Appendix F1. Flood Risk Management

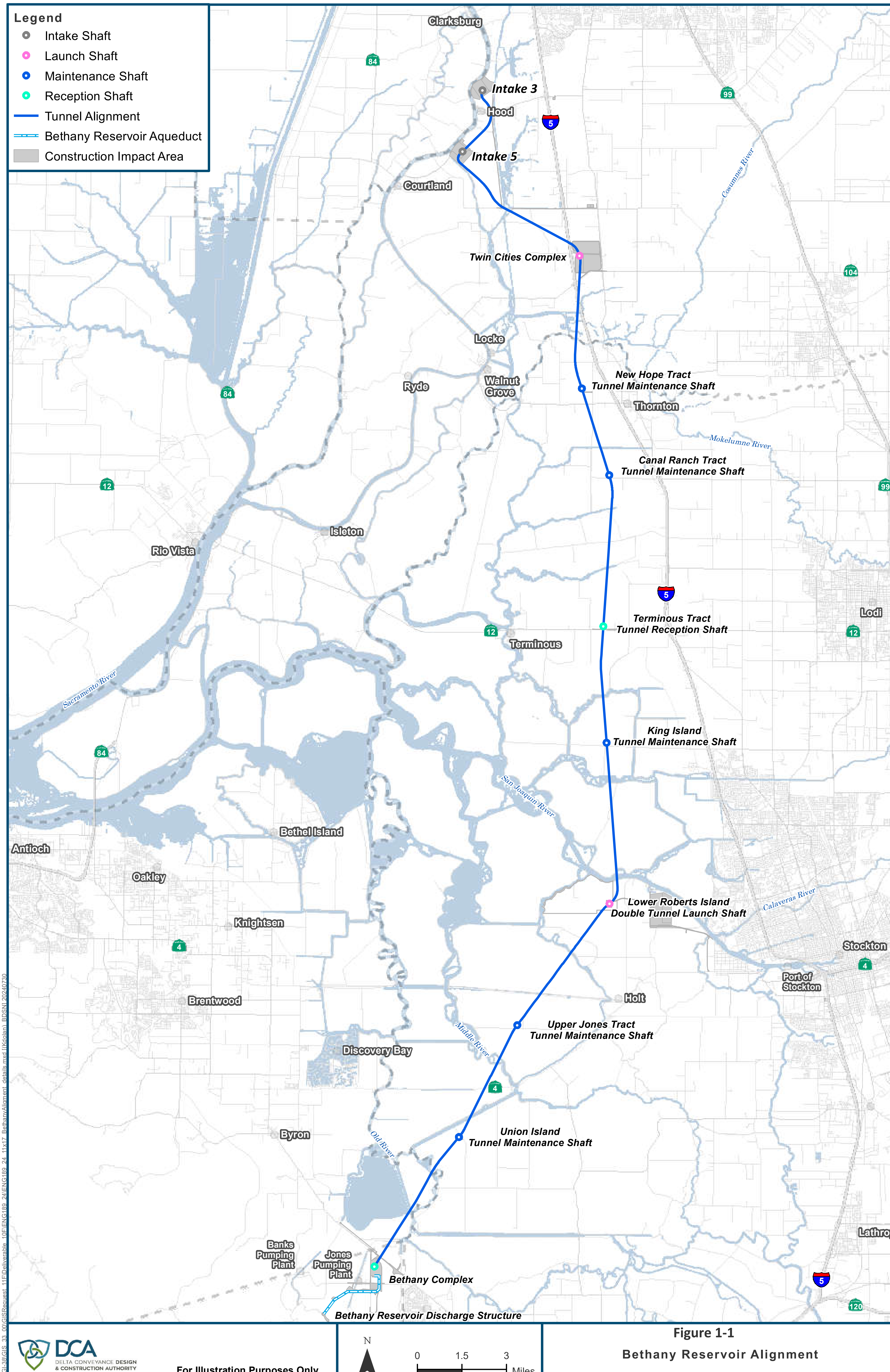
1. Introduction and Purpose

The Bethany Reservoir Alignment a part of the Delta Conveyance Project (Project) would include new intake facilities located along the Sacramento River between the confluences of the American River and Sutter Slough, a tunnel to convey water from the intakes to the southern end of the Delta, and a pumping plant with associated facilities to deliver water to the existing State Water (Figure 1-1).

The tunnel internal diameter would be 36 to meet Project design capacity of 6,000cubic feet per second (cfs). The purpose of this technical memorandum (TM) is to evaluate potential flood risks during construction of the tunnel and methods to protect the workers at large construction sites that would be difficult to evacuate during a flood emergency.

The tunnel would be constructed using multiple Tunnel Boring Machines (TBMs) to excavate and install tunnel lining segments from the launch shafts. The TBMs would be retrieved at the reception shafts that would be located approximately 15 miles from the launch shafts. Maintenance shafts would be placed approximately 4 to 6 miles apart along the tunnel alignments to facilitate access for personnel and equipment to inspect and maintain the TBMs. Excavated material from the tunnel boring, referred to as reusable tunnel material (RTM) would be brought to the surface at the launch shaft sites, processed and reused onsite, and stockpiled. Tunnel launch shaft sites would require several hundred acres for equipment, supplies, offices, parking, and excavated materials handling. Construction activities at each of the launch shafts would continue for 7 to 9 years. RTM would not be removed at the reception and maintenance shafts; and therefore, the maintenance and reception shaft sites would be smaller than the launch sites. The maintenance and reception shaft sites would consist of a fenced enclosure with an elevated earthen work platform, shaft, overhead crane, relatively small stockpiles of soil excavated from the shaft, and supporting facilities and equipment. Construction activities at the maintenance and reception shaft sites would continue for approximately 2 years.

The tops of all tunnel shafts would be constructed to fully contain an internal 200-year water surface elevation (WSEL) originating at the Sacramento River near the intakes, taking into consideration operational fluctuations, increasing storm intensities and the projected effects of climate change and sea level rise for the year 2100 (DWR, 2020). 200-year WSELs on the Sacramento River near the intakes are higher than equivalent return period WSELs within the Delta, so establishing the top of shaft elevations as described above provides long-term protection to Project infrastructure upon completion and to workers within the tunnel during construction since the top of the shaft would be several feet above potential local WSELs.



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DCA DELTA CONVEYANCE DESIGN & CONSTRUCTION AUTHORITY

For Illustration Purposes Only

Figure 1-1
Bethany Reservoir Alignment

Data Source: DCA, DWR

Earthen pads surrounding the tunnel shafts would be constructed to elevations slightly above the local 100-year flood elevation with anticipated sea level rise and climate change hydrology in Year 2040 (DWR, 2020). These shaft pads would provide a working platform for construction of shaft diaphragm walls to minimize potential artesian conditions that may be encountered. Additionally, the shaft pads would also provide a refuge for workers during construction in the event of a levee breach that inundates the surrounding land up to a 100-year WSEL. However, shaft pads could require multiple years to construct and flood risks to workers would remain even after shaft pad construction generally associated with activities conducted at ground level and the ability of workers to get to high ground refuge during an emergency. This TM is intended to further evaluate flood risks and potential mitigation measures at each tunnel shaft site to protect life safety during construction of the Project. The following information is provided:

- An overview of existing levees in Sacramento-San Joaquin Delta.
- Summarize factors contributing to historical flooding in the Delta.
- Evaluate non-structural and structural flood risk management measures to improve flood risk protection and life safety during Project construction.
- Evaluation of flood risks during Project construction and after Project completion at Twin Cities launch shaft site.
- Assessment of existing shallow, overland flooding identified at the Intake locations.

1.1 Background

1.1.1 Sacramento-San Joaquin Delta Levees

The Project facilities would be located within the Sacramento River and San Joaquin River Delta (Delta). The Delta includes over 700 miles of sloughs and waterways with more than 1,100 miles of levees surrounding more than 60 leveed tracts and islands. Elevations of land on many of the leveed tracts and islands are lower than the surface water elevations. Therefore, the Delta levees are more comparable to dams than levees. However, unlike most dams, Delta levees were not built with strict engineering standards. Many of the Delta levees were built by local landowners in the late 1800s and early 1900s without using modern engineering practices (DWR, 2009a).

Approximately 35 percent of the Delta levees are categorized as Project Levees because they are part of the Federal Flood Control Project. The Project Levees were built to standards at the time of construction based on United States Army Corps of Engineers (USACE) guidelines. These levees are maintained by local agencies and periodically inspected by USACE. The remaining 65 percent of the Delta levees are categorized as Non-Project Levees and are constructed and maintained by landowners or local reclamation districts, generally referred to as Levee Maintaining Agencies, or LMAs. Non-Project Levees are generally built to an agricultural standard specific to the Delta (DWR, 1993).

Conditions of the levees vary throughout the Delta; and levee weaknesses and uncertainty occur at various locations. The specific locations of levee weakness are not always easy to identify. Levee weaknesses can take many forms. Among the most significant are inadequate levee geometry (crown elevation and cross-section) and geotechnical integrity, inadequate erosion protection, weak foundations, penetrations such as drainage and irrigation pipes, damage due to animal burrowing, damage to levees due to human activity, such as vehicle traffic or construction activity, and even drying and cracking during extended periods of drought. Ongoing scour, animal burrowing, boat traffic, rodent activities, human actions to improve and degrade levees, subsidence, and deep foundation changes all could affect the integrity of the Delta levee systems over time.

The best available information upon which reasonable estimates of levee integrity can be based include surveys of levee and channel geometry, visual inspections, geotechnical exploration, records of past performance, and computer simulations. As part of the Project, the DCA has performed a Levee Vulnerability Assessment based on the best readily-available data for Delta channel geometry, levee geometry, and island interior elevations and is provided in the Conceptual Engineering Report (CER)Appendix F2 *Levee Vulnerability Assessment*. Comparing existing levee geometric data to established Delta Levee Performance Criteria and flood elevations can help identify levee reaches where the geometry may be deficient and remedial work can reduce the risk of overtopping or structural failure.

