04	65.45
J12	18.28	139	302.07	70.66
J13	18.05	134	302.09	72.83
J14	14	137	302.15	71.56
J15	14.31	129	302.18	75.04
J16	13	135	302.52	72.59
J17	26.3	188	303.52	50.06
J18	8.68	179	302.4	53.47
J19	12.09	173	302.33	56.04
J2	16.4	124	302.08	77.16
J20	15.41	176	302.34	54.74
J21	14.2	154	302.33	64.27
J22	12.96	164	302.32	59.93
J23	40.83	161	302.36	61.25
J24	11.66	169	302.36	57.78
J25	7.64	171	302.36	56.92
J26	12.99	173	302.35	56.05
J27	12.78	193	304.13	48.15
J28	18.66	130	302.07	74.56
J3	14.14	130	302.05	74.55
J30	44.93	165	302.31	59.5
J32	8.66	130	302.21	74.62
J34	23.84	136	302.16	72
J36	10.93	134	302.13	72.85
J38	15.23	132	302.11	73.71
J4	19.31	124	302.12	77.18
J40	26.13	142	302.05	69.35
J42	13.97	137	302.05	71.52
J44	7.45	131	302.05	74.11
J46	8.07	131	302.05	74.11
J48	34.89	147	302.04	67.18
J5	11.29	131	302.04	74.11
J50	17.49	146	302.04	67.61
J52	11.84	144	302.04	68.48
J54	32.96	144	302.04	68.48
J56	29.36	156	302.33	63.41
J58	36.79	156	302.32	63.4
J6	16.28	141	302.04	69.78
J60	13.89	157	302.33	62.97
J62	11.49	178	302.39	53.9
J64	6.8	169	302.43	57.82
J66	13.84	169	302.36	57.79
J7	3.42	146	302.04	67.61
J70	10.98	169	302.39	57.8
J72	12.58	171	302.38	56.93
J74	5.59	178	302.35	53.88
J76	4.84	171	302.34	56.91
J78	1.46	165	302.34	59.51
J8	5.81	148	302.04	66.74
J80	13.09	164	302.36	59.95
J82	48.07	145	302.1	68.07
J9	19.35	150	302.04	65.88

Phases 1,2 and 3 Potable Water Maximum Day Demand Junctions

ID	Demand (gpm)	Elevation (ft)	Head (ft)	Pressure (psi)
J1	19.02	124	301.47	76.9
J10	93.51	148	301.01	66.3
J11	61.65	151	301.08	65.03
J12	36.58	139	301.24	70.3
J13	36.13	134	301.3	72.49
J14	28.02	137	301.45	71.26
J15	28.64	129	301.49	74.74
J16	26.01	135	302.1	72.41
J17	52.62	188	302.4	49.57
J18	17.36	179	300.52	52.66
J19	24.2	173	300.31	55.16
J2	32.82	124	301.44	76.88
J20	30.83	176	300.33	53.87
J21	28.41	154	300.3	63.39
J22	25.93	164	300.26	59.04
J23	81.71	161	300.48	60.44
J24	23.33	169	300.53	56.99
J25	15.29	171	300.54	56.13
J26	25.99	173	300.58	55.28
J27	25.56	193	303.45	47.86
J28	37.35	130	301.43	74.28
J3	28.29	130	301.44	74.28
J30	89.92	165	300.24	58.6
J32	17.33	130	301.55	74.33
J34	47.71	136	301.45	71.69
J36	21.87	134	301.42	72.54
J38	30.47	132	301.43	73.41
J4	38.64	124	301.44	76.88
J40	52.29	142	301.16	68.96
J42	27.96	137	301.18	71.14
J44	14.91	131	301.25	73.77
J46	16.14	131	301.44	73.85
J48	69.81	147	300.99	66.72
J5	22.6	131	301.17	73.74
J50	35	146	300.95	67.14
J52	23.7	144	300.95	68.01
J54	65.95	144	300.92	67.99
J56	58.75	156	300.32	62.54
J58	73.61	156	300.26	62.51
J6	32.58	141	301.04	69.35
J60	27.79	157	300.3	62.09
J62	22.99	178	300.47	53.07
J64	13.61	169	300.61	57.03
J66	27.69	169	300.48	56.97
J7	6.85	146	300.93	67.13
J70	21.97	169	300.58	57.01
J72	25.17	171	300.58	56.15
J74	11.18	178	300.58	53.11
J76	9.68	171	300.58	56.15
J78	2.92	165	300.58	58.75
J8	11.62	148	300.95	66.27
J80	26.19	164	300.56	59.17
J82	96.14	145	300.84	67.52
J9	38.72	150	301.08	65.46

Phases 1,2 and 3 Potable Water Peak Hour Demand Junctions

ID	Demand (gpm)	Elevation (ft)	Head (ft)	Pressure (psi)
J1	38.04	124	297.75	75.29
J10	187.02	148	295.87	64.07
J11	123.31	151	296.18	62.91
J12	73.16	139	296.83	68.39
J13	72.25	134	297.06	70.65
J14	56.05	137	297.58	69.58
J15	57.29	129	297.69	73.09
J16	52.03	135	299.24	71.16
J17	105.24	188	297.39	47.4
J18	34.72	179	291.97	48.95
J19	48.4	173	291.2	51.22
J2	65.65	124	297.76	75.29
J20	61.66	176	291.27	49.95
J21	56.81	154	291.18	59.44
J22	51.86	164	291.03	55.04
J23	163.42	161	291.83	56.69
J24	46.66	169	292.05	53.32
J25	30.58	171	292.1	52.47
J26	51.97	173	292.25	51.67
J27	51.12	193	300.43	46.55
J28	74.7	130	297.7	72.67
J3	56.58	130	297.82	72.72
J30	179.83	165	290.97	54.58
J32	34.65	130	297.86	72.73
J34	95.41	136	297.53	69.99
J36	43.74	134	297.51	70.85
J38	60.93	132	297.6	71.76
J4	77.27	124	297.64	75.24
J40	104.58	142	296.51	66.95
J42	55.91	137	296.63	69.17
J44	29.82	131	296.96	71.91
J46	32.28	131	297.88	72.31
J48	139.63	147	295.75	64.45
J5	45.2	131	296.63	71.77
J50	69.99	146	295.53	64.79
J52	47.4	144	295.55	65.67
J54	131.91	144	295.34	65.58
J56	117.49	156	291.26	58.61
J58	147.23	156	291.03	58.51
J6	65.16	141	296.01	67.17
J60	55.58	157	291.17	58.14
J62	45.98	178	291.77	49.3
J64	27.22	169	292.26	53.41
J66	55.38	169	291.83	53.22
J7	13.7	146	295.43	64.75
J70	43.95	169	292.23	53.4
J72	50.34	171	292.23	52.53
J74	22.35	178	292.27	49.51
J76	19.37	171	292.3	52.56
J78	5.85	165	292.34	55.18
J8	23.25	148	295.54	63.93
J80	52.37	164	292.18	55.54
J82	192.28	145	294.71	64.87
J9	77.44	150	296.17	63.34

Phases 1,2 and 3 Potable Water Average Day Demand Pipes

ID	From Node	To Node	Length (ft)	Diameter (in)	Roughness	Flow (gpm)	Velocity (ft/s)	Headloss (ft)	HL/1000 (ft/k-ft)
P101	J32	J16	1,027.54	12	130	-313.79	0.89	0.3	0.3
P105	J34	J36	958.33	12	130	98.05	0.28	0.03	0.03
P107	J36	J14	730.64	12	130	-93.12	0.26	0.02	0.03
P109	J14	J15	711.86	12	130	-107.12	0.3	0.03	0.04
P111	J15	J32	676.29	12	130	-121.44	0.34	0.03	0.05
P113	J1	J4	1,145.92	12	130	63.77	0.18	0.02	0.02
P115	J4	J38	1,133.89	12	130	44.46	0.13	0.01	0.01
P117	J38	J36	723.34	12	130	-73.03	0.21	0.01	0.02
P119	J36	J13	957.7	12	130	107.21	0.3	0.04	0.04
P121	J13	J12	716.15	12	130	89.16	0.25	0.02	0.03
P123	J12	J42	937.53	12	130	70.88	0.2	0.02	0.02
P125	J38	J28	1,127.74	12	130	102.27	0.29	0.04	0.04
P127	J28	J46	953.64	12	130	83.6	0.24	0.02	0.03
P129	J30	J58	1,932.13	12	130	-23.93	0.07	0	0
P131	J58	J22	1,453.22	12	130	-5.7	0.02	0	0
P133	J60	J21	428.8	12	130	-26.16	0.07	0	0
P135	J21	J20	1,604.30	12	130	-40.35	0.11	0.01	0.01
P137	J20	J19	561.79	12	130	51.75	0.15	0.01	0.01
P141	J20	J62	1,161.91	12	130	-107.52	0.31	0.05	0.04
P143	J62	J18	375.06	12	130	-119.01	0.34	0.02	0.05
P145	J18	J64	486.16	12	130	-127.68	0.36	0.03	0.06
P147	J58	J60	803	12	130	-55.02	0.16	0.01	0.01
P149	J60	J56	604.43	12	130	-42.74	0.12	0	0.01
P151	J56	J66	1,692.40	12	130	-72.1	0.2	0.03	0.02
P153	J66	J64	1,021.01	12	130	-139.72	0.4	0.07	0.07
P155	J66	J23	525.4	12	130	53.78	0.15	0.01	0.01
P161	J64	J70	445.7	12	130	169.07	0.48	0.04	0.09
P163	J70	J72	329.98	12	130	78.04	0.22	0.01	0.02
P165	J72	J26	1,651.26	12	130	65.46	0.19	0.03	0.02
P167	J24	J25	309.06	12	130	1.29	0	0	0
P169	J25	J80	411.15	12	130	-6.35	0.02	0	0
P171	J70	J80	1,410.69	12	130	80.06	0.23	0.03	0.02
P173	J26	J74	619.75	12	130	52.47	0.15	0.01	0.01
P175	J74	J76	598.96	12	130	46.89	0.13	0.01	0.01
P177	J76	J78	677.06	12	130	42.04	0.12	0	0.01
P179	J78	J80	1,259.60	12	130	-60.62	0.17	0.02	0.01
P181	J64	J17	1,946.12	12	130	-443.27	1.26	1.09	0.56
P183	J17	J27	962.49	12	130	-469.57	1.33	0.6	0.63
P197	J34	J40	2,632.93	12	130	108.39	0.31	0.11	0.04
P213	J16	J34	2,141.15	12	130	230.29	0.65	0.36	0.17
P217	RES9010	J27	1,330.64	12	130	482.35	1.37	0.87	0.66
P219	RES9008	J46	665.41	12	100	-109.4	0.31	0.05	0.07
P221	J30	J22	2,589.55	12	130	-21	0.06	0.01	0
P223	J22	J19	2,367.35	12	130	-39.66	0.11	0.02	0.01
P225	J23	J24	2,298.86	12	130	12.95	0.04	0	0

P233	J78	J82	4,057.96	12	100	101.2	0.29	0.24	0.06
P235	J82	J54	1,206.50	12	100	88.28	0.25	0.06	0.05
P237	J82	J32	13,724.69	12	100	-35.14	0.1	0.11	0.01
P245	J16	RES9022	563.36	12	130	-557.07	1.58	0.48	0.86
P53	J1	J2	2,835.47	12	130	75.27	0.21	0.06	0.02
P55	J2	J3	2,071.12	12	130	58.87	0.17	0.03	0.01
P57	J3	J46	704.66	12	130	44.74	0.13	0.01	0.01
P59	J46	J44	515.4	12	130	10.88	0.03	0	0
P61	J44	J42	958.43	12	130	-30.83	0.09	0	0
P63	J44	J5	574.96	12	130	34.25	0.1	0	0
P67	J40	J42	1,035.34	12	130	15.15	0.04	0	0
P69	J42	J9	1,285.03	12	130	41.23	0.12	0.01	0.01
P71	J9	J48	1,871.25	12	130	21.88	0.06	0	0
P73	J40	J11	559.91	12	130	67.12	0.19	0.01	0.02
P75	J10	J48	1,660.98	12	130	-10.42	0.03	0	0
P77	J11	J10	981.29	12	130	36.31	0.1	0.01	0.01
P79	J5	J6	1,278.10	12	130	22.96	0.07	0	0
P81	J6	J52	1,314.19	12	130	6.68	0.02	0	0
P83	J52	J8	376.91	12	130	14.08	0.04	0	0
P85	J8	J50	455.92	12	130	8.28	0.02	0	0
P87	J48	J50	980.29	12	130	-23.43	0.07	0	0
P89	J52	J7	903.39	12	130	-19.25	0.05	0	0
P91	J7	J54	714.38	12	130	-22.67	0.06	0	0
P93	J50	J54	818.7	12	130	-32.64	0.09	0	0
P99	J1	J32	995.3	12	130	-148.55	0.42	0.07	0.07