1.2 Organization

This TM is organized as follows:

- Introduction and Purpose
- Analysis of Flood Risk in the Delta
- Flood Risk Management Measures
- Evaluation of Delta Flood Risk at Launch, Reception, and Maintenance Shafts
- Site Specific Recommendations
- Observations and Conclusions
- References
- Attachment 1 – Levee Branches in the North Delta, February 1986
- Attachment 2 – Flood Inundation Analysis
- Attachment 3 – Twin Cities Complex Site Flood Analysis
- Attachment 4 – Shallow Flooding at Intakes

2. Analysis of Flood Risk in the Delta

2.1 High Water Conditions

The chance of flooding of lands in the Delta region varies greatly, depending on seasonal conditions and location. Levees on the periphery of the Delta, where the ground surfaces transition from below sea level to above, are only vulnerable to flood failures during flood events, which in California's Mediterranean climate, are most likely to occur during the winter months. The probability of major flooding rises in the fall, and peaks in the December through February timeframe, then decreases through the spring.

Much of the interior Delta region lies below sea level, under constant threat of inundation and protected by a generally fragile levee system. The lower the interior island elevation, the greater the hydraulic pressure. Furthermore, the lower the island interior elevation, the greater the hydraulic differential available to generate rapid breach erosion and high breach inflows. The levees are under greatest threat during major flood events, when huge flood inflows, high tides, wind waves, and rainfall put enormous strain on the levee system. High water increases the hydraulic pressure from the water side, strong currents cause erosion, high water and wave wash threaten levees with overtopping, and the high water combined with heavy rains saturate the levee sections and weaken them.

In general, levee foundation conditions are better on the perimeter of the Delta, where the levees are founded on mineral soils, whereas in the central and western Delta regions levees are often founded on, or are adjacent to, highly variable layers of deep peat, poorly consolidated sands, silts, and clays that are prone to under seepage and structural weaknesses.

Some generalizations can be made about the geographic differences in the nature of the flood threats in various regions of the Delta:

- **North Delta:** Flood concerns in the North Delta are particularly acute. Here the combined flood flows of the Morrison Stream Group, Dry Creek, the Cosumnes River, and the Mokelumne River converge and accumulate because the downstream Delta channels lack the capacity to convey the combined flow to the San Joaquin River. River stages rise until levees give way or are overtopped, such as occurred in February of 1986. In that flood event, the levees failed on McCormack-Williamson Tract, Glanville Tract, Dead Horse Island, and Tyler Island like dominoes over a period of hours on the afternoon and evening of February 18, subsequently followed by a failure on New Hope Tract.
- **West Delta:** In the west Delta region, high water stages due to tides and total Delta inflow (especially from the Yolo Bypass) and high winds can result in extreme wave wash erosion, displacement of riprap, and waves overtopping the levees. Deep peat and weak foundations combined with island interiors well below sea level all contribute to the structural stresses on west Delta levees.
- **South Delta:** Extended periods of snowmelt, extending into June and July, are more likely to affect the southern portion of the Delta in wet years, due to large accumulations of snow at high elevations in the southern Sierra mountains. These conditions can increase the risk of levee failures due to scour, seepage, and slumping.

2.2 Seismic Risk

The Delta is vulnerable to seismic events given the presence of multiple faults underlying the western Delta and the proximity of the San Andreas Fault system to the west. Despite extensive geotechnical exploration and multiple analyses by seismic experts there remains uncertainty regarding the effects of potential seismic events on Delta levee integrity. The United States Geological Survey estimated that there is a 62 percent probability of an earthquake of magnitude 6.7 or greater occurring in the San Francisco Bay Area between 2003 and 2032 (DWR, 2009a). An earthquake of that magnitude can cause multiple levee failures in the Delta that could result in fatalities and extensive property damage.

2.3 Sunny-Day Levee Failures

Sunny-day levee failures occasionally occur in the Delta. These can be attributed to several factors such as burrowing animals, pre-existing weaknesses in levees and their foundations, slow deterioration of levees over time, damage due to excavation or dredging activity close to the levee, and other circumstances (DWR, 2009b).

2.4 Flood Risk and Proportional Response

2.4.1 Flood Risk Definition

As used in this TM, flood risk is proportional to the combined effect of the chance of flooding and the consequences of flooding. Theoretically the chance of flooding at any given time and place can be estimated by evaluating the combined seasonal probability of various types of flood events, triggered by storms, high tides, wind waves, seismic events, structural weaknesses, human actions, or other events. The consequences of flooding can be defined or measured in terms of loss of human lives, depth and duration of flooding, damage to property, effects on the functioning of important infrastructure, damage to fisheries and wildlife, and other significant measures.

2.4.2 Proportional Response

Flood risk management consists of reducing the chance of flooding, limiting the damageable property and population exposed to flooding, or a combination of both. Flood risk management is guided by the common-sense concept of proportional response, wherein the extent of investment in flood risk reduction measures is proportional to the flood risk as described above. Thus, a large investment in flood risk management may be warranted even if the chance of flooding is small but the consequences are large, as might be the case for a large urban area. A similar investment may be justified in cases where the chance and frequency of flooding is greater, but the consequences are relatively smaller.

Most flood events in the Delta are associated with major winter storm events that evolve over a period of days or weeks. In accordance with the proportional risk concept, as the chance of flooding associated with synoptic events increases, the entire flood management system is activated to a high state of readiness, including actions such as activating flood-fighting personnel at the various levels of public agencies and supporting contractors, conducting around-the-clock levee patrols, increasing the frequency and coverage of weather, tide and river stage forecasts, activating mutual aid contracts, and evacuating vulnerable personnel and equipment, as appropriate in various at-risk locations.

Consistent with the proportional response principle, tunnel shaft sites that face a greater chance of flooding and/or greater consequence of flooding due to the potential speed and depth of flooding, duration of occupancy, number of workers, and damageable infrastructure require greater investments in risk mitigation than less threatened sites.

2.4.3 Consequences of Flooding

2.4.3.1 Focus of Analysis

This TM is focused on the consequences of flooding at the tunnel launch, reception, and maintenance shaft sites during the period of construction. Worker safety is the paramount concern; specifically, the intent is to limit the risk to workers of being injured or losing their lives in the event of a flood during the period of construction. A proportional level of flood protection for the infrastructure and equipment at each construction site is also an important consideration.

2.4.3.2 Guidance from Historic Events

The consequences of flooding in the Delta depend a great deal on the nature of each flood event. In particular, given that most of the tunnel shaft sites are protected by levees, the consequences of flooding are directly related to the potential locations and characteristics of levee failures. While it is impossible to define either the locations or the specific levee failure scenarios in advance, historic events are instructive and can provide useful guidance for managing the consequences of levee failures (URS, 2008a). Additional useful guidance can be obtained through computer modeling of levee breach and island inundation scenarios.

Some useful observations of historic Delta levee breach events:

- **Early Detection:** Levee failures usually develop over a period of time, with visible indications of structural distress: boils, slumps, waterside scour from wave wash, loss of erosion protection, are typical examples. Frequent and vigilant levee patrols have resulted in early detection, advance warning for affected personnel, and often successful flood fights. The most recent example is the well documented flood fight on Tyler Island in mid-February 2017, in which nearly half of the levee section failed, but a levee breach was averted through quick and effective remedial measures.
- **Breach Conditions:** The locations of levee failures often develop dangerous currents and extensive scour excavations, as much as 2,000 feet long and 50 to 60 feet deep in spots. Where peat soils are present, they are often excavated in large blocks, locally referred to as peat bergs. A levee breach typically grows rapidly as the sides are eroded away from below, with mass caving of levee sections creating dangerous conditions in the vicinity of the breach. Floodwaters then spread out, moving overland towards the lowest parts of the island, backfilling like a lake until the water surface elevations in the interior and the channel equalize.
- **Rate of Flooding:** A small Delta island can flood and fill in an hour, a large island can take 24 hours or more to fill. An example taken from the mid-February 1986 flood event, which resulted in several levee failures, is instructive (See Attachment 1). From these and numerous other Delta levee breach events it is clear that a levee breach can result in very rapid flooding, scouring flows, and extreme danger in the vicinity of a breach and that Delta islands can fill rapidly.
- **Flood Damage:** Once an island is flooded the interior slopes of the levee system are attacked by wave wash, structures and infrastructure are damaged.

3. Flood Risk Management Measures

The Project would include a combination of non-structural and structural flood risk management measures to reduce the risk of flooding at the Project construction sites, including the sites supporting launch shafts, reception shafts, and maintenance shafts. In this context, non-structural measures could involve temporary facilities or equipment, but such facilities or equipment would not significantly affect the construction footprint or onsite activities.

The non-structural measures would involve fully integrating the Project construction team with the existing Delta flood preparation, response, and recovery system, as summarized in the following section. This would provide for the construction team members to understand the nature of flood risk in the Delta, be properly trained and equipped to deal with flood emergencies, be aware of real-time conditions, and participate in mitigating flood risks, if necessary.

Structural measures would also be integrated with the design of the tunnel infrastructure such that the short-term risks during construction, and the long-term risks during the subsequent operation of the facility, are minimized.

3.1 Non-Structural Flood Risk Management Measures

The options considered for managing flood risk at the launch, reception, and maintenance sites during construction would be evaluated in the context of the existing Delta flood risk management system which in is embedded in statewide, national, and international systems that are summarized in this section.

3.1.1 Involved Agencies

In addition to the LMAs that manage the majority of Delta levees, key federal and State agencies with direct responsibilities, authorities, or emergency support roles over Delta levees are USACE, the Federal Emergency Management Agency (FEMA), the Bureau of Reclamation (Reclamation), the California Office of Emergency Services (CalOES), the Central Valley Flood Protection Board (CVFPB), and DWR. A multitude of other federal and State agencies, utilities, non-governmental entities, property owners, businesses, and residents also have roles and interests that affect Delta levee management.