Phases 1,2 and 3 Potable Water Maximum Day Demand Pipes

ID	From Node	To Node	Length (ft)	Diameter (in)	Roughness	Flow (gpm)	Velocity (ft/s)	Headloss (ft)	HL/1000 (ft/k-ft)
P101	J32	J16	1,027.54	12	130	-432.32	1.23	0.55	0.54
P105	J34	J36	958.33	12	130	87.34	0.25	0.03	0.03
P107	J36	J14	730.64	12	130	-104.73	0.3	0.03	0.04
P109	J14	J15	711.86	12	130	-132.75	0.38	0.04	0.06
P111	J15	J32	676.29	12	130	-161.39	0.46	0.06	0.09
P113	J1	J4	1,145.92	12	130	87.94	0.25	0.03	0.03
P115	J4	J38	1,133.89	12	130	49.3	0.14	0.01	0.01
P117	J38	J36	723.34	12	130	29.89	0.08	0	0
P119	J36	J13	957.7	12	130	200.08	0.57	0.12	0.13
P121	J13	J12	716.15	12	130	163.96	0.47	0.06	0.09
P123	J12	J42	937.53	12	130	127.37	0.36	0.05	0.06
P125	J38	J28	1,127.74	12	130	-11.05	0.03	0	0
P127	J28	J46	953.64	12	130	-48.4	0.14	0.01	0.01
P129	J30	J58	1,932.13	12	130	-48.34	0.14	0.02	0.01
P131	J58	J22	1,453.22	12	130	-5.57	0.02	0	0
P133	J60	J21	428.8	12	130	-37.1	0.11	0	0.01
P135	J21	J20	1,604.30	12	130	-65.51	0.19	0.03	0.02
P137	J20	J19	561.79	12	130	97.27	0.28	0.02	0.03
P141	J20	J62	1,161.91	12	130	-193.61	0.55	0.14	0.12
P143	J62	J18	375.06	12	130	-216.61	0.61	0.06	0.15
P145	J18	J64	486.16	12	130	-233.97	0.66	0.08	0.17
P147	J58	J60	803	12	130	-116.39	0.33	0.04	0.05
P149	J60	J56	604.43	12	130	-107.07	0.3	0.02	0.04
P151	J56	J66	1,692.40	12	130	-165.82	0.47	0.15	0.09
P153	J66	J64	1,021.01	12	130	-199.34	0.57	0.13	0.13
P155	J66	J23	525.4	12	130	5.83	0.02	0	0
P161	J64	J70	445.7	12	130	132.64	0.38	0.03	0.06
P163	J70	J72	329.98	12	130	42.04	0.12	0	0.01
P165	J72	J26	1,651.26	12	130	16.87	0.05	0	0
P167	J24	J25	309.06	12	130	-99.21	0.28	0.01	0.04
P169	J25	J80	411.15	12	130	-114.49	0.32	0.02	0.05
P171	J70	J80	1,410.69	12	130	68.63	0.19	0.03	0.02
P173	J26	J74	619.75	12	130	-9.12	0.03	0	0
P175	J74	J76	598.96	12	130	-20.3	0.06	0	0
P177	J76	J78	677.06	12	130	-29.98	0.09	0	0
P179	J78	J80	1,259.60	12	130	72.05	0.2	0.02	0.02
P181	J64	J17	1,946.12	12	130	-579.56	1.64	1.8	0.92
P183	J17	J27	962.49	12	130	-632.18	1.79	1.04	1.08
P197	J34	J40	2,632.93	12	130	184.02	0.52	0.29	0.11
P213	J16	J34	2,141.15	12	130	319.06	0.91	0.65	0.31
P217	RES9010	J27	1,330.64	12	130	657.74	1.87	1.55	1.17
P219	RES9008	J46	665.41	12	100	425.87	1.21	0.56	0.85
P221	J30	J22	2,589.55	12	130	-41.58	0.12	0.02	0.01
P223	J22	J19	2,367.35	12	130	-73.07	0.21	0.05	0.02
P225	J23	J24	2,298.86	12	130	-75.88	0.22	0.05	0.02

P233	J78	J82	4,057.96	12	100	-104.95	0.3	0.26	0.06
P235	J82	J54	1,206.50	12	100	-106.64	0.3	0.08	0.07
P237	J82	J32	13,724.69	12	100	-94.45	0.27	0.72	0.05
P245	J16	RES9022	563.36	12	130	-777.4	2.21	0.9	1.59
P53	J1	J2	2,835.47	12	130	52.19	0.15	0.03	0.01
P55	J2	J3	2,071.12	12	130	19.37	0.05	0	0
P57	J3	J46	704.66	12	130	-8.92	0.03	0	0
P59	J46	J44	515.4	12	130	352.41	1	0.19	0.37
P61	J44	J42	958.43	12	130	138.4	0.39	0.06	0.07
P63	J44	J5	574.96	12	130	199.1	0.56	0.07	0.13
P67	J40	J42	1,035.34	12	130	-79.92	0.23	0.02	0.02
P69	J42	J9	1,285.03	12	130	157.9	0.45	0.11	0.08
P71	J9	J48	1,871.25	12	130	119.18	0.34	0.09	0.05
P73	J40	J11	559.91	12	130	211.65	0.6	0.08	0.14
P75	J10	J48	1,660.98	12	130	56.48	0.16	0.02	0.01
P77	J11	J10	981.29	12	130	149.99	0.43	0.07	0.08
P79	J5	J6	1,278.10	12	130	176.5	0.5	0.13	0.1
P81	J6	J52	1,314.19	12	130	143.92	0.41	0.09	0.07
P83	J52	J8	376.91	12	130	41.84	0.12	0	0.01
P85	J8	J50	455.92	12	130	30.21	0.09	0	0
P87	J48	J50	980.29	12	130	105.85	0.3	0.04	0.04
P89	J52	J7	903.39	12	130	78.38	0.22	0.02	0.02
P91	J7	J54	714.38	12	130	71.53	0.2	0.01	0.02
P93	J50	J54	818.7	12	130	101.06	0.29	0.03	0.04
P99	J1	J32	995.3	12	130	-159.15	0.45	0.08	0.08

Phases 1,2 and 3 Potable Water Maximum Day Demand Pipes

ID	From Node	To Node	Length (ft)	Diameter (in)	Roughness	Flow (gpm)	Velocity (ft/s)	Headloss (ft)	HL/1000 (ft/k-ft)
P101	J32	J16	1,027.54	12	130	-710.02	2.01	1.38	1.34
P105	J34	J36	958.33	12	130	77.34	0.22	0.02	0.02
P107	J36	J14	730.64	12	130	-169.34	0.48	0.07	0.09
P109	J14	J15	711.86	12	130	-225.39	0.64	0.11	0.16
P111	J15	J32	676.29	12	130	-282.68	0.8	0.17	0.24
P113	J1	J4	1,145.92	12	130	171.92	0.49	0.11	0.1
P115	J4	J38	1,133.89	12	130	94.65	0.27	0.04	0.03
P117	J38	J36	723.34	12	130	199.5	0.57	0.09	0.13
P119	J36	J13	957.7	12	130	402.45	1.14	0.45	0.47
P121	J13	J12	716.15	12	130	330.2	0.94	0.23	0.33
P123	J12	J42	937.53	12	130	257.03	0.73	0.19	0.2
P125	J38	J28	1,127.74	12	130	-165.78	0.47	0.1	0.09
P127	J28	J46	953.64	12	130	-240.48	0.68	0.17	0.18
P129	J30	J58	1,932.13	12	130	-96.73	0.27	0.06	0.03
P131	J58	J22	1,453.22	12	130	-10.14	0.03	0	0
P133	J60	J21	428.8	12	130	-71.45	0.2	0.01	0.02
P135	J21	J20	1,604.30	12	130	-128.27	0.36	0.09	0.06
P137	J20	J19	561.79	12	130	193.5	0.55	0.07	0.12
P141	J20	J62	1,161.91	12	130	-383.42	1.09	0.5	0.43
P143	J62	J18	375.06	12	130	-429.41	1.22	0.2	0.53
P145	J18	J64	486.16	12	130	-464.13	1.32	0.3	0.61
P147	J58	J60	803	12	130	-233.82	0.66	0.14	0.17
P149	J60	J56	604.43	12	130	-217.95	0.62	0.09	0.15
P151	J56	J66	1,692.40	12	130	-335.44	0.95	0.57	0.34
P153	J66	J64	1,021.01	12	130	-381.93	1.08	0.44	0.43
P155	J66	J23	525.4	12	130	-8.89	0.03	0	0
P161	J64	J70	445.7	12	130	148.02	0.42	0.03	0.07
P163	J70	J72	329.98	12	130	3.18	0.01	0	0
P165	J72	J26	1,651.26	12	130	-47.16	0.13	0.01	0.01
P167	J24	J25	309.06	12	130	-218.96	0.62	0.05	0.15
P169	J25	J80	411.15	12	130	-249.54	0.71	0.08	0.19
P171	J70	J80	1,410.69	12	130	100.9	0.29	0.05	0.04
P173	J26	J74	619.75	12	130	-99.14	0.28	0.02	0.04
P175	J74	J76	598.96	12	130	-121.49	0.34	0.03	0.05
P177	J76	J78	677.06	12	130	-140.86	0.4	0.05	0.07
P179	J78	J80	1,259.60	12	130	201.01	0.57	0.16	0.13
P181	J64	J17	1,946.12	12	130	-1,021.30	2.9	5.13	2.64
P183	J17	J27	962.49	12	130	-1,126.55	3.2	3.04	3.16
P197	J34	J40	2,632.93	12	130	363	1.03	1.02	0.39
P213	J16	J34	2,141.15	12	130	535.76	1.52	1.71	0.8
P217	RES9010	J27	1,330.64	12	130	1,177.66	3.34	4.57	3.43
P219	RES9008	J46	665.41	12	100	1,246.55	3.54	4.12	6.2
P221	J30	J22	2,589.55	12	130	-83.1	0.24	0.07	0.03
P223	J22	J19	2,367.35	12	130	-145.1	0.41	0.17	0.07
P225	J23	J24	2,298.86	12	130	-172.3	0.49	0.22	0.1

P233	J78	J82	4,057.96	12	100	-347.72	0.99	2.36	0.58
P235	J82	J54	1,206.50	12	100	-329.69	0.94	0.64	0.53
P237	J82	J32	13,724.69	12	100	-210.31	0.6	3.15	0.23
P245	J16	RES9022	563.36	12	100	-1,297.80	3.68	3.76	6.68
P53	J1	J2	2,835.47	12	130	-27.58	0.08	0.01	0
P55	J2	J3	2,071.12	12	130	-93.22	0.26	0.06	0.03
P57	J3	J46	704.66	12	130	-149.8	0.42	0.05	0.08
P59	J46	J44	515.4	12	130	823.98	2.34	0.91	1.77
P61	J44	J42	958.43	12	130	340.31	0.97	0.33	0.34
P63	J44	J5	574.96	12	130	453.85	1.29	0.34	0.59
P67	J40	J42	1,035.34	12	130	-193.62	0.55	0.13	0.12
P69	J42	J9	1,285.03	12	130	347.81	0.99	0.46	0.36
P71	J9	J48	1,871.25	12	130	270.36	0.77	0.42	0.22
P73	J40	J11	559.91	12	130	452.04	1.28	0.33	0.58
P75	J10	J48	1,660.98	12	130	141.72	0.4	0.11	0.07
P77	J11	J10	981.29	12	130	328.74	0.93	0.32	0.32
P79	J5	J6	1,278.10	12	130	408.66	1.16	0.62	0.48
P81	J6	J52	1,314.19	12	130	343.49	0.97	0.46	0.35
P83	J52	J8	376.91	12	130	91.94	0.26	0.01	0.03
P85	J8	J50	455.92	12	130	68.69	0.19	0.01	0.02
P87	J48	J50	980.29	12	130	272.45	0.77	0.22	0.23
P89	J52	J7	903.39	12	130	204.15	0.58	0.12	0.13
P91	J7	J54	714.38	12	130	190.45	0.54	0.08	0.12
P93	J50	J54	818.7	12	130	271.15	0.77	0.19	0.23
P99	J1	J32	995.3	12	130	-182.38	0.52	0.11	0.11

Phase 1 Nonpotable Fire Flow Demand Junctions

ID	Static Demand (gpm)	Static Pressure (psi)	Static Head (ft)	Fire-Flow Demand (gpm)	Residual Pressure (psi)	Available Flow at Hydrant (gpm)	Available Flow Pressure (psi)
J27	24.09	66.33	346.07	3,000.00	29.83	3,466.55	20
J17	56.51	68.49	346.07	3,000.00	40.21	4,165.31	20
J30	87.7	78.32	345.74	3,000.00	51.72	4,874.30	20
J50	40.12	86.65	345.98	3,000.00	53.42	4,535.92	20
J7	8.13	86.65	345.98	3,000.00	54.44	4,585.39	20
J19	25.62	74.88	345.81	3,000.00	54.55	5,485.37	20
J20	33.11	73.59	345.83	3,000.00	55.56	5,851.82	20
J22	27.39	78.76	345.76	3,000.00	56.26	5,366.77	20
J62	24.61	72.78	345.97	3,000.00	57.34	6,370.02	20
J74	12	72.88	346.2	3,000.00	57.51	6,290.54	20
J18	18.61	72.37	346.03	3,000.00	58.38	6,750.52	20
J26	28.05	75.07	346.24	3,000.00	60.3	6,608.29	20
J76	9.13	75.9	346.16	3,000.00	60.42	6,479.36	20
J58	78.68	82.22	345.76	3,000.00	60.83	5,786.88	20
J54	70.14	87.52	345.98	3,000.00	61.34	5,309.75	20
J25	16.42	75.88	346.12	3,000.00	62.9	7,272.53	20
J24	25.03	76.74	346.1	3,000.00	63.06	7,098.49	20
J60	29.73	81.81	345.8	3,000.00	63.33	6,273.48	20
J82	90.63	87.09	345.99	3,000.00	63.36	5,636.31	20
J78	3.14	78.48	346.12	3,000.00	63.41	6,790.63	20
J66	30.08	76.7	346	3,000.00	63.5	7,369.04	20
J21	30.52	83.11	345.8	3,000.00	63.72	6,144.96	20
J56	63.37	82.25	345.83	3,000.00	64.29	6,433.66	20
J64	14.81	76.75	346.12	3,000.00	65.61	8,233.76	20
J23	87.67	80.16	346.01	3,000.00	65.91	7,276.91	20
J72	27	76.01	346.41	3,000.00	66.02	8,599.31	20
J80	28.12	78.93	346.15	3,000.00	67.18	8,042.12	20
J70	23.6	76.89	346.46	3,000.00	69.02	10,257.61	20