There is a complex existing system of governmental and nongovernmental entities with an interest in Delta levees and the assets they protect; and there exists a remarkably effective, cooperative system in place for managing the risk of Delta flooding. An awareness of this cooperative system and how construction of the Project should interface with it is the most important and fundamental first step in managing flood risk associated with the construction process.

The existing Delta flood risk management system addresses flood preparedness, emergency response operations, and flood damage recovery operations.

3.1.2 Flood Preparedness

Flood preparedness includes many elements, such as individual and coordinated training for the involved agencies and personnel, the stockpiling of flood fighting supplies and equipment, preparation of sites for conducting flood operations, establishing and operating hydrometeorological and hydrologic data collection systems and the means for forecasting future conditions, establishment of the California State Standardized Emergency Management System (SEMS) and the Federal Emergency Management System (FEMS) for command and control of emergency events, establishing emergency communication protocols and equipment, training and equipping flood-fight specialists, establishment of mutual aid agreements, and so on.

3.1.3 Emergency Response

Emergency response operations are focused on real-time assessment of conditions in the field through manual levee patrols and a complex web of data collection systems, the early identification of potential levee failures and preventing them from developing into actual failures through flood-fight operations, minimizing the loss of life and property damage in the event of flooding, and limiting the damage to levees once flooding has occurred.

3.1.4 Post-Flood Recovery

Flood damage recovery operations involve repairing levee breaches, pumping out flooded areas, restoring levee cross-sections damaged by scour and wave wash, restoring critical infrastructure, clearing debris, and addressing toxic cleanup issues.

3.1.5 Non-Structural Measures for Managing Flood Risk for Construction Personnel and Equipment

3.1.5.1 Coordinate and Cooperate with Levee Maintaining Agencies

On the front lines of this multi-layered, multi-agency system are the LMAs. Agency trustees, engineers, and maintenance personnel have accumulated many years of operational experience and are intimately familiar with the characteristics of their levees and the surrounding river channels and sloughs. They are also familiar with the conditions associated with heightened chances of flooding and are prepared to offer seasoned advice on how best to respond to them. It is therefore advisable that Project personnel would establish good relations and clear communication channels with the reclamation districts that surround construction sites.

3.1.5.2 Provide SEMS and FEMS Training

A key to worker safety is appropriate training to understand the nature of the flood risks and how best to respond to emergencies. Therefore, it is recommended that construction personnel be trained in emergency notification protocols, provided with emergency contact information for the key emergency management personnel within the multi-layered flood risk management system, and be trained to have a basic understanding of SEMS and FEMS.

3.1.5.3 Provide Delta-Specific Safety Training

Project workers would be trained to safely deal with the typical hazards associated with working in the Delta under flood conditions which can include poor visibility, driving rain, high winds, breaking waves, river currents, soft and unstable ground, debris, and so on. The potential for drowning, hypothermia, and vehicle accidents are much higher under flood conditions. The extreme danger and unpredictable conditions posed by levee breaks in progress should be especially well understood.

Driving in the Delta, especially on levee roads poses significant risk. With a steep drop-off on each side, narrow shoulders, and a river channel on one side, there is little room for error. Fatalities involving vehicles going off the road and landing in water or flipping over are fairly common. The risks are especially high at night with fog and rain. Workers could be trained to take these conditions into account any time they are driving in the Delta, and especially during emergency evacuations.

3.1.5.4 Provide Individual and Facility Emergency Kits

Each worker would be provided with a kit of emergency response gear to keep in their vehicles for the duration of the construction period. This could include a personal floatation device (PFD), all-weather gear, a cell phone with good reception in the Delta, a first aid kit, a flashlight, flares, a shovel, a pack of sandbags, some stakes, twine, and a throw line. Workers could also be provided with training in flood fighting techniques, as such a trained workforce could potentially play a critical role in preventing a local levee failure during an emergency, thereby providing a valuable service to the reclamation district and Delta at large, as well as preventing delays and damage to tunnel infrastructure. Additional emergency response supplies and equipment would be stockpiled onsite, scaled to the needs of the entire construction team onsite.

3.1.5.5 Risk Awareness and Evacuation

With a commitment for close coordination with the existing Delta flood management system, Project tunnel construction workers would need to have a high degree of situational awareness for the most likely types of flood events and the dangers they pose. In most cases there would be advance warning of dangerous conditions during which time there is an elevated risk of levee failures. During such periods, likely to last for a period of days, or at most a few weeks, it is typical for Delta residents and businesses to prepare for evacuation and move mobile equipment to high ground. Similarly, for the Project tunnel construction project the most reasonable risk management measure would be to evacuate workers from the construction site and secure vulnerable equipment, if possible, by moving to high ground.

Thus, it is essential to have in place clear emergency response plans, uniquely suited to each construction site, well before any flood emergency develops, including designated evacuation routes or safe spots to assure worker safety when there is little or no warning. Pre-planned evacuation routes would consider the location of a levee breach and which roads will be passable and lead to safety. Real-time information about the occurrence and location of a breach and the likely flooding scenario would be essential to safe evacuation. Close coordination and communication with the local reclamation district and the regional, State, and federal emergency response system would be essential. It would also be important that construction workers remain generally alert to their surroundings and prepare to take emergency action if flooding is threatened or underway.

3.1.5.6 Shelter in Place

Advance warning is not always possible. Seismic events and unanticipated levee failures can take everyone by surprise. In the event that evacuation ahead of rising floodwaters is impractical because the flooding is occurring rapidly and without warning and/or roads and bridges to escape the floodwaters have become unsafe or impassable, there are a variety of options for allowing workers to escape floodwaters onsite or in close proximity to the site. Any such flood safety measure should be secure up to the 100-year flood level and not be dependent upon subsequent rescue efforts to assure worker safety. Several practical options are summarized below, and depending upon the site-specific risks, could be implemented singularly or in combination:

- If the island levee is in close proximity, construct a short all-weather road, elevated several feet above the surrounding ground to the levee crown, and widen the levee crown sufficiently at this location to provide space for worker vehicles.
- For the duration of construction, tether one or more boats to high points on the land structure that, in the event of rapid site flooding, could accommodate site workers and critical equipment, safely floating with the rising floodwaters. Depending upon specific site requirements this could involve leasing or renting one or more boats with sufficient capacity for the number of workers expected to routinely be onsite. The boats could be securely tethered to vertical steel piles or with a cable and buoy in such a way that they can freely rise with the floodwater but remain onsite.
- Install a fixed, elevated platform with a weather shelter, such as a stout steel shed on steel posts or wooden piles, with the floor elevation above the 100-year WSEL. Such a temporary facility could Include steel stairs and a construction lift, chemical toilet facilities, water, a generator, and emergency supplies.

3.2 Structural Flood Risk Reduction Measures

Levees in the Delta are primarily rural and are made of sediment dredged from adjacent channels, excavated from island interiors, or imported from other areas. These levees are exposed to many hazards that may damage or cause failure, resulting in flooding. The most significant hazards are due to hydrologic, hydraulic, and seismic (earthquake) loading. Several structural remediation measures to reduce risk during Project construction such as setback levees, ring levees and geometry repairs would be available to reduce the flood risk based on the hazard identified. The elevated shaft pads would provide high ground refuge in the event of a flood. The structural measures discussed below would be considered in addition to construction of the elevated shaft pads.

3.2.1 Setback Levees / Ring Levees

Setback levees are earthen embankment that are located at some distance from a river channel that allow the streamflow to spread by creating a wider riverbed with increased conveyance capacity of the floodway. For areas that are frequently flooded by overtopping of a levee, a setback levee may be constructed to lower the water surface elevations when compared to a levee along the riverbank. These improve flood risk management by reducing the flood risk to lives and property, minimize flood control system operation and maintenance and improve riparian and floodplain ecosystem habitat.

Ring levees are built surrounding an area subject to inundation from all directions. These ring levees provide a secondary barrier of flood protection.

3.2.2 Geometry Repairs

A variety of conditions can contribute to a levee's vulnerability for failure when subjected to loading including poor/weak embankment or foundation soils, insufficient levee geometry (height, width, and slope inclination), that can lead to seepage and stability related failures. Geometry repairs can be performed to improve the levees using alternatives such as cutoff walls, pressure relief systems, seepage berms, shallow drainage and stability berms.

3.2.2.1 Cutoff Walls

Locations that have underseepage and through seepage concerns can be addressed by a cutoff wall. Cutoff walls are vertical, low-permeability, barriers that reduce seepage through pervious layers in the levee, blanket and aquifer. Cutoff walls are generally designed to extend fully through an aquifer and key into a relatively impervious soil layer. For cutoff wall construction, a portion of the levee is typically degraded to create a working surface, and from that working surface, typically a 3-foot-wide cutoff wall is installed. The depth of this cutoff wall would vary depending on the site conditions. Figure 3-1 shows a conceptual cross-section of a cutoff wall.

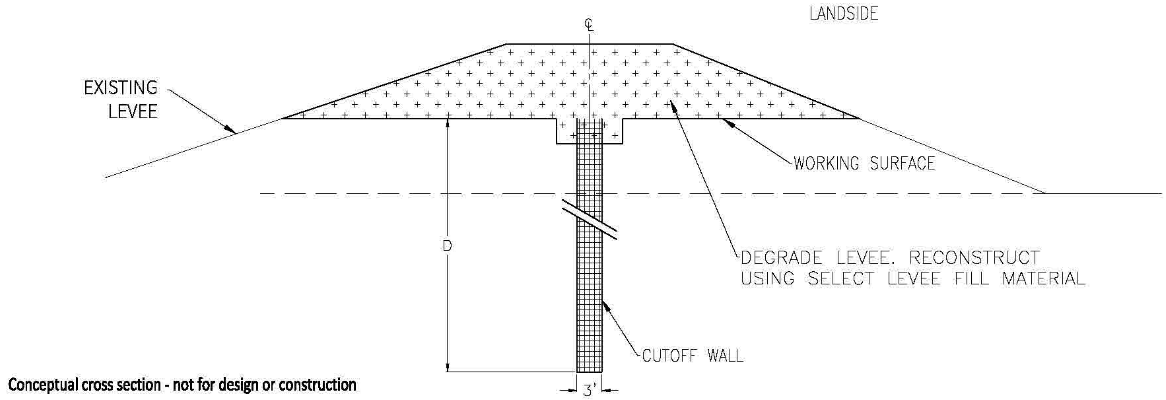


Figure 3-1. Cutoff Wall Conceptual Cross-section (Knights Landing Flood Risk Reduction Feasibility Study)

3.2.2.2 Pressure Relief Systems

Pressure relief wells or trench drain systems located near the landside levee toe relieve underseepage uplift pressures beneath the blanket by providing direct drainage from the underlying aquifer. Underseepage flows from within the aquifer can be captured by pressure relief wells and transmitted using a toe drain trench, typically located near the landside toe of the levee. Toe drain trenches are excavated continuously along the landside levee toe, extending down to the top of the aquifer. By providing an engineered, filtered exit for the pressurized seepage within the aquifer, the relief system provides an exit path for seepage flows that may otherwise emerge in an uncontrolled manner on the landside ground surface. Figure 3-2 is a schematic of a relief well and toe drain trench.