Phases 1 and 2 Nonpotable Fire Flow Demand Junctions

ID	Static Demand (gpm)	Static Pressure (psi)	Static Head (ft)	Fire-Flow Demand (gpm)	Residual Pressure (psi)	Available Flow at Hydrant (gpm)	Available Flow Pressure (psi)
J6	36.01	87.5	342.95	3,000.00	20.86	3,059.06	20
J32	3.97	92.33	343.1	3,000.00	26.65	3,180.17	20
J27	24.73	65.98	345.28	3,000.00	28.98	3,427.91	20
J1	4.45	94.92	343.07	3,000.00	30.45	3,283.96	20
J11	50.44	83.18	342.96	3,000.00	34.68	3,580.68	20
J2	24.74	94.89	342.99	3,000.00	35.26	3,463.59	20
J5	19.91	91.84	342.95	3,000.00	36.17	3,521.40	20
J9	38.49	83.62	342.99	3,000.00	36.49	3,647.39	20
J10	82.33	84.48	342.97	3,000.00	36.73	3,691.85	20
J3	24.15	92.28	342.97	3,000.00	37.78	3,584.00	20
J40	22.58	87.08	342.96	3,000.00	38.73	3,691.61	20
J17	46.35	68.15	345.28	3,000.00	39.36	4,105.42	20
J46	4.02	91.84	342.96	3,000.00	39.48	3,640.84	20
J44	11.68	91.84	342.96	3,000.00	41.17	3,724.71	20
J42	16.21	89.24	342.96	3,000.00	42.24	3,824.42	20
J48	67.78	84.96	343.07	3,000.00	42.77	3,989.33	20
J50	32.9	85.52	343.36	3,000.00	49.36	4,375.55	20
J7	6.67	85.64	343.66	3,000.00	49.63	4,337.16	20
J30	71.93	77.99	344.99	3,000.00	51.12	4,822.31	20
J19	21.01	74.55	345.04	3,000.00	53.84	5,422.95	20
J20	27.16	73.25	345.06	3,000.00	54.82	5,776.83	20
J74	9.85	72.34	344.96	3,000.00	55.43	6,049.82	20
J22	22.47	78.43	345	3,000.00	55.61	5,314.81	20
J62	20.18	72.44	345.19	3,000.00	56.51	6,272.40	20
J54	57.53	86.51	343.66	3,000.00	56.52	4,990.58	20
J18	15.27	72.03	345.24	3,000.00	57.51	6,637.54	20
J76	9.37	75.32	344.83	3,000.00	58.11	6,213.63	20
J26	23	74.58	345.11	3,000.00	58.44	6,383.34	20
J82	93.03	86.14	343.79	3,000.00	58.72	5,295.69	20
J58	64.53	81.89	345	3,000.00	60.18	5,723.27	20
J78	2.58	77.86	344.68	3,000.00	60.82	6,475.84	20
J25	13.47	75.44	345.11	3,000.00	61.38	7,060.65	20
J24	20.53	76.31	345.11	3,000.00	61.62	6,907.75	20
J66	24.67	76.32	345.13	3,000.00	62.54	7,234.65	20
J60	24.39	81.47	345.03	3,000.00	62.61	6,203.77	20
J21	25.04	82.77	345.03	3,000.00	63	6,078.54	20
J56	51.97	81.91	345.04	3,000.00	63.53	6,351.57	20
J64	12.15	76.4	345.31	3,000.00	64.66	8,075.29	20
J72	22.14	75.64	345.58	3,000.00	64.78	8,372.53	20
J23	71.9	79.78	345.11	3,000.00	64.87	7,134.74	20
J80	23.07	78.47	345.11	3,000.00	65.54	7,780.40	20
J70	19.36	76.56	345.68	3,000.00	68.01	10,003.40	20

Phases 1,2 and 3 Nonpotable Fire Flow Demand Junctions

ID	Static Demand (gpm)	Static Pressure (psi)	Static Head (ft)	Fire-Flow Demand (gpm)	Residual Pressure (psi)	Available Flow at Hydrant (gpm)	Available Flow Pressure (psi)
J27	25.32	65.3	343.71	3,000.00	27.22	3,347.82	20
J11	53.78	81.29	338.61	3,000.00	36.69	3,743.57	20
J9	33.78	81.75	338.67	3,000.00	37.34	3,749.24	20
J10	81.58	82.6	338.63	3,000.00	37.55	3,795.30	20
J17	45.91	67.47	343.71	3,000.00	37.60	4,002.34	20
J16	22.69	88.21	338.59	3,000.00	41.32	3,841.36	20
J12	31.91	86.48	338.60	3,000.00	41.43	3,892.85	20
J14	24.45	87.35	338.59	3,000.00	41.65	3,880.53	20
J40	45.62	85.19	338.61	3,000.00	41.68	3,950.57	20
J48	60.9	83.1	338.79	3,000.00	42.06	4,037.51	20
J2	28.63	92.98	338.59	3,000.00	43.01	3,846.98	20
J34	41.62	87.78	338.59	3,000.00	43.08	3,972.15	20
J13	31.52	88.65	338.59	3,000.00	43.31	3,955.21	20
J3	24.68	90.38	338.60	3,000.00	43.65	3,929.45	20
J6	28.42	85.75	338.91	3,000.00	43.85	4,046.76	20
J28	32.58	90.38	338.59	3,000.00	44.73	4,000.67	20
J8	10.14	82.83	339.17	3,000.00	44.85	4,182.99	20
J36	19.08	88.65	338.59	3,000.00	44.88	4,040.67	20
J42	24.39	87.36	338.62	3,000.00	44.95	4,083.10	20
J38	26.58	89.51	338.59	3,000.00	44.99	4,032.57	20
J15	24.99	90.81	338.59	3,000.00	45.05	4,001.37	20
J32	15.12	90.38	338.59	3,000.00	45.99	4,059.88	20
J46	14.08	89.95	338.60	3,000.00	46.15	4,080.46	20
J4	33.7	92.98	338.59	3,000.00	46.38	4,037.68	20
J50	30.53	83.7	339.17	3,000.00	46.44	4,299.11	20
J7	5.98	83.79	339.38	3,000.00	46.80	4,289.12	20
J52	20.67	84.57	339.17	3,000.00	46.85	4,287.55	20
J5	19.71	90	338.71	3,000.00	47.15	4,144.28	20
J44	13.01	89.97	338.63	3,000.00	47.46	4,163.93	20
J1	16.59	92.98	338.59	3,000.00	47.58	4,092.99	20
J30	78.44	77.27	343.34	3,000.00	49.27	4,712.20	20
J54	57.54	84.73	339.55	3,000.00	50.55	4,623.49	20
J19	21.11	73.84	343.40	3,000.00	52.04	5,271.98	20
J74	9.75	71.39	342.77	3,000.00	52.58	5,746.04	20
J20	26.9	72.54	343.42	3,000.00	53.02	5,604.78	20
J82	95.23	84.47	339.95	3,000.00	53.22	4,924.60	20
J22	22.62	77.71	343.35	3,000.00	53.79	5,177.85	20
J62	20.06	71.75	343.59	3,000.00	54.73	6,074.23	20
J76	9.59	74.3	342.47	3,000.00	55.03	5,885.84	20
J18	15.14	71.34	343.65	3,000.00	55.74	6,417.90	20
J26	22.67	73.7	343.09	3,000.00	55.81	6,094.29	20
J78	2.55	76.75	342.14	3,000.00	57.47	6,102.10	20
J58	64.22	81.18	343.35	3,000.00	58.36	5,578.29	20
J25	13.34	74.64	343.26	3,000.00	59.05	6,762.90	20
J24	20.35	75.51	343.27	3,000.00	59.35	6,634.43	20
J66	24.15	75.59	343.44	3,000.00	60.68	6,990.94	20
J60	24.24	80.76	343.37	3,000.00	60.80	6,035.31	20
J21	24.78	82.06	343.38	3,000.00	61.19	5,920.02	20
J56	51.25	81.19	343.38	3,000.00	61.71	6,178.30	20
J72	21.96	74.98	344.04	3,000.00	62.83	8,049.90	20
J64	11.87	75.72	343.75	3,000.00	62.91	7,794.25	20
J23	71.28	79.03	343.38	3,000.00	62.94	6,906.01	20
J80	22.84	77.67	343.25	3,000.00	63.12	7,431.25	20
J70	19.17	75.94	344.25	3,000.00	66.4	9,628.19	20

APPENDIX C

Water Supply Alternatives for Consideration in the Environmental Impact Report

VVH CONSULTING ENGINEERS

AECOM

TECHNICAL MEMORANDUM

TO: Matthew Gerken, AICP, AECOM **DATE**: October 24, 2017

CC: Jeff Goldman, AICP, AECOM
Matt Machado, PE, Stanislaus County Public Works
Keith Boggs, Stanislaus County
Alex Bargmeyer, PE, E-PUR
Kevin Berryhill, PE, Provost & Pritchard

PREPARED BY: John M. Lambie, PG, PE, E-PUR
Dena Traina, PE, Provost & Pritchard **PROJ. NO.** 0624-001-02

SUBJECT: CROWS LANDING INDUSTRIAL BUSINESS PARK WATER SUPPLY ALTERNATIVES FOR CONSIDERATION IN THE ENVIRONMENTAL IMPACT REPORT

INTRODUCTION

Stanislaus County's planned Crows Landing Industrial Business Park (CLIBP) requires a water supply for both potable and non-potable water demands. The purpose of this document is to describe for AECOM updated concepts for water supply that consider the impacts and implications of California Senate Bill 1263. Under SB 1263, any new drinking water system seeking a permit from the State Water Resources Control Board's Division of Drinking Water (DDW) must conduct a meaningful dialogue with all existing systems within three miles of any portion of the respective water service areas to evaluate the feasibility of consolidation, annexation, or extension of water services. The CLIBP is within three miles of both the City of Patterson (Patterson) and Crows Landing Community Services District (the CSD) water systems. Preliminary discussions have been held by Stanislaus County (the County) with both systems' engineering and administrative staff to assess viable alternatives to extend their respective service areas to include the CLIBP.

An initial meeting between DDW and Stanislaus County Public Works on September 26, 2017 identified that if Stanislaus County applies for a drinking water supply permit that SB 1263 will require consideration of operating CLIBP's water supply under one or both existing system permits. DDW also indicated that for a new permit for the CLIBP they would impose both primary and secondary drinking water standards rather strictly. The pending Public Draft Environmental Impact Report (DEIR) will want to consider each of three preliminary alternatives that would comply with SB 1263.

The three water supply alternatives to be considered are:

Alternative A) extension of the Crows Landing Community Services District service area to the CLIBP to cooperatively supply water and system improvements under the existing drinking water supply permit, Alternative B) the County performs all the steps necessary to obtain a new permit to provide drinking water to the CLIBP including the required evaluations with nearby systems, and

Alternative C) the City of Patterson's water service area is extended to include the CLIBP under its existing drinking water supply permit.

Since a preferred alternative has not yet been identified, the DEIR will want to consider the three alternatives identified and described herein co-equally. By doing so, when the Notice of Determination is filed, the EIR will have appropriately addressed the selected alternative. This memo describes the initial development of each Alternative and identifies their material features for consideration in the DEIR.

BACKGROUND ELEMENTS TO USE OF GROUNDWATER IN SUPPLY FOR THE CLIBP

As for the water supply alternatives, earlier work in 2015 and 2016 to support the pending DEIR assessed that both potable and non-potable water supply for the CLIBP would need to come from a groundwater source since surface water is not reliable nor available in the region.[1] That earlier water supply assessment still holds.

The earlier work for the pending DEIR evaluated the needed flow rates and yearly quantities of potable water supply from water producing zones beneath a region wide thick clay layer, the Corcoran Clay. The same type of evaluation was done for non-potable water supply to the CLIBP and it was felt that water producing zones above the Corcoran Clay might be best suited to those needs due to water quality and reliability concerns (the shallow aquifer zones at the south end of the airstrip have been known to dewater in dry years). This strategy of supplies was assessed against Stanislaus County's Well Ordnance for sustainability and against CEQA Guidelines. [2] E-PUR's Technical Memorandum (TM) of May 17, 2017 documents the field findings that a likely sufficient quantity and rate of groundwater production is available at the north end of the airstrip beneath the Corcoran Clay but with concentrations of sulfate that would necessitate either blending or treatment in order to produce potable water. Thus, the configuration and conceptual engineering design of potable water supply to the CLIBP from groundwater zones beneath the CLIBP deserve to be revisited to limit the higher costs associated with treating groundwater for potable supply. Non-potable supply can be reconsidered at this same time. This memorandum is intended to outline the changes in concept for water supply under consideration for each of Alternatives A, B and C.

There are several common considerations for each of the three alternatives as well as overall project water supply provisions. First and foremost, our understanding is that the estimated annual water demand for potable and non-potable supplies for the CLIBP have not changed neither by planned buildout phase nor in total. Second, each water supply alternative must develop the same new supply capacity since neither Alternative A or C anticipate being supplied a net quantity of raw or finished water to meet CLIBP needs from outside the CLIBP by the CSD or Patterson, respectively. As a result of these

[1] VVH, 2015, "Crows Landing Industrial Business Park, Water Supply (Potable and Non-Potable) Infrastructure and Facilities Study, February 27, 2015

[2] JJA, 2016, "Groundwater Resources Impact Assessment, Crows Landing Industrial Park, Stanislaus County, California," Draft August 19.

considerations the projected annualized rates of groundwater production by phase have not been changed from the earlier estimates.

The vertical well-screen intervals for the supply of potable and non-potable water have changed as have the number and the lateral locations of wells. Each of the three alternatives envisions using two or more wells in each of the phases. Based on E-PUR's earlier TM analysis of the aquifer zones below the Corcoran Clay and information gleaned from both the field and the literature regarding the aquifer units above the Corcoran Clay, each well location can be anticipated to be capable of producing 1,000 gallons per minute or more. Potable and non-potable water will be derived from the same wells. The groundwater pumping exclusion zone associated with the former NASA Crows Landing Flight Facility operations has been taken into consideration and the lateral locations of wells set outside that area as depicted on Figure 5 of E-PUR's TM regarding the CLIBP groundwater supply field evaluations dated May 17, 2017.

Non-potable water may or may not be split out after water is piped to a Water Plant at the southeast corner of the CLIBP at the juncture of Fink and Bell Roads. This Water Plant is common to all three alternatives (see Figures A2 to A4 for example). A split of non-potable from potable water supplies would occur if either water treatment is required for potable water or there is a desire or need by the County for piping facilities to accept non-potable water from other prospective sources (e.g., use of highly treated reclaimed water). As a result, each alternative demonstrates a potable and non-potable water piping system. We note here that the location and phasing of the overall piping of potable and non-potable water remains largely unchanged from the earlier work on water supply assessments for the CLIBP (VVH, 2015). We note that some modifications have been made to the conceptual location of potable and non-potable water tanks shown on the figures; however, in concept the size of the needed storage tanks by capacity and purpose remains the same from those same earlier evaluations.

The addition of two or more wells to each of the two initial buildout phases addresses several issues around the security and surety of water supply for the CLIBP. Having at least two wells in Phase 1 provides added reliability should a well fail or should water quality vary suddenly in a well. Then for each phase and the project overall, having two or more wells will enable both the design and the ultimate operation of a supply wellfield with more flexibility during operations to minimize or better control hydraulic drawdown. Utilizing more wells also has the added benefits of: reliability in supply should the shallow aquifer unit dewater in certain areas, decreased likelihood of drawing in water of lower quality from adjoining areas most notably from adjoining agricultural areas, and providing added flexibility to control aquifer drawdown if project related subsidence effects are believed to be occurring.

ALTERNATIVE A – EXTENSION OF CROWS LANDING COMMUNITY SERVICES DISTRICT SERVICE AREA TO CLIBP

Under Alternative A the County would combine its needs for a water supply at the CLIBP with the CSD's needs around water service to conform with changes to state laws on drinking water supply. There are two principal goals for combining the needs of the CSD and CLIBP to one water system:

1) produce the best quality water possible, and

2) produce that water at the lowest possible cost, both administratively and technically.

Based on water quality data collected to date within the CSD's water system and from groundwater test wells within the CLIBP area it appears that bringing together and blending the water sourced from each area can produce good quality drinking water. Blending the two water sources may eliminate the potential need for treatment. This large potential benefit results from the offsetting concentrations of chemicals of concern. For example, hexavalent chromium, Cr(VI), has been detected above 10 parts per billion (ppb) in the Crows Landing CSD water system at both of their groundwater supply wells. The CSD currently has a planning grant from the DDW to address Cr(VI). The concentrations of Cr(VI) detected in shallow and deep groundwater in the CLIBP area range from undetectable to a high of 6.6 ppb. Conversely, concentrations of sulfate in groundwater beneath the CLIBP have been detected above the "recommended" secondary drinking water standard of 250 parts per million (ppm) and sulfate concentrations in both groundwater wells at the CSD are below that lower threshold. Thus, a blending of water sources can lower concentrations of each of these troublesome water quality constituents and may eliminate the need for treatment. Importantly, groundwater sources within each area have concentrations of total dissolved solids (TDS) and nitrate below drinking water standards. As a result, blending the two water sources can produce a suitable supply of water for both quantity and quality with little to no treatment.

Administratively there are many state requirements for small water systems that can be more efficiently met by a consolidated single water system. With two independent systems, the CLIBP and Crows Landing CSD, would each need to have its own system operations and administration. With a single system arising from simple extension of the CSD's current service area there would be efficiency in operations and administration from the combined operation. Meters are now required for all service connections to a drinking water supply system. A consolidated system could readily obtain state grant funding for such meters and enable sufficient capital planning for these meters to be outfitted to existing customers of the Crows Landing CSD. The water system could also utilize a tiered rate structure for service connections to ease affordability for residential customers by charging a higher rate to industrial customers. A consolidated system could also provide reserve capital planning for system maintenance to avoid service disruptions.

The broad framework of a combined water supply system would likely consist of the following high-level elements:

- An interconnection of the current Crows Landing CSD area to the CLIBP area by way of a service corridor along Fink Road to enlarge the service area for the CSD water system shown on Figure A1.
- A raw water supply pipeline of 6-inches or more in diameter placed in a trench excavation 2-feet wide by 5-feet in depth along the north side of Fink Road as depicted in concept on Figure A2.
- Two or more source wells within the current Crows Landing CSD Service Area, such as the current Wells 4 and 5 if suitable (not depicted).
- Approximately four groundwater supply wells within the CLIBP to produce raw water for blending and to meet its planned water demands/needs in a phased manner consistent with the CLIBP development plan as depicted in concept on Figure A2;
- Source (raw) water blending within the CLIBP footprint in the southeast corner as depicted on Figures A2 and A3.

- Water supply piping to transmit potable water for both emergency and non-emergency water demands installed along the north side of Fink Road to Crows Landing is depicted in concept on Figure A3. This supply line would be a minimum of 12-inch-diameter pipe placed in a separate trench offset at least 10-feet from the raw water pipe depicted in Figure A2 in accordance with California drinking water system standards. The supply piping trench would likely be excavated 3-feet wide by 5-feet deep.
- Water supply piping to transmit raw water along the west side of Bell Road via an 18-inch diameter pipe installed in a trench excavated 3.5-feet wide by 5-feet deep (see Figure A2).
- Water supply piping for non-potable water depicted in concept on Figure A4 would be constructed in excavation trenches a minimum of 2 -feet wide by 5 -feet deep.
- Water supply piping for potable water depicted in concept on Figure A3 would be placed in excavation trenches a minimum of 2-feet wide by 5-feet deep offset at least 10-feet from raw and non-potable water pipes along parallel routes, in accordance with California drinking water system standards .
- Potable water supply piping, water storage tanks to the west and northwest toward the CLIBP developed areas in general accord with the earlier work by VVH and AECOM[3] regarding storage capacity needs for potable water and the layout of piping by development phases (depicted in Figure A3).
- The consolidated system for Crows Landing CSD and CLIBP may be a combined water system for both potable and non-potable water needs depending upon treatment needs but for the DEIR the non-potable needs should be evaluated against the depiction on Figure A4 that generally follows the layout and phasing for piping and storage done previously in AECOM/VVH, 2016.
- Specifics of water storage, pipe sizes, booster pumps, blending requirements within CLIBP as well as any modifications to distribution system piping and service connections may vary somewhat based on future detailed engineering design.
- Further specification of water storage, pipe sizes, booster pumps, and other details on the supply system within the CSD's current service area as wells as any modifications to distribution system piping and service connections may vary somewhat based on future detailed engineering design but these types of modifications appear to be outside the scope of the DEIR analysis needed for the CLIBP.

Providing for the water supply needs for both areas under the existing Crows Landing CSD permit to operate a drinking water system alleviates the need for some of the extensive evaluations that will be required of each area if they operate separate systems. The CLIBP would not need to obtain a new water system operating permit under SB 1263, and the Crows Landing CSD may not need to provide hexavalent chromium treatment to meet anticipated drinking water standards. LAFCO considerations would need to be met in extending the service area of Crows Landing CSD to encompass the CLIBP for water service.

[3] VVH, 2015 and AECOM/VVH, 2016, "Crows Landing Industrial Business Park, Water Supply (Potable and Non-Potable) Infrastructure and Facilities Study, February 27, 2015 and Updated September 27, 2016.

ALTERNATIVE B DEVELOP A STAND ALONE CLIBP WATER SUPPLY SYSTEM

Under Alternative B the County would provide a water supply at the CLIBP. Since a portion of the water supply will be for drinking water SB 1263 will apply directly and must be met. Those requirements are generally addressed in the development of this Alternative B general elements.

Based on water quality data collected to date from groundwater wells at and adjacent the CLIBP it appears possible that potable quality water can be produced without treatment. However, some treatment for sulfate may be required by DDW unless waiver of the "Recommended" secondary drinking water standards for both sulfate and total dissolved solids (TDS) can be obtained during permitting by DDW.[4] Concentrations of TDS for all wells in the region are above the "Recommended" secondary drinking water standard of 500 parts per million (ppm), and are at or just below the "Upper" secondary drinking water standard of 1,000 ppm. Similarly concentrations of sulfate in groundwater beneath the CLIBP in water producing zones range between 320 ppm and 600 ppm with an anticipated concentration of sulfate of around 380 ppm from a vertically integrated production well based upon the chemistry of water from an agricultural water supply well that draws from both shallow and deep aquifer units in the area of the planned wellfield. The "Recommended" secondary drinking water standard for sulfate is 250 ppm while the "Upper" secondary drinking water standard is 500 ppm. Hence it is likely that water produced at the CLIBP will be potable in regard to these two constituents. Regarding two other key constituents, Cr (VI) and nitrate, groundwater quality samples from both shallow and deep units beneath the CLIBP are below the primary drinking water standards.

Administratively there are many state requirements for permitting a new water system under SB 1263 since the CLIBP service area is within three miles of two existing water systems, Crows Landing CSD and City of Patterson. The primary mechanism for this administration is the requirement for the County to submit a "preliminary technical report" under the law. The particular contents of such a preliminary technical report under SB 1263 include:

- The name of each public water system within three miles
- A discussion of the feasibility of annexing, connecting or otherwise supplying water to the CLIBP via those existing systems
- Documentation of the consultation with adjacent public water systems about supplying water to the CLIBP
- Documentation of any information provided by adjacent water systems on the feasibility of annexing, connecting or otherwise supplying water to the CLIBP
- A discussion of all actions taken by the County to secure a supply from an existing water system
- A comparison of costs between a new system and cost of joining an existing system
- An analysis of supply resilience a 20-year projection inclusive of normal, single dry, or multiple dry water years
- Any information provided by the local agency formation commission (LAFCO) for consideration.

[4] The drinking water standard typically applied for secondary standards is or has been the "Upper" concentrations

Then there are the added requirements of establishing water quality standards that can be met and ways in which they are proposed to be met.

The broad framework of a CLIBP stand-alone water supply system would likely consist of the following high-level elements:

- A CLIBP service area that takes into account the two nearby existing systems depicted on Figure B1.
- Approximately four groundwater supply wells within the CLIBP to produce raw water as depicted in concept on Figure B2;
- Source (raw) water disinfection (and potentially treatment) at a Water Plant within the CLIBP footprint in the southeast corner as depicted on Figures B2 and B3.
- Water supply piping to transmit raw water along Bell Road via an 18-inch diameter pipe installed in a trench excavated 3-feet wide by 5-feet deep (see Figure B2).
- Water supply piping for non-potable water along Bell Road and Fink Roads in excavation trenches a minimum of 3-feet wide by 5-feet deep depicted in concept on Figure B4.
- Water supply piping for potable water along Bell Road and Fink Roads depicted in concept on Figure B3 installed in excavation trenches a minimum of 2 feet wide by 5 feet deep offset at least 10-feet from raw and non-potable water pipes along parallel routes in accordance with California drinking water system standards.
- Potable water supply piping, water storage tanks to the west and northwest toward the CLIBP developed areas in general accord with the earlier work by VVH and AECOM(AECOM/VVH, 2016) regarding storage capacity needs for potable water and the layout of piping by development phases (depicted in Figure B3).