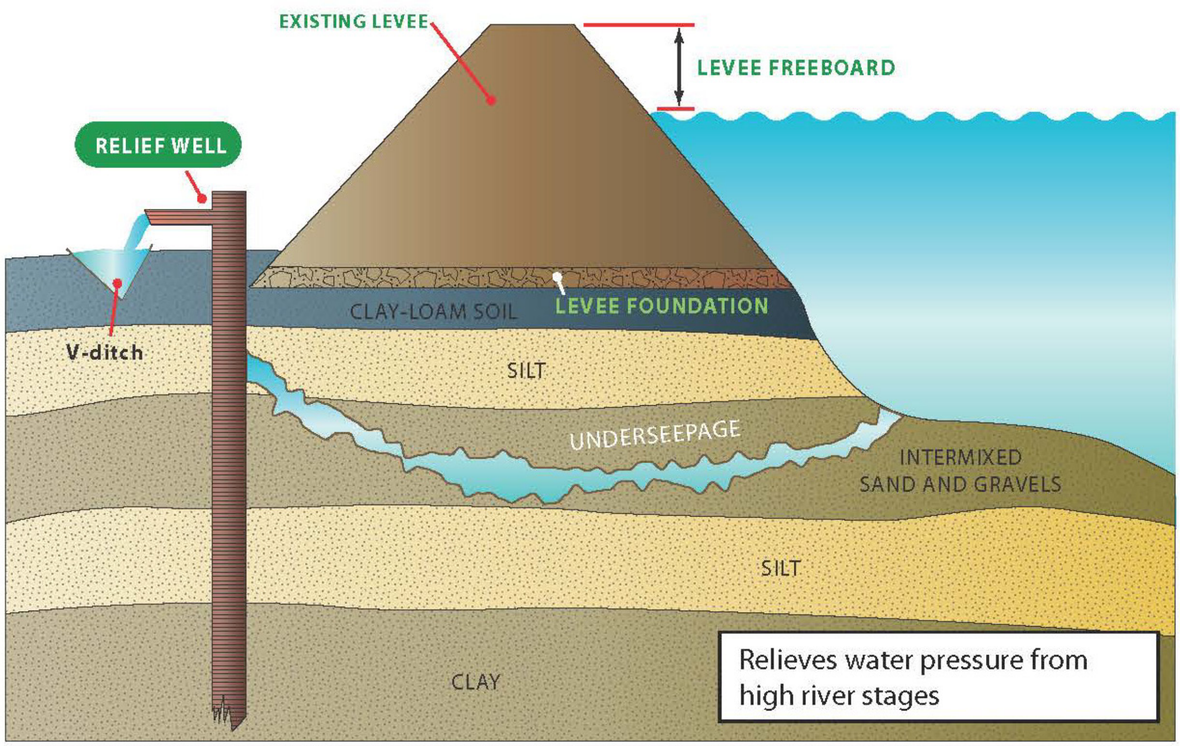


Figure 3-2. Schematic of Pressure Relief Well (Three Rivers Levee Improvement Authority)

3.2.2.3 Seepage Berms

Seepage berms mitigate underseepage by the addition of weight to counteract underseepage uplift pressures near the landside levee toe and increase seepage path lengths so that high seepage gradients are reduced and shifted farther away from the levee toe. Seepage berms are generally designed to be on the order of 100 to 300 feet wide (minimum 4 times the levee height), measured from the levee toe with thicknesses varying from about 5 feet at the levee toe to about 3 feet at the berm toe. It should be noted that seepage berms in the Delta are often taller than 5 feet; as much as half of the landside height of the levee. Figure 3-3 is a schematic of a seepage berm.

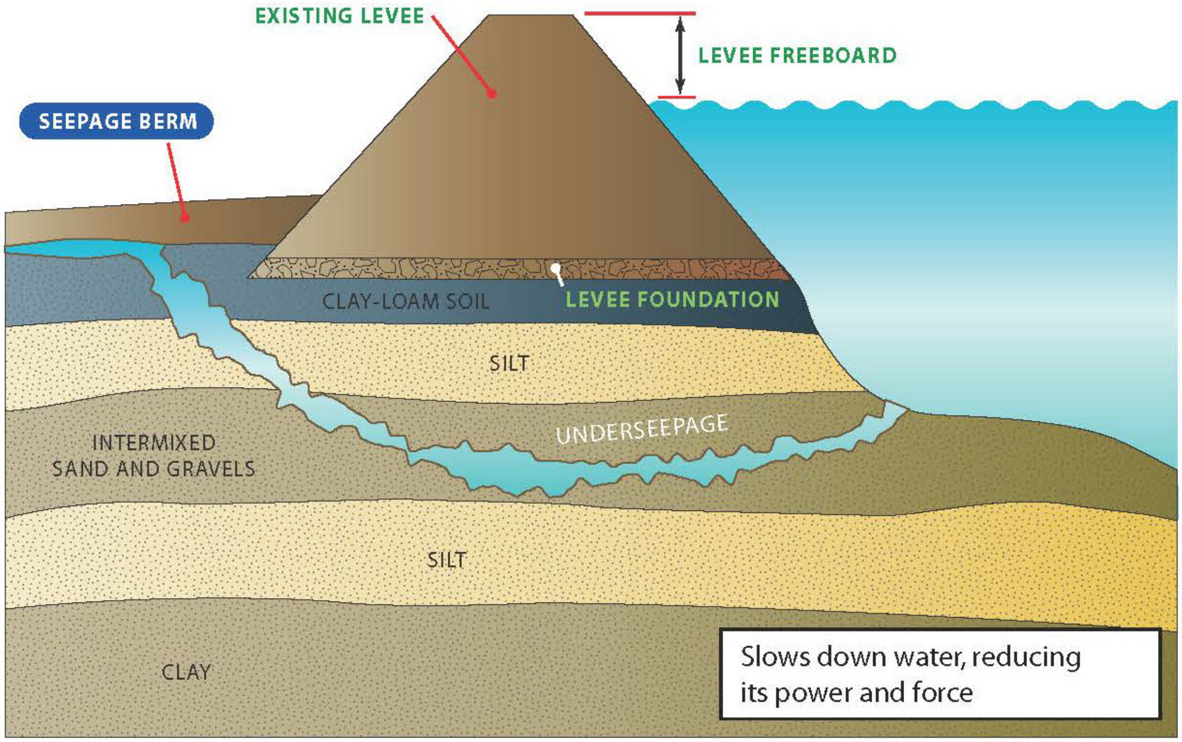


Figure 3-3. Schematic of a seepage berm (Three Rivers Levee Improvement Authority)

3.2.2.4 Shallow Drainage and Stability Berms

Shallow drainage and stability berms address through-levee seepage and landside slope stability by providing a filtered exit to prevent the movement of fine soil particles, and provide weight to buttress the levee. The purpose of shallow drainage systems is to provide a filtered exit for through-levee seepage flows that would otherwise exit the levee slope or ground surface at velocities sufficient to erode embankment or foundation materials. A toe drain would typically be used when through-seepage or through-seepage-driven landside slope stability is problematic.

A stability berm is a prism of compacted soil placed on the slope of a levee to act as a buttress to increase stability factors of safety. When placed on the landside slope, a filter/drain zone can be incorporated into the stability berm to capture seepage that would otherwise exit on the unprotected slope, potentially eroding embankment material. Typical stability berms are about 10 feet high and about 10 to 25 feet wide. Stability berms could be a more cost-effective approach than toe drain systems because they are easier to construct and do not require extensive excavation into the existing levee slope or toe. Figure 3-4 is a schematic of a typical drained stability berm.

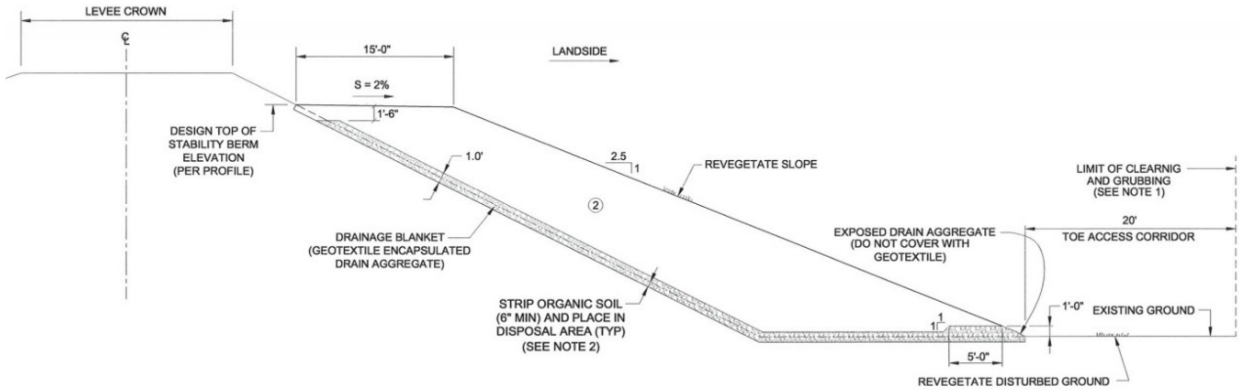


Figure 3-4. Typical landside drained stability berm construction detail (GEI Consultants, 2014)

4. Evaluation of Delta Flood Risk at Launch, Reception and Maintenance Shafts

As shown in Figure 1-1, double launch sites would be located at the Twin Cities Complex and Lower Roberts Island. Reception Shafts would be located at intake C-E-3, Terminous Tract, and the Bethany Reservoir Pumping Plant (BRPP) Surge Basin. Maintenance Shafts would be located at the C-E-5 intake, New Hope Tract, Canal Ranch Tract, King Island, Upper Jones Tract, and Union Island.