The CLIBP would need to obtain a new water system operating permit under SB 1263. It should be noted in this description of Alternative B for the CLIBP water supply that it will be necessary for the County to develop a rate structure for its own system and compare it to a rate structure when the service area is combined with the CSD into and extended system or similarly with Patterson into an extended system. This will necessitate a request for information about both the CSD's and Patterson's current rate structure to develop a conceptual rate structure to satisfy those requirements of SB 1263. It is not necessary to develop anything definitive at this time for actual infrastructure, rate structures or staffing. This presentation is more conceptual in nature.

Lastly, LAFCO considerations may be pertinent to the DEIR and should be considered appropriately.

ALTERNATIVE C EXTENSION OF WATER SERVICES TO CLIBP FROM CITY OF PATTERSON

Given the location of Patterson's supply sources and storage the viable alternative to be considered is an extension of services area. The source water for the extension would be produced by groundwater wells within the CLIBP footprint, and an intertie between the current and the extended area for supply redundancy.

Initial discussions with the Patterson on October 3, 2017 revealed that there is inadequate capacity to supply the CLIBP potable water and that extension of water service to CLIBP is not currently covered by Patterson's recently updated Water Master Plan nor the companion updates to Patterson's Capital

Improvement Plan. However, Patterson staff understood the need for the County to explore a prospective combination of potable water supply infrastructure under SB 1263 to ensure that viable alternatives were adequately addressed in the DEIR.

The overall concept for potable supply in Alternative C is for the County to drill and install a series of groundwater potable-water-supply wells at the CLIBP to provide the required capacity and to design and install an interconnecting water supply pipeline between the current Patterson service area and the CLIBP. This proposed intertie would provide desirable supply redundancy. The routing for an interconnecting pipeline was discussed and an appropriate route is to follow Ward Avenue south from Patterson to Marshall Road and then along Marshall Road to the east to where it intersects the northwest corner of the CLIBP. This overall extension of service area with an intertie corridor is depicted in Figure C1.

Based on groundwater quality data collected at CLIBP to date it appears possible that potable quality water can be generated without treatment. Both shallow and deep groundwater beneath the CLIBP will be used to optimize water quality. The broad details of Alternative C to be evaluated in the DEIR include bringing the groundwater sourced at the CLIBP to a local blending facility (and treatment, if necessary) at the juncture of Fink Road and Bell Road in the southeast corner of CLIBP. This location is suited to supplying the earliest phases of CLIBP. The intertie for water from Patterson would be brought to Phase 2 or 3 potable supply piping behind (i.e. after) any local water disinfection (and treatment, if necessary) or in order to not mix raw water at the CLIBP with finished water from Patterson.

Extension of the Patterson Water Service Area to the CLIBP would obviate the need to complete SB 1263 requirements since a new public drinking water system is not being created. The new source wells and configuration would require approval by DDW prior to their being commissioned under Patterson's current permit. Creation of an interconnecting pipeline (i.e., an intertie) between the two areas offers some potential benefits of supply redundancy in the event of service disruptions, maintenance needs, and other operational events within either area. It is assumed for the purposes of presentation herein that the drinking water system staffing and billing would be handled by Patterson. It is also assumed herein that a non-potable water system would be developed from raw water sources at the CLIBP and could be managed by Patterson as well.

The broad framework of a consolidated water supply system with Patterson would likely consist of the following high-level elements:

- An interconnection of the current Patterson area to the CLIBP area by way of a service corridor along Ward Avenue and Marshall Road to enlarge the service area for the water system (see Figure C1);
- Approximately four groundwater supply wells within the CLIBP to produce raw water to meet its planned water demands/needs in a phased manner consistent with the CLIBP development plan (see Figure C2);
- Source (raw) water production and chlorination/treatment within the CLIBP footprint;
- Water supply piping to transmit raw water along the west side of Bell Road via an 18-inch diameter pipe installed in a trench excavated 3.5-feet wide by 5-feet deep (see Figure C2).
- Water supply piping for non-potable water along Bell Road and Fink Roads in excavation trenches a minimum of 3-feet wide by 5-feet deep depicted in concept on Figure C4.

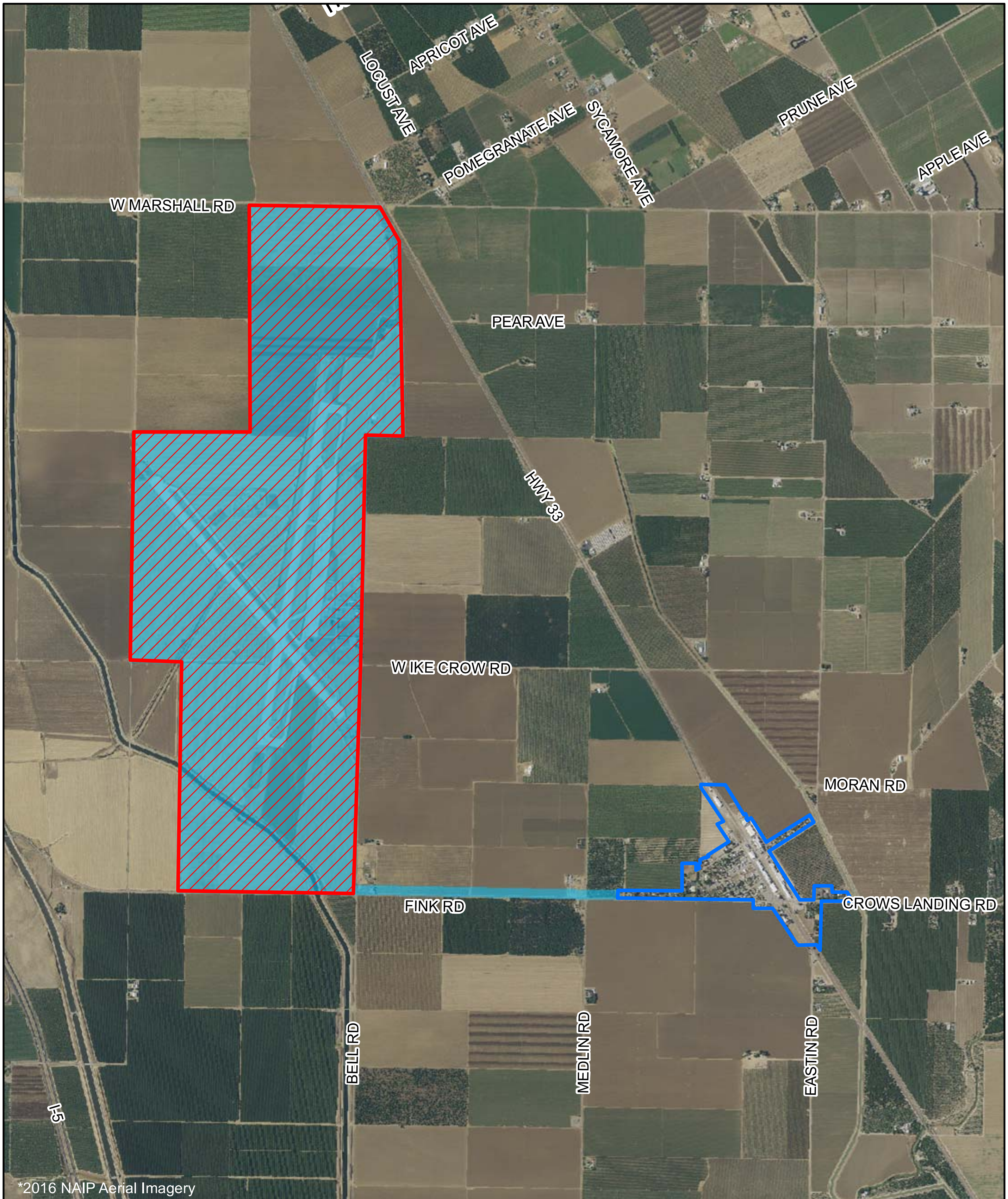
- Water supply piping for potable water depicted in concept on Figure C3 in excavation trenches a minimum of 2 feet wide by 5 feet deep offset at least 10-feet from raw and non-potable water pipes along parallel routes in accordance with California drinking water system standards.
- Water supply piping and infrastructure to transmit drinking water between the CLIBP and Patterson. It is anticipated that this piping will be located within the Marshall and Ward Avenue rights of way and will traverse under the Delta Mendota Canal twice, one on each roadway. The pipelines will need to be directionally drilled at least 25 feet below the Delta Mendota Canal. The remaining part of the alignment will be open cut trenched with a trench section of approximately 5-feet deep by 3.5-feet wide. Current working estimates are that the intertie pipe size is to be between 12-inches and 18-inches in diameter.
- Potable water supply piping within the CLIBP would transmit water from the blending and disinfection/treament facility to the west and northwest into developed areas with pressure supplied by local storage tanks and booster pumps (see Figure C3);
- Water storage and booster pumps at CLIBP in two or three locations within the development will be needed for potable and non-potable water demands;
- Water demand versus supply for the CLIBP fit within the CLIBP's Water Master Plan assessment of water availability, and as a result water sustainability remains the same for this drinking water system alternative, Alternative C; and,
- Specifics of water storage, pipe sizes, booster pumps, blending requirements within CLIBP as well as any modifications to distribution system piping and service connections may vary somewhat based on future detailed engineering design.
- Non-potable water configurations would most likely consist of sources and plumbing all within the CLIBP footprint as depicted in Figure C-4.

The DEIR's Project Specific Plan will describe and discuss the three water supply alternatives at this high level. Details of a prospective Alternative C integration with Patterson as well as evaluations of Alternative B in light of SB 1263 will be done in cooperation with Patterson, outside of the DEIR analysis to be performed by AECOM.

Attachments:
Figures A1 to A4, B1 to B4, and C-1 to C-4 for each of the respective Alternatives A, B, and C

Figure Depictions of a Conceptual Crows Landing CSD Water Supply for the Crows Landing Industrial Business Park

Alternative A



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Crows Landing CSD
Crows Landing Industrial Business Park
Proposed Extension Area to CLCSD

Figure A1
Crows Landing CSD and CLIBP Extension Service Area

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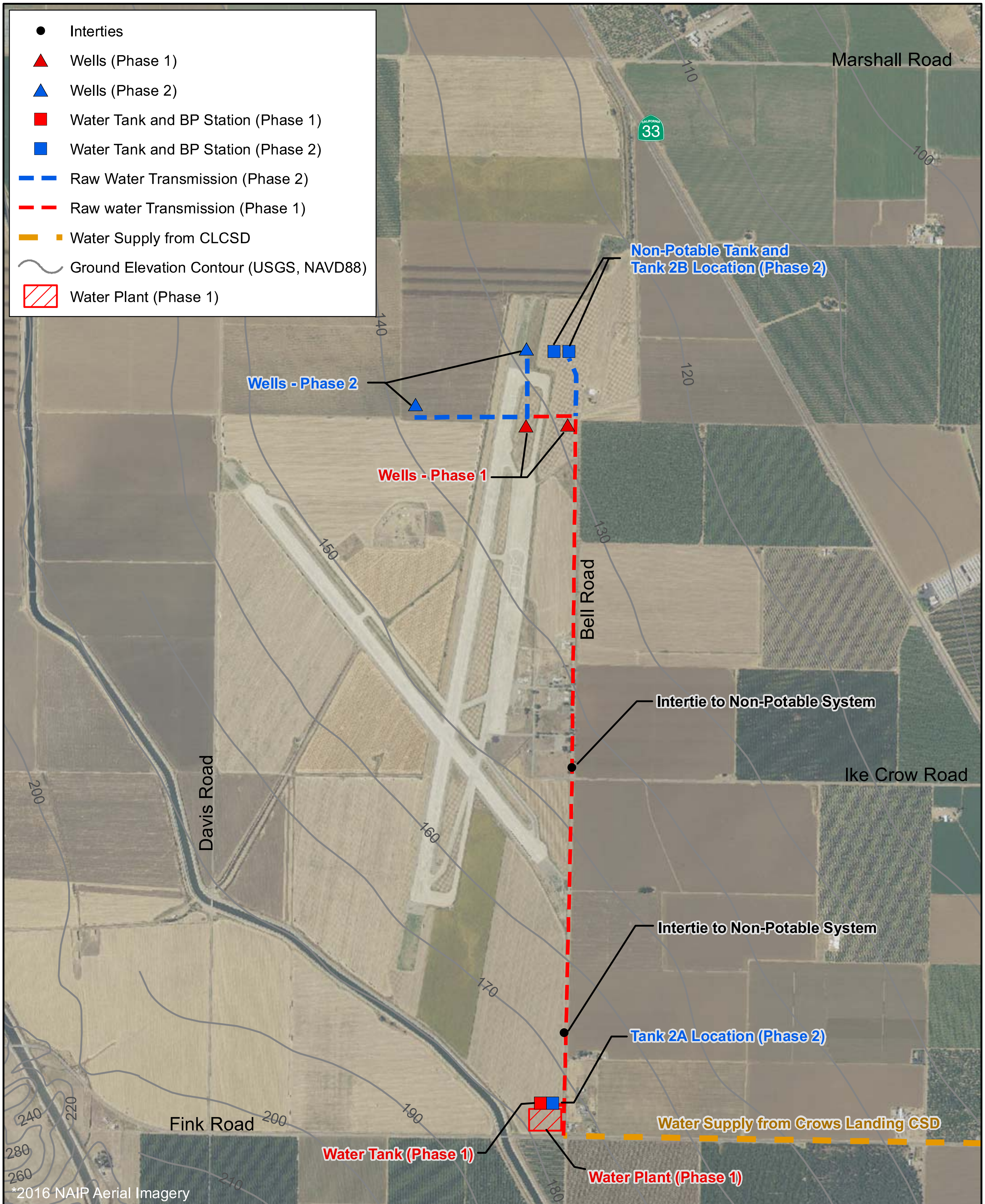