These sites would be occupied for the duration of construction, although the number of workers onsite would change over time. For the majority of the construction period, up to 12 or 13 years for the launch shaft site, it is anticipated that there would be as many as 100 workers onsite, including construction managers, engineers, inspectors, heavy equipment operators, equipment maintenance personnel, and support staff. Shelter-in-place options, such as those described above, would be considered for each site and implemented prior to construction of the launch access shafts.

Once the site preparations have been completed and tunneling started, activity would continue continuously. It is therefore assumed for the purpose of this risk analysis that the full workforce of 100 would be onsite continuously. Parking would be provided, and it is assumed that workers would commute to the work site using trucks, automobiles, and buses, a significant consideration in the event that site evacuation would be implemented quickly due to an emergency.

Given the long duration of the work at these sites, island perimeter levee improvements to meet PL 84-99 geometric and geotechnical standards or ring levees, as well as addressing any known geotechnical weaknesses, are warranted to limit the long-term flood risk. To gain an order-of-magnitude estimate of the extent of the remedial work that would be required to achieve this standard, a Levee Vulnerability Assessment has been completed for all of the island levees along the Bethany Reservoir Alignment (CER Appendix F2), and presented in a separate TM. The extent and types of recommend levee repairs would be refined prior to construction and in coordination with the local reclamation districts. The levee improvements would be initiated in the early phases of Project construction and may overlap to some extent the initiation of shaft pad construction at the shaft sites. However, if critical weaknesses are identified in these levee systems, additional remediation would be completed before shaft sites are constructed. Ongoing and continuous levee maintenance and monitoring would be critical to reducing flood risks at the shaft sites during Project construction and should be closely coordinated with the reclamation districts. Following construction of the levee improvements, the modified levees would become part of the local reclamation district's facilities and would be maintained during and following construction by the local reclamation district. The Levee Vulnerability Assessment presented in a separate TM (CER Appendix F2) is summarized in the following sections.

It is anticipated that the elevated earthen pads at the shaft sites could be constructed relatively quickly, within a time frame of about one to two years for each site. After site preparation, including any needed foundation strengthening, an elevated pad with access ramps would be constructed to create a work platform equal to, or above the projected 2040 100-year WSEL as summarized in Table 4-1. Subsequently, a vertical shaft would be constructed for launching or accessing the tunnel. In the event of a sudden and unforeseen levee breach, workers at the shaft sites would generally be protected at pad elevations but would face the potential of being trapped. The earthen pads would provide a structural refuge for flood conditions, but should also be combined with evacuation elements such as rafts or launchable boats in the event that rapid site evacuation is needed. Table 4-1 shows the locations of launch, reception, and maintenance shafts, as well as local 100-year WSELs and top of shaft pad elevations.

Table 4-1. Location of Launch, Reception and Maintenance Shafts

Site Description	Maximum 100-Year WSELs (NAVD88) at Shaft Pad Sites Based on Lowest Perimeter Levee Crest Elevation	Shaft Pad Elevation (NAVD88)
Launch Shafts at: Twin Cities Complex	19.4 feet	21 feet
Launch Shafts at: Lower Roberts Island	9.6 feet	13 feet
Reception Shafts at: Intake C-E-3	20.7 feet	26 feet
Reception Shafts at: Terminous Tract	10.2 feet	13 feet
Reception Shafts at: BRPP Surge Basin Reception Shaft[a]	NA	NA
Maintenance Shafts at: Intake C-E-5	20.7 feet	24 feet
Maintenance Shafts at: New Hope Tract	12.6 feet	19 feet
Maintenance Shafts at: Canal Ranch Tract	11.2 feet	15 feet
Maintenance Shafts at: King Island	9.6 feet	13 feet
Maintenance Shafts at: Upper Jones Tract	9.6 feet	13 feet
Maintenance Shafts at: Union Island	9.9 feet	12 feet

[a]BRPP Reception Shaft is located above the floodplain and does not require a shaft pad for flood protection.

4.1 Levee Vulnerability Assessment

A preliminary assessment of Delta levees was performed to evaluate relative levee vulnerability based on indicators of levee performance. Detailed discussions of the individual criterion, rating systems, and the overall levee vulnerability rating are provided in the separate CER Appendix F2. A summary of the levee vulnerability study is provided below.

The relative vulnerability criterion was developed based on the geometric design standards provided in the Flood Hazard Mitigation Plan (HMP), Public Law 84-99 (PL 84-99), and DWR Bulletin 192-82. Table 4-2 summarizes the minimum levee configurations suggested by these design standards. Additional factors, such as the presence of landside ditches or vulnerability to sea level rise were also considered, but the employed weighting system factored existing geometry heavily in the assessment.

Table 4-2. Minimum Levee Configurations

Criteria/Design Standard	Flood Hazard Mitigation Plan	Public Law 84-99	DWR Bulletin 192-82
Crown Width	16 feet	16 feet	16 feet
Freeboard	1 ft above 100-yr WSEL	1.5 ft above 100-yr WSEL	1.5 ft above 300-yr WSEL (non-urban) 3 ft above 300-yr WSEL (urban)
Waterside Slope	1.5H:1V	2H:1V	2H:1V
Landside Slope	2H:1V	3H:1V to 5H:1V	3H:1V to 7H:1V (without berm) 3H:1V to 13H:1V (with berm)

Each criterion was evaluated at a spacing of 500 feet along the levee centerlines at more than 5,000 individual cross-sections. Figure 4-1 shows the levee geometry evaluation based on HMP, PL 84-99, and DWR Bulletin 192-82 along with the locations of launch, reception and maintenance shafts. As shown in the above table, HMP has the least stringent criteria for levee geometry, whereas, DWR Bulletin 192-82 has the most stringent criteria using a 300-year WSEL for establishing levee crest height and freeboard, as well, as potentially flatter landside slopes. Bulletin 192-82 also allows for the use of berms in some conditions, which were also considered in conducting the geometry evaluations included in the levee vulnerability study.

In addition to the individual criterion evaluated at each cross-section, a levee vulnerability score was developed for each cross-section based on a scoring and weighting system to provide a single metric that can be used to compare the relative vulnerability of one levee cross-section to another. The range of levee vulnerability scores were divided into quartiles representing Levee Vulnerability Ratings of "Very Low," "Low," "Medium," or "High" relative vulnerability. Figure 4-2 shows the relative vulnerability of cross -sections in the Delta within the boundary of the Project along with the locations of launch, reception and maintenance shafts.

This analysis is intended as a screening-level assessment to identify where potential repairs may be needed and does not replace the need for site-specific evaluations. The results from this study are intended to help locate Project infrastructure within levee systems that may require less mitigation to meet Project standards relative to other levee systems and is generally consistent with reclamation districts' flood risk reduction approaches. Refer to the CER Appendix F2 for more detailed discussions on the levee vulnerability assessment approach and process.

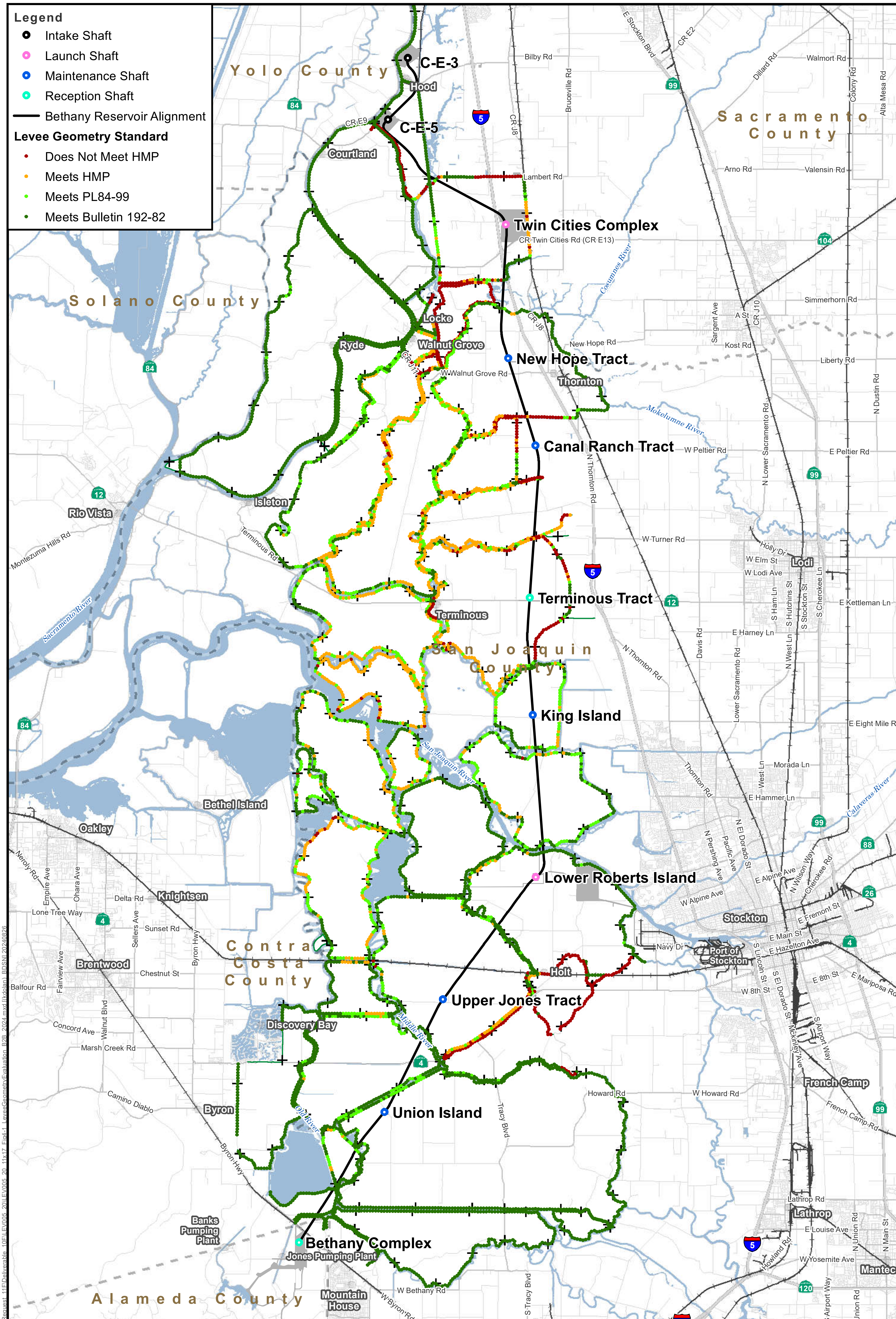
Newer studies such as the Delta Stewardship Council's Delta Adapt program have analyzed flooding and levee vulnerability in the Delta. However, these studies are focused on assessing potential future conditions. The assessments in this TM are focused on current conditions to provide a relative assessment of levee performance with the acknowledgment that the local LMA's are continuously improving levees to meet federal standards, and current conditions may not represent future conditions,