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Figure A2
CLIBP Area Raw Water System for Crows Landing CSD Extension of Service

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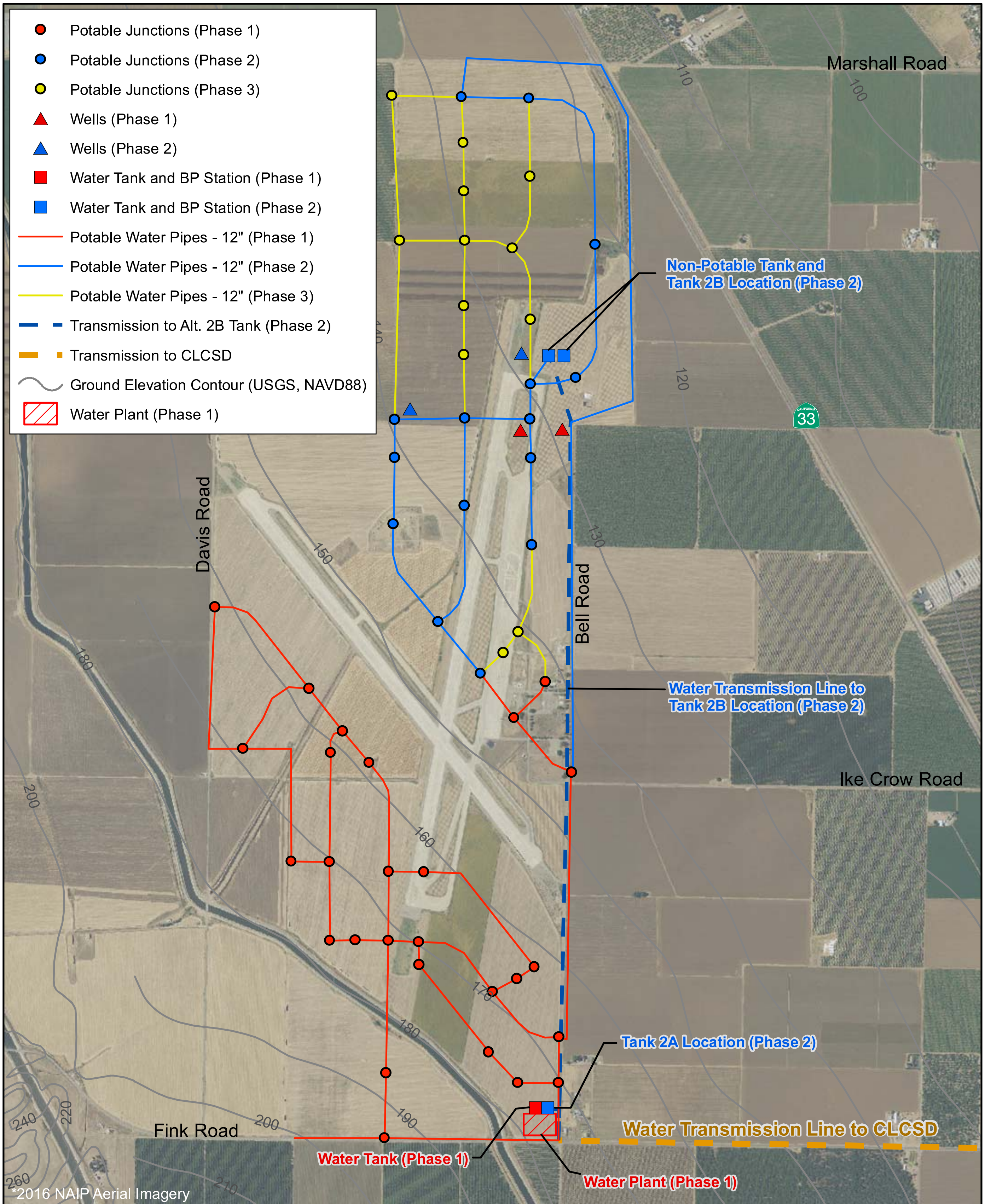

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Figure A3
CLIBP Area Potable Water System for Extension of Crows Landing CSD

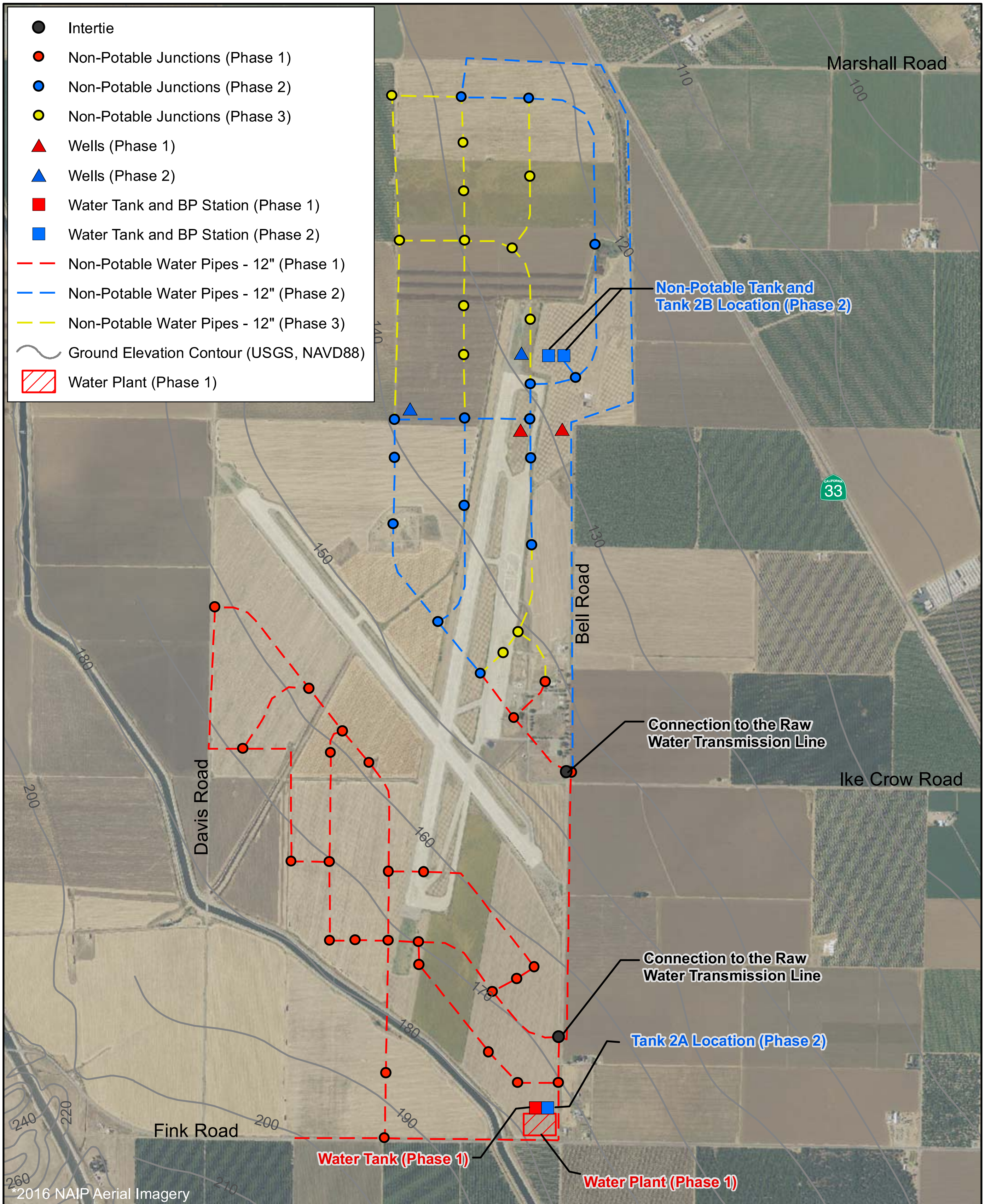

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Figure A4
CLIBP Area Non-Potable Water System for Extension of Crows Landing CSD

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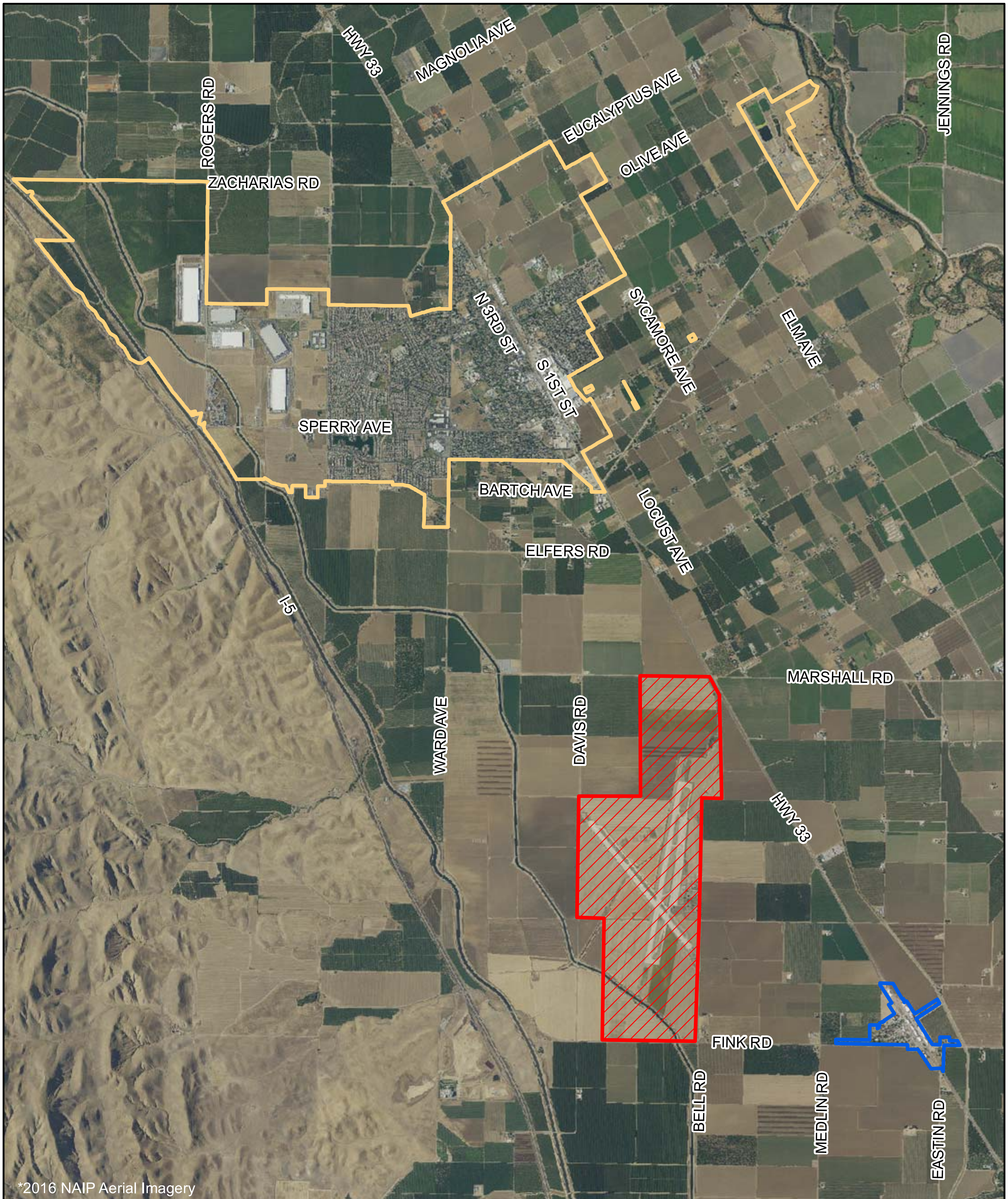
Figure Depictions of a Conceptual Stand Alone Water Supply for the Crows Landing Industrial Business Park