4.1.1 Levee Setbacks for Project Stockpiles

Potential changes to surrounding levees were given consideration when siting Project facilities. Stockpiles were identified as one feature which could potentially affect the performance of adjacent levees depending upon ground conditions, stockpile sizes, and their location relative to existing levees. Potential changes could arise from increased pore pressures on the levee foundation either through blocking seepage outlets or surcharging the levee foundation. The State of California developed the Urban Levee Design Criteria (ULDC) to provide consistent engineering levee design criteria for the protection of urban areas (DWR, 2012). No similar guidance has been developed by the State for levees in rural areas. The ULDC provides clear criteria for levee improvements including seepage berm widths up to 300 ft from the levee toe. USACE has also studied seepage berm widths in soft soils and stated that berms do not need to be wider than 300 to 400 ft since a levee that includes a seepage berm of this extent would likely be safe against underseepage (USACE, 1956). As a result of this guidance, 300 ft is typically considered to be the maximum design berm width due to reduced affects to the levee as the distance away from the levee increases.

In addition to the seepage considerations, the potential effect of stockpiles acting to surcharge soft soil (i.e., peat) which is present beneath some of the levees was also considered. In accordance with the USACE Engineering Manual EM-1110-1-1904 (USACE, 1990), the area of influence beyond the stockpile footprint can be estimated using the 2V:1H method to approximate the stress distribution beneath a loaded area. The area where those stresses would occur can be approximated by increasing the footprint by 1 foot horizontally for every 2 feet of depth below the stockpile. Based on a review of the Delta Risk Management Study (DRMS) (URS, 2008b), the depth of interest for the soft soils beneath levee embankments throughout the Project area range from 0 to up to 40 feet. Using the 2V:1H method, the area of affected soil at the deepest peat zones of interest would extend up to approximately 20 feet beyond to the toe of the stockpile.

Site specific analyses of levee stability were not performed as part of the conceptual design process, but should be considered during future design phases. For the purposes of conceptual design, it was assumed that all stockpiles would be placed at least 300 feet away from any levee toe to limit potential effects associated with placement of fill.



For Illustration Purposes Only

0 1.75 Miles

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Figure 4-1.
Levee Geometry Evaluation

DCA DELTA CONVEYANCE DESIGN & CONSTRUCTION AUTHORITY

Data Source: DCA, DWR

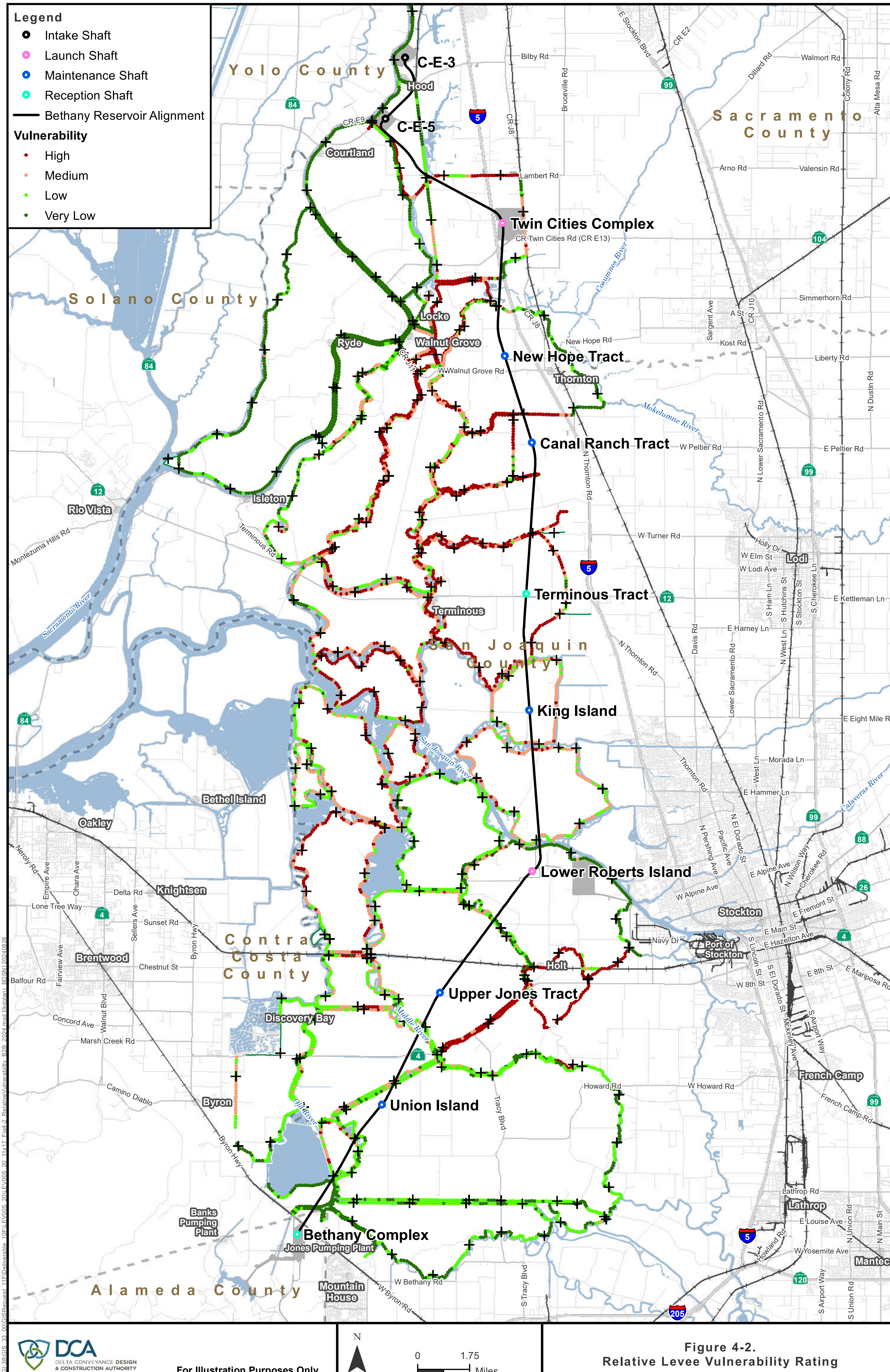

Figure 4-2.
Relative Levee Vulnerability Rating

Data Source: DCA, DWR

4.2 Flood Inundation Analysis

4.2.1 Speed of Flooding

Levees in the Delta are exposed to many hazards that may damage them or cause failure, resulting in flooding. The unique geographic, topographic, and hydrologic characteristics of each shaft site affect the level of flood risk at that site. Important determinative factors are: 1) the likely speed of flooding (and thus escape and rescue time windows), 2) the likely depth of flooding, available evacuation routes, and 3) the extent to which the flood risk varies over the seasons. Of particular concern is the time to flood in the event of a sudden and catastrophic levee failure. The faster a flood occurs, the less chance there is to take safety precautions. Thus, it is important to obtain a measure of the likely time to flood in the event of a breach for each shaft location. However, the time it takes for a Delta island or tract to flood, or for floodwaters to reach a specific elevation or location on the island, are difficult to predict. Records of past Delta island failures are informative and can provide some order-of-magnitude estimates (see Attachment 1).

Simulations of levee breach scenarios can be conducted to various levels of complexity. In assessing the site-specific flood risk associated with each shaft site it is useful to estimate the time it would take for floodwaters from a breach of the island levee to reach the shaft site, and subsequently how quickly the flood water would continue to rise. The time for rising floodwaters to reach a particular elevation and location in a flooding Delta island would depend upon the water level in the channel at the breach location, the flow capacity of the nearby channel, the geotechnical characteristics of the levee and its foundation, and the topography of the island interior.

A preliminary estimate of the time to flood for each of the shaft sites has been completed, as described in Attachment 2: Flood Inundation Analysis. To determine the inundation characteristics a simplified approach was used to get a range of flood times and elevations at each shaft site. This analysis utilized work performed previously by the DRMS study team to determine the average levee breach geometry (depth and width)[1]. Based in review of past Delta levee breaches, it was assumed for this study that a 500-foot levee breach would occur instantly, removing the entire levee cross-section down to the levee toe. Head differential, used to compute the flow through the breach, was estimated as the difference between the 100-year water surface elevation (WSEL) and the minimum landside toe elevation of the island perimeter levee based on LiDAR (DWR, 2017). The 100-year WSELs used for the assessment are based on geographic information system (GIS) data compiled by DWR for *Analysis of Delta Levees Compliance of HMP [Hazard Mitigation Plan] and PL 84-99 Design Geometry* (DWR, 2011) as described in the DLIS. The hydrologic inputs are largely based on previous hydrology studies prepared by USACE in 1976 and 1992 for the Sacramento – San Joaquin Delta (USACE, 1976; 1992). It is recognized that the effects of sea level rise will become more understood in the future and will change the WSEL. However, the timing of water flowing through a levee breach is not anticipated to change substantially. Therefore, the historical values for WSEL have been used in this analysis. As a conservative approach, the location of the largest head differential along the levee perimeter of each island was assumed for the breach characteristics. Additional details on the methodology used to estimate the flow are provided in Attachment 2.

Based on the 2017 LiDAR data, elevation capacity curves were estimated for Delta islands that would potentially contain the Project infrastructure. Using the elevation capacity curves, the storage capacity

[1] In the DRMS analysis 14 breach scour holes remaining from historic levee breaches were measured from aerial photography. Scour holes ranged in width from 176 feet to 1,018 feet, with an assumed average of 500 feet. The scour holes were generally found to be wider than the levee breaches that caused them, hence the 500-foot breach width is a reasonably conservative estimate for the purpose of the current analysis.

was estimated at the shaft pad ground elevation and the minimum levee crest elevation for the perimeter levee of the island. Storage capacity was also estimated at the inundation depth representing 66% of the head differential (between the 100-year WSEL and interior levee toe), since the broad-crested weir calculation is considered reasonably representative and accurate up to this elevation (Hamill, 2010). Beyond this threshold the backwater pressure from the filling island gradually reduces the flow rate until the water level inside the island and outside equalize; at which point, flow through the breach would stop.

Bookend values of the weir coefficient[2] of 0.4 and 0.2 (Lee, 2019) were used to develop the discharge coefficients (C_d) and estimates of island fill time, thus providing an indication of the range of uncertainty in the time it would take for floodwaters to reach the ground elevation at the shaft sites and the floodwater depth 1 hour after the initiation of the breach. Table 4-3 shows the minimum and maximum estimated time to overflow the ground elevations at the shaft sites, applying the high and low values of C_d and the minimum and maximum depth of flooding at one hour after the initiation of the breach.

By this simulation approach, floodwaters pouring through a 500-foot-wide breach on Lower Roberts Island would take less than 1 hour to overflow the ground elevation at the tunnel launch shaft site. Similarly, floodwater pouring through levee breaches at Intake C-E-3 or on King Island, and Union Island would take less than 1 hour to overflow the ground elevations near the shaft pads on these islands. The time to inundate the tunnel shaft locations at New Hope Tract, Canal Ranch Tract, Terminous Tract, and Upper Jones Tract would take at least 3 hours, 2 hours, 1.9 hours, and 3.5 hours, respectively.

For levee breach events that result in rising floodwaters in less than 1 hour to reach the ground elevation at the shaft pads, the inundation depths varied from 0.2 feet to 3.8 feet as also shown in Table 4-3.

4.2.2 Maximum Depth of Flooding

The flood danger at a shaft site would generally increase with increasing depth of flooding, particularly as water depths exceed drowning depths and the elevations of readily-available refugia such as car tops and building roof tops. The maximum flood depths relative to the ground surface at each of the shaft sites during a 100-year flood event was estimated by comparing the 100-year WSEL at the assumed point of levee failure with the ground surface elevation at each shaft site. This set the maximum depth, assuming the floodwaters would be contained within the levee system.

In some cases, especially on the perimeter of the Delta, the water surface gradient along the waterways might be steep enough so that floodwaters entering the levee-enclosed island might flow across the island to a low point and spill into the channel at a lower water surface elevation than where it entered. New Hope Tract is one such example. The Mokelumne River Channel flows around the northern perimeter of the island from the east to the southwest, dropping as much as 10 feet in elevation along the way. To take this potential effect into account in this analysis, the levee crown elevation was checked around the island perimeter and the lowest crown elevation was assumed to establish the highest flood water elevation at the shaft site. The maximum estimated flood depth at each shaft pad site is also tabulated in Table 4-3, and this value is qualitatively taken into account in the overall site hazard rating included in Table 4-4. The time required for floodwater to rise to the maximum level was not computed since as noted earlier, the Broad Crested Weir equation cannot be applied when backwater exceeds 66%, without appropriately reducing the weir coefficient as backwater increases.

[2] Minimum time to effect the shaft locations was computed using a maximum discharge coefficient (C_d) of 0.4. Similarly, maximum time to effect the shaft locations was estimated with a minimum discharge coefficient (C_d) of 0.2.

An external adjacent riverine flood WSEL was considered in establishing the shaft pad elevations. WSEL for each launch, maintenance, and reception shaft pad was developed based on DWR's recent hydrology and hydraulic analyses for the 100-year flood event with anticipated sea level rise (SLR) and climate change hydrology in Year 2040 as documented in the Preliminary Flood Water Surface Elevations (Not for Construction) memorandum (DWR, 2020). WSEL's were taken as the average of the USACE Gage estimates and the DSM2 model results. The WSEL's at each shaft pad were calculated by taking the two nearest nodes where WSEL were provided, and linearly interpolating along the waterways to the site of the shaft pad. Shaft pad elevations were established based on these WSEL's plus an additional 2 feet to provide freeboard at these sites. These elevations should be considered a minimum to provide flood protection during site construction; and during the design phase, future calculations may necessitate higher elevations as additional information becomes available related to climate change and sea level rise.

Table 4-3. Flood Inundation Analysis Summary for Launch, Reception and Maintenance Shafts

Site Description	Shaft Pad Elevation (ft)	Ground Elevation at Shaft Site (ft)	Time to Affect Shaft Site Min (hrs)	Time to Affect Shaft Site Max (hrs)	Depth at Shaft Site at 1-hr Max (ft)	Depth at Shaft Site at 1-hr Min (ft)	Maximum Depth of Flooding (ft)
Launch Shafts at: Twin Cities Complex	21.0	10.0	5.5	12.8	not applicable	not applicable	5.2
Launch Shafts at: Lower Roberts Island	13.0	-10.0	0.9	2.0	0.2	not applicable	20.5
Reception Shafts at: Intake C-E-3	26.0	6.0	0.2	0.5	3.0	0.8	15.0
Reception Shafts at: Terminous Tract	13.0	-8.0	1.9	4.3	not applicable	not applicable	12.2
Reception Shafts at: BRPP Surge Basin Shaft	40.0	40.0[a]	not applicable	not applicable	not applicable	not applicable	not applicable
Maintenance Shafts at: Intake C-E-5	24.0	6.5	2.9	6.8	not applicable	not applicable	14.5
Maintenance Shafts at: New Hope Tract	19.0	5.0	3.0	7.1	not applicable	not applicable	8.9
Maintenance Shafts at: Canal Ranch Tract	15.0	3.0	2.0	4.7	not applicable	not applicable	4.6
Maintenance Shafts at: King Island	13.0	-12.0	0.0	0.0	3.8	1.6	22.5
Maintenance Shafts at: Upper Jones Tract	13.0	-3.0	3.5	8.2	not applicable	not applicable	12.7
Maintenance Shafts at: Union Island West	12.0	-4.9	0.6	1.4	0.2	not applicable	14.9

[a] Surge basin shaft location is above the projected floodplain and therefore not subject to flood inundation

Notes:
hr = hour(s)
Max = maximum
Min = minimum

4.2.1.3 Evacuation Routes and Other Factors

During a levee breach, flood emergency personnel working at a shaft site would need to decide whether to evacuate to safety or shelter-in-place, relying on facility options discussed earlier in this analysis. The evacuation option would generally preferable if it is safe. Ideally each site should have more than one reasonably short, direct, well-marked, and well-maintained road of adequate capacity leading to high ground so that evacuation can be affected regardless of where the levee breach occurs on an island perimeter.

It is also important to consider the proximity of a shaft site to the levee because the site of the breach would pose extreme hazards due to extreme currents, scour, waves, and floating debris. In addition, if a shaft pad site is close to the location of the levee breach the floodwaters may have an almost immediate effect upon the site, regardless of the rate of filling of the island, simply due to the overland flow of the floodwaters from the breach.

These qualitative factors were analyzed for each of the shaft sites and tabulated in Table 4-4.

5. Site-Specific Recommendations

Several site-specific risk factors have been discussed in the foregoing sections:

- The quality of the existing levee system surrounding each shaft site.
- The speed of flooding, measured in terms of the time required for rising floodwaters to reach the site ground elevation or, if that time is less than 1 hour, the depth of flooding at 1 hour.
- The ultimate depth of flooding after the flooding process has reached dynamic equilibrium.
- Other risk factors such as the quality of evacuation routes and proximity to a potential levee failure site.

These risk factors can be considered together to arrive at a cumulative qualitative safety rating. This rating may serve as a useful tool for judging the comparative risk associated with the various shaft sites in a broad and strategic evaluation. It may also contribute qualitatively to the final placement of shaft sites. It is not to be interpreted as a rigid, absolute, or quantitative rating.

The estimates of the pace of flooding were calculated using book-end values for the Broad-Crested Weir Equation, with corresponding ranges in the time required for floodwaters to reach the shaft pad sites or the depth of flooding after 1 hour. This range is shown in Table 4-3 in order to reflect the substantial uncertainty involved in estimating the progression of future levee breaches. For the purpose of defining a site-specific cumulative safety rating the most conservative value is carried into Table 4-4, which shows all these factors together, along with an assigned cumulative safety rating of Low, Medium, and High. Non-structural measures provided in Section 3.1 can be applied to all sites listed in Table 4-4 and are therefore not listed specifically in the table.