Alternative B



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Crows Landing CSD

Crows Landing Industrial Business Park

City of Patterson

Figure B1
CLIBP Location Map and Regional Public Water Systems

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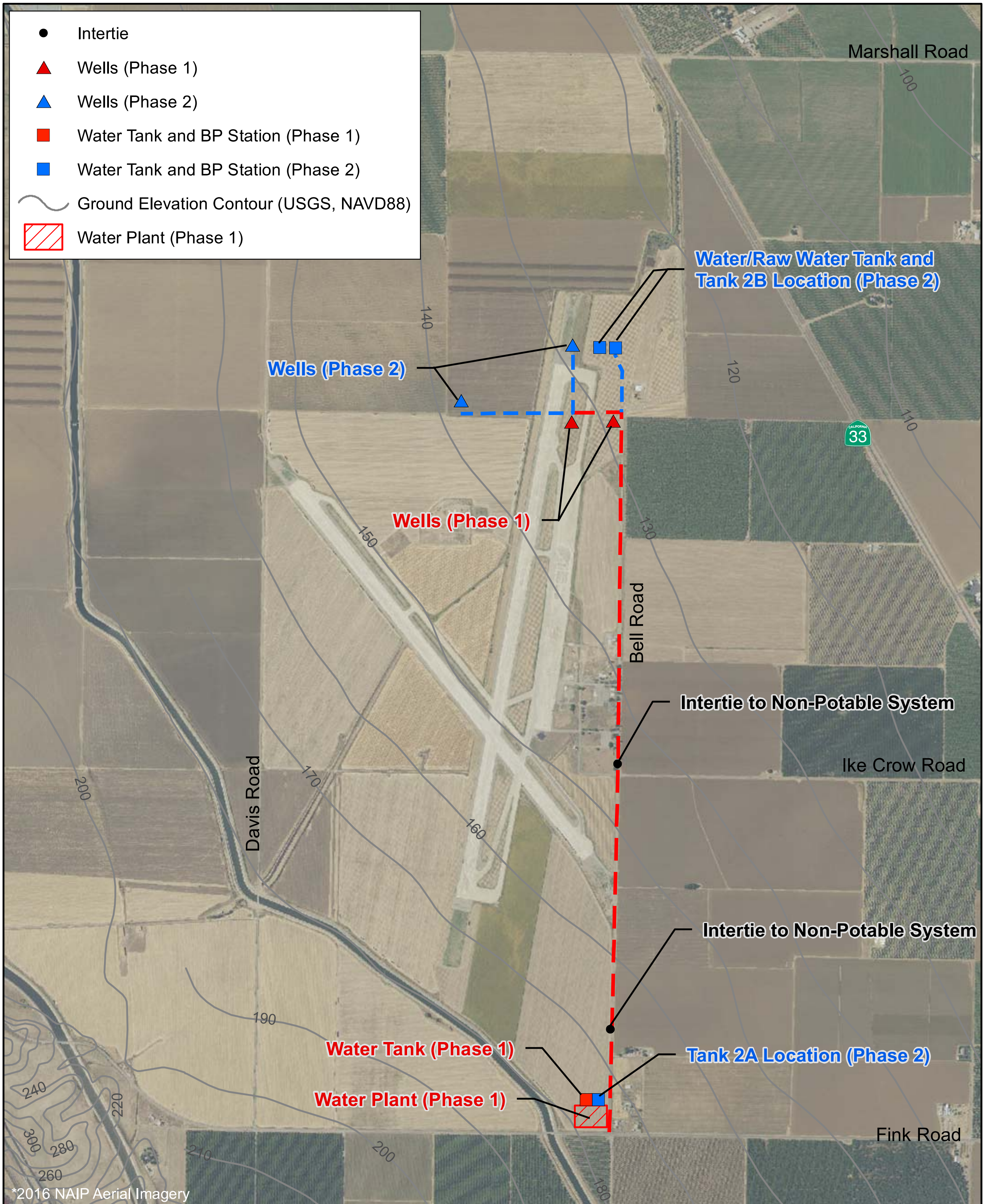

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Figure B2
CLIBP Stand Alone Raw Water System Layout

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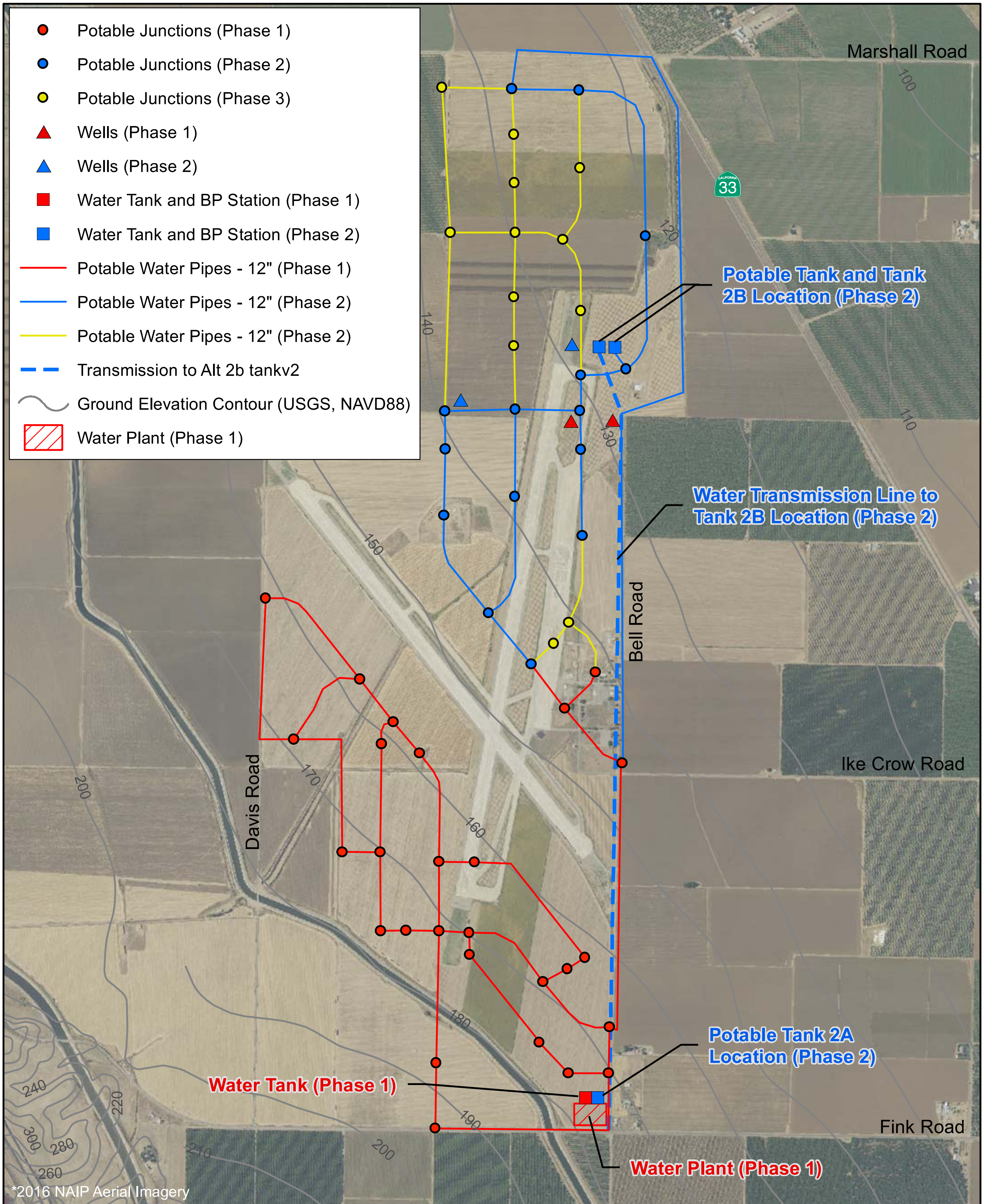

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Figure B3
CLIBP Stand Alone Potable Water System

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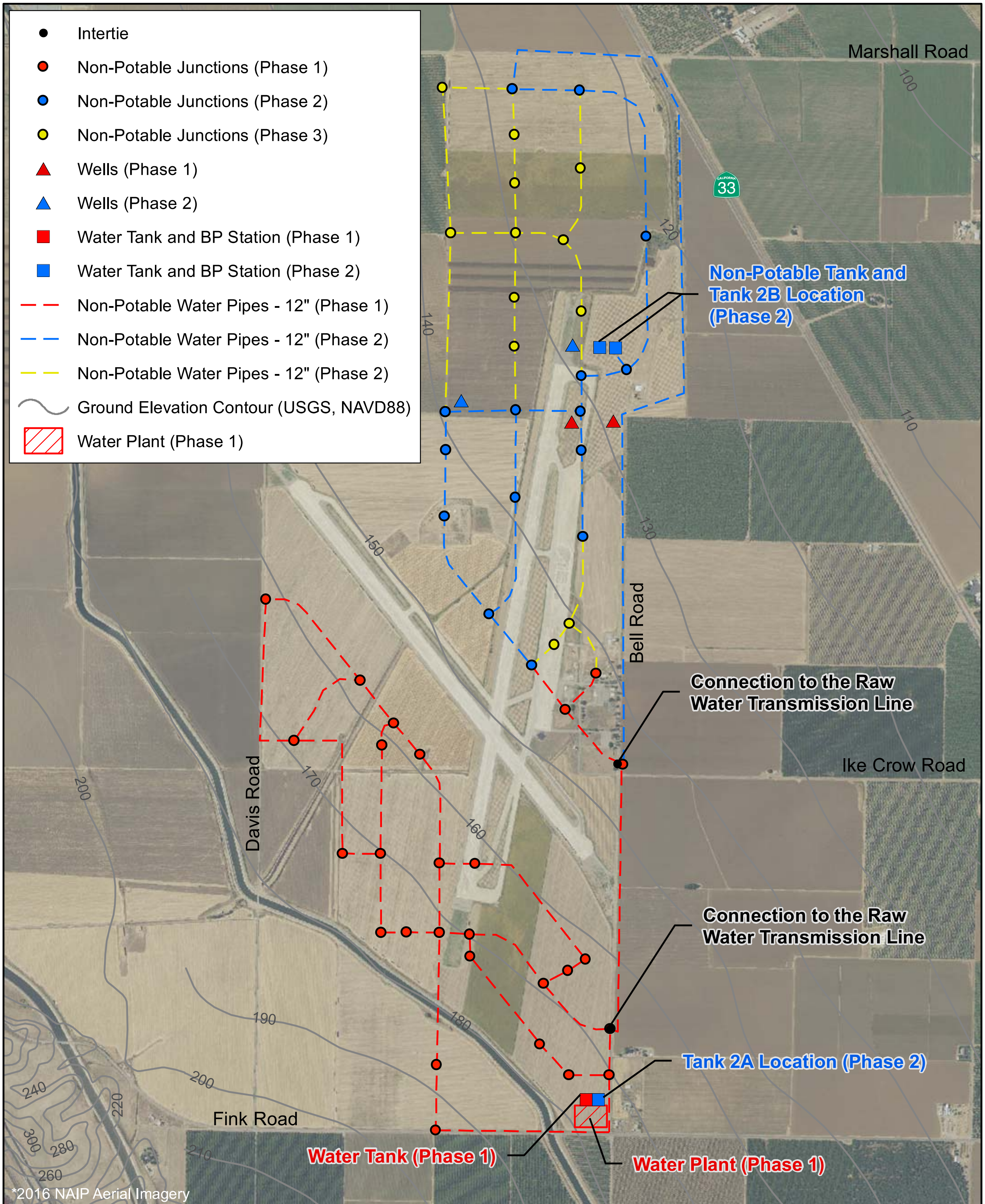

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Figure B4
CLIBP Stand Alone Non-Potable Water System

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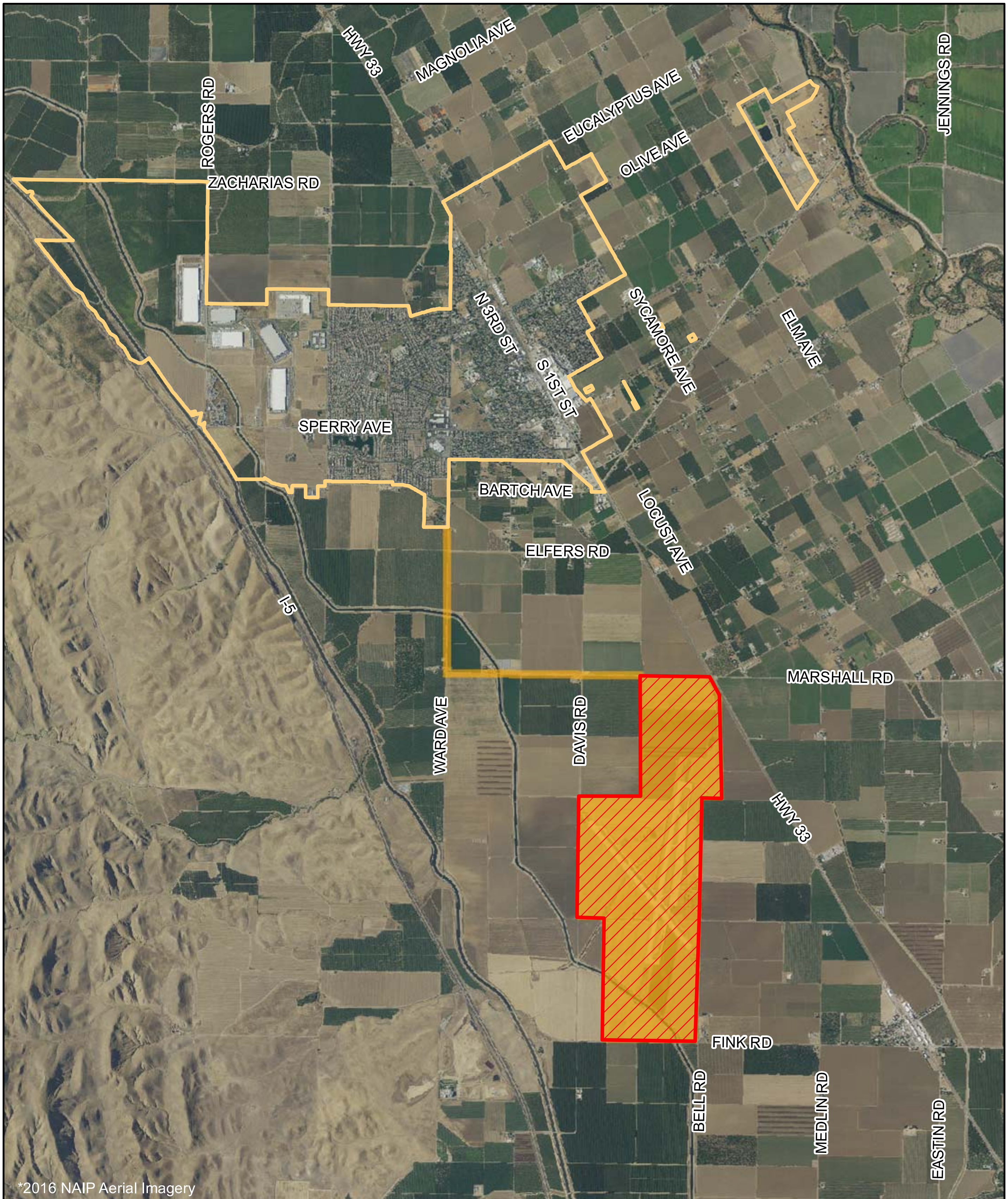
Figure Depictions of a Conceptual City of Patterson Water Supply for the Crows Landing Industrial Business Park

Alternative C



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Service Area Extension

Crows Landing Industrial Business Park

City of Patterson

Figure C1
CLIBP Water System Extension of City of Patterson Water Service Area

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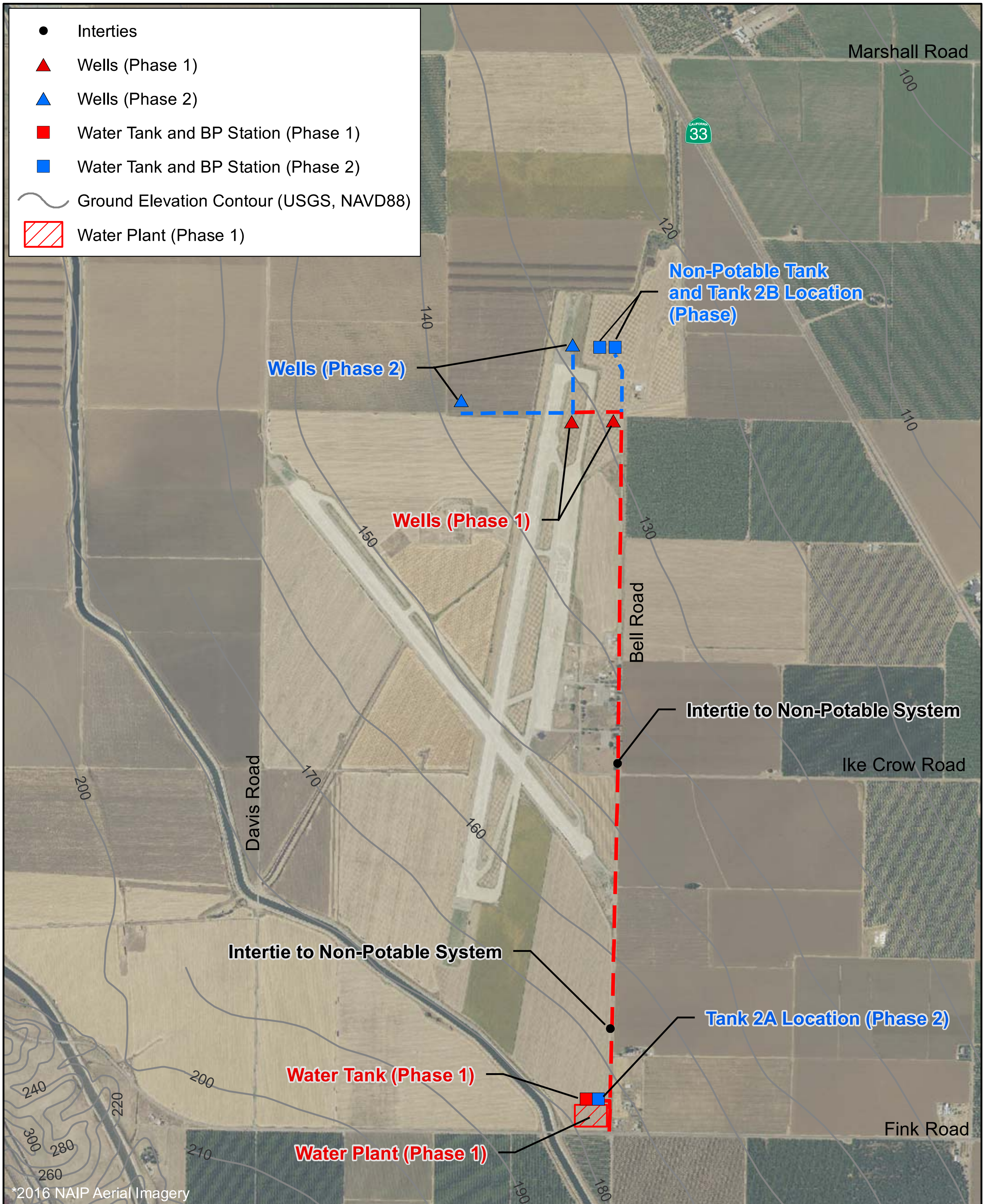

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Figure C2
CLIBP Raw Water System for City of Patterson Extension of Service Area

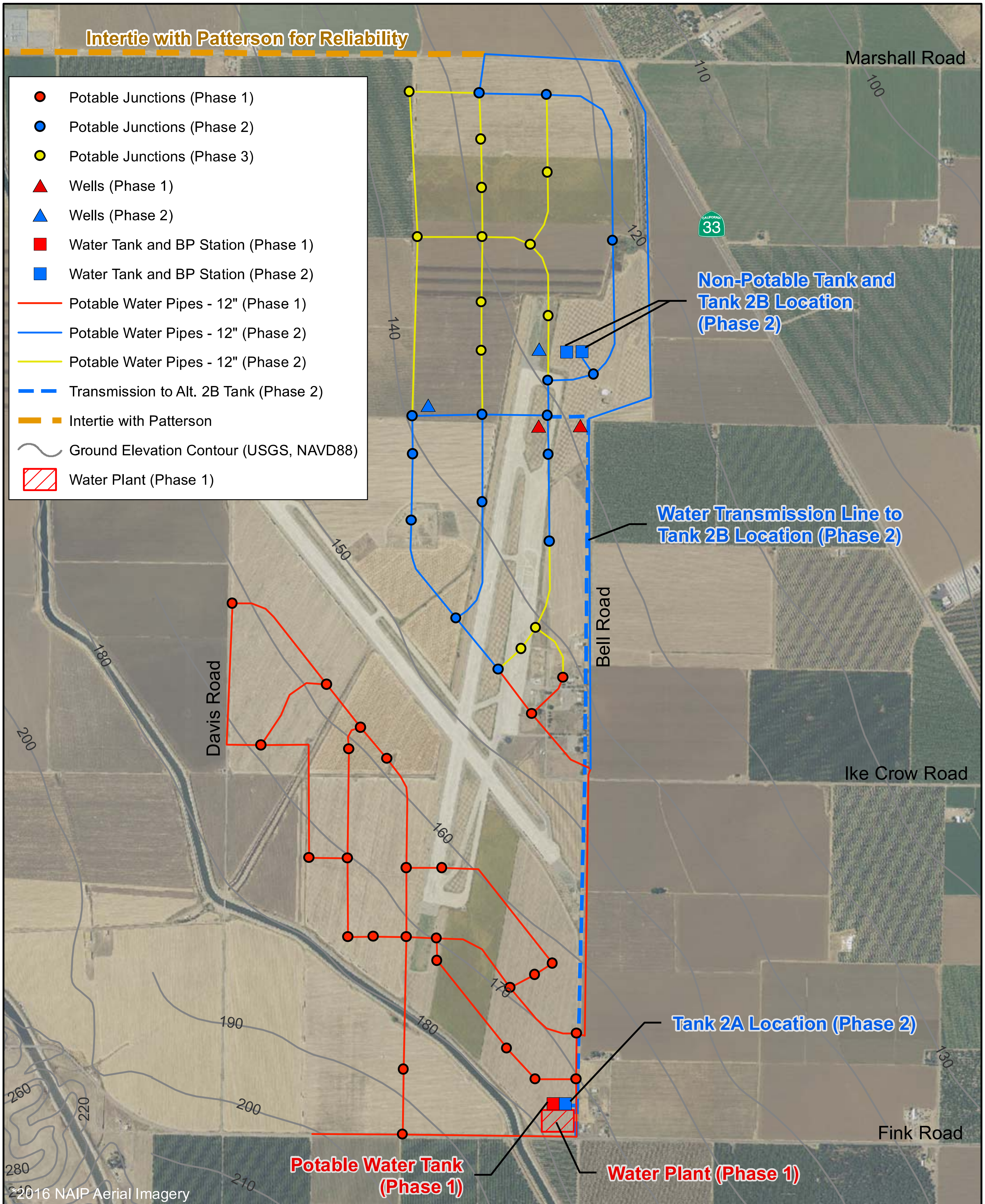

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Figure C3
CLIBP Area Potable Water System for City of Patterson Extension of Service Area

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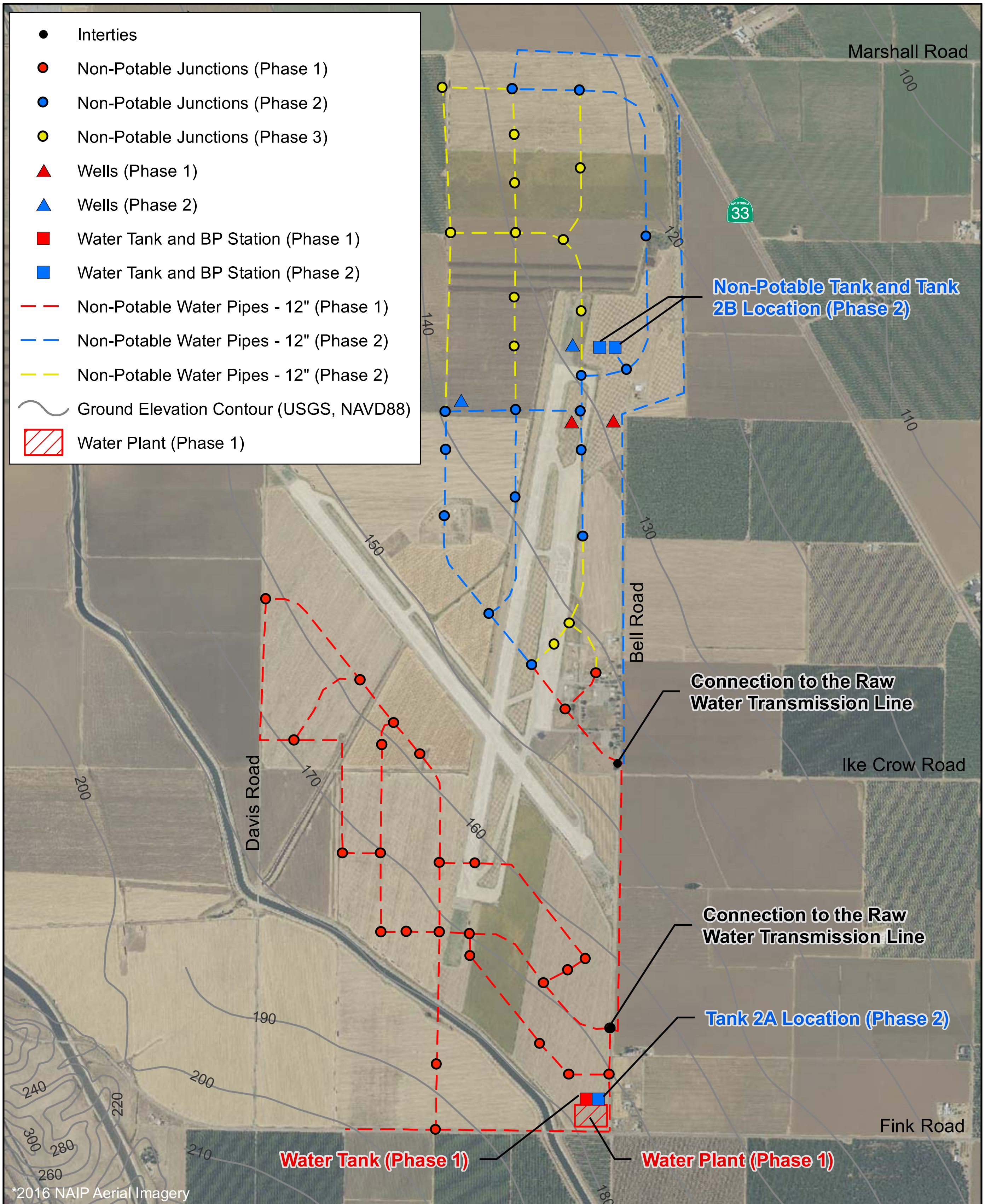

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Figure C4
CLIBP Non-Potable Water System for City of Patterson Extension of Service Area

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Safe Water for All

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