As discussed in Section 4.1, the levee systems surrounding each Delta island along the alignment provides the first line of defense against flooding. The reliability was evaluated in terms of compliance with PL 84-99 criteria, which, being intermediate in stringency between the Delta HMP and the DWR Bulletin 192-82 criteria, were a reasonable basis for this preliminary evaluation.

Among the shaft locations along the alignment, the launch sites justify a response proportional to the greater level of risk compared to the reception and maintenance shafts. The launch shaft sites would be active worksites for up to 13 years during construction and would require a substantial number of workers and equipment onsite. Based on the information presented in this TM, Lower Roberts Island would be in the highest risk category, due to the combined effects of levee deficiencies (Figures 4-1 and 4-2), and the timing and depth of flooding (Table 4-3). For this site, it is recommended that levee improvements be initiated at the beginning of the Project to achieve minimum PL 84-99 standards for the perimeter levees. During the design phase, detailed analyses will be conducted to specifically identify the extent of levee improvements, including consideration of sea level rise and climate change assumptions and any levee improvements completed by the reclamation districts prior to construction of the tunnel launch shafts. Updated sea level rise and climate change assumptions will also be used to update post-construction tunnel shaft heights.

The Twin Cities Complex launch site would have a different challenge. Glanville Tract is not fully protected by perimeter levees. The Union Pacific Railroad embankment forms the eastern boundary of the district, but as demonstrated in the February 1986 flood, this embankment is porous and may fail when floodwaters pond on the east side of the embankment. For this site it is recommended that a ring levee be constructed around the worksite rather than constructing a new levee adjacent to the existing railroad embankment along the eastern boundary of the district. The ring levee at the Twin Cities Complex would be constructed to the FEMA Base flood Elevation plus 2 feet of freeboard (FEMA, 2012). This elevation is similar to studies done by Sacramento County. A flood effects analysis of this potential ring levee was performed so that it could be configured to minimize effects to surrounding flood conditions that may occur during a 100-yr hydrologic event on the nearby combined Mokelumne and Cosumnes River watershed as described in Attachment 3.

In addition to the flood analysis discussed above, the intakes were studied to analyze the potential impact of Project facilities on shallow overland flooding which has historically occurred during high-water and storm events. Based on available information, the shallow flooding appears to collect in lower elevation areas and against the abandoned railroad embankment on the eastern side of the intake areas. Existing drainage ditches and pump systems are used to manage flooding in these areas by discharging water into the Stone Lakes canal. The exact source of this flooding cannot be determined without detailed observation at the site and cooperation from local landowners. Local Reclamation Districts were contacted for this study but declined requests to meet with DCA staff to confirm site conditions and observations during past storm events. However, measures can be implemented at the intakes to limit the potential effects of construction on existing flooding near these sites. These measures include rerouting existing agricultural or drainage ditches, implementing stormwater runoff Best Management Practices (BMP), and working with the local reclamation districts and landowners to ensure adequate drainage pump capacity would be available. Additionally, the construction of the intakes includes construction of new levees and improvement of adjacent levees including deep cutoff walls which may help address some seepage problems. Additional detail is described in Attachment 4.

All of the risk factors taken together were considered in arriving at the risk rating shown in Table 4-4, along with site-specific observations and risk mitigation recommendations.

Table 4-4. Site-Specific Cumulative Safety Rating for Shaft Pad Sites

Site Description	Site Characteristics	Ground Elevation at Shaft (ft)	Time to Effect Pad Min (hrs)	Depth at Pad at 1-hr Max (ft)	Maximum Depth of Flooding (ft)	Flood Risk Rating	Recommended Structural Flood Risk Mitigations (Non-Structural Flood Risk Mitigations Apply to all Shaft Sites)
Launch Shafts at: Twin Cities Complex	Main potential of flooding from North Delta streams and tides. The Union Pacific RR embankment east of site was not designed to function as a levee; as seen in February 1986, this embankment may fail if there is high water from Cosumnes River and Dry Creek ponded upstream. Site has ample warning time for weather-related flood events and past inundations of the area generally only take days to drain. There is direct access to Interstate 5 (I-5) overpass, Dierssen Road to west and Union Pacific RR embankment to east.	10.0	5.5	-	5.2	L	Construct 100-year ring levee around site. Assure direct all-weather access to I-5, Franklin Road, Twin Cities Road, Dierssen Road and I-5 overpass.
Launch Shafts at: Lower Roberts Island	Site centrally located in the northern end of moderately deep tract entirely surrounded by Delta channels. Greatest threat is San Joaquin River to north, with large capacity to feed a breach, but threat also exist from Turner Cut and Empire Cut. Shaft site would not be directly affected by breach hydraulics on perimeter but island flooding could occur without warning, to depths exceeding 15-20 feet. Main escape route is HWY 4 about 4 miles to the south.	-10.0	0.9	0.2	20.5	H	Improve Lower Roberts Island levee to meet PL 84-99 standard.[3] Assure all weather access to proximal San Joaquin River levee road. Provide onsite flood refuge for personnel.
Reception Shafts at: Intake C-E-3	Located on left bank Sacramento River at RM 39.4, one mile upstream of Hood, protected by federal-State Sacramento River Flood Control Project Levee. Site is immediately adjacent to levee, with evacuation routes north and south along State Highway 160, and Hood Franklin Road to the east. Flood risk is low, with days or weeks of warning and assessment ahead of a potential breach event.	6	0.2	3.0	15	L	Assure that escape routes have all weather surface. Provide onsite flood refuge for personnel. Construction phasing will assure that new temporary levee with necessary foundation improvements will be in place before river levee is breached. Reroute agricultural or drainage ditches, implement stormwater runoff Best Management Practices (BMP), and design drainage pump within site footprint to address runoff.
Reception Shafts at: Terminous Tract	Site centrally located in large tract, which reduces overland flood consequences. Main potential for flooding from South Fork Mokelumne River (west), Little Potato Slough (west). Also potential flooding from Hog Slough (north), and White Slough (south) Direct access to Hwy 12. Pace and depth of flooding at this site will be moderate; floodwaters will collect at west end, then fill eastward.	-8.0	1.9	-	12.2	M	Assure all-weather access to Highway 12. Provide onsite flood refuge for personnel.
Reception Shafts at: BRPP Surge Basin Shaft	Site located approximately 1.25 miles south of Clifton Court Forebay, South of Byron Highway and East of Mountain House Road. This site has ground elevations between 38 to 54 ft. Because of this, the site is significantly higher than the flood stage in adjacent waterways and would be unlikely to be inundated by a flood event, but may be impacted by local, overland flows.	38.0	-	-	-	L	Site not subject to riverine flooding; normal inclement weather precautions for major construction sites to be developed during design and construction.

Site Description	Site Characteristics	Ground Elevation at Shaft (ft)	Time to Effect Pad Min (hrs)	Depth at Pad at 1-hr Max (ft)	Maximum Depth of Flooding (ft)	Flood Risk Rating	Recommended Structural Flood Risk Mitigations (Non-Structural Flood Risk Mitigations Apply to all Shaft Sites)
Maintenance Shafts at: Intake C-E-5	Located on left bank Sacramento River at RM 36.8, just upstream of Randall Island, protected by federal-State Sacramento River Flood Control Project Levee. Site is immediately adjacent to levee, with evacuation routes north and south along State Highway 160. Flood risk is low, with days or weeks of warning and assessment ahead of a potential breach event.	6.5	2.9	-	14.5	L	Assure that escape routes have all weather surface. Provide onsite flood refuge for personnel. Construction phasing will assure that new temporary levee with necessary foundation improvements will be in place before river levee is breached. Reroute agricultural or drainage ditches, implement stormwater runoff Best Management Practices (BMP), and work with the local reclamation districts to ensure adequate drainage pump capacity is available.
Maintenance Shafts at: New Hope Tract	Main potential for flooding from Mokelumne River (east, north), and South Fork Mokelumne River (west); eastern levee strengthened by DWR, 1996. Escape route via farm road or levee road to Walnut Grove-Thornton Road, leading to I-5 Pace of inundation from east would be modest due to barriers caused by RR and I-5; pace and depth of flooding from west during major flood event would be mild as the island has large volume to the west of site. Ample warning time for flood events and for reacting in the event of a levee failure.	5.0	3.0	-	8.9	L	Assure all weather access to Walnut Grove-Thornton Road. Provide onsite flood refuge for personnel.
Maintenance Shafts at: Canal Ranch Tract	Main potential for flooding from South Fork Mokelumne River (west), via breach on South Fork (west), or via Beaver Slough (north), or Hog Slough (south). Levee failures would result in flooding accumulating at west end of tract, filling slowly eastward. Escape route eastward to high ground via West Peltier Road. Ample warning time for flood events and for reacting in the event of a levee failure	3.0	2.0	-	4.6	L	Assure all weather access to W Peltier Road. Provide onsite flood refuge for personnel.
Maintenance Shafts at: King Island	Site centrally located in moderately deep tract entirely surrounded by Delta channels. Would not be directly affected by breach hydraulics on perimeter, but island is only 3,300 acres and could fill quickly. The only one escape route off island is Eight Mile Road, with bridge. Sudden catastrophic levee failure possible w/o warning.	-12.0	0.0	3.8	15.3	H	Assure all weather access to Eight Mile Road. Provide onsite flood refuge for personnel.

Site Description	Site Characteristics	Ground Elevation at Shaft (ft)	Time to Effect Pad Min (hrs)	Depth at Pad at 1-hr Max (ft)	Maximum Depth of Flooding (ft)	Flood Risk Rating	Recommended Structural Flood Risk Mitigations (Non-Structural Flood Risk Mitigations Apply to all Shaft Sites)
Maintenance Shafts at:	Site located in the center of Upper Jones Tract, adjacent to West Bacon Island Road. Upper Jones tract is generally lower in elevation around the perimeter, and higher in elevation in the center, approximately where the maintenance shaft is located. Site is not close enough to perimeter levee to be directly affected by breach hydraulics. Evacuation route would rely upon Bacon Island Road, which is slightly elevated and would therefore provide longer time to evacuate. Site is in the center of the island, so if the island had inundated the adjacent road, onsite staff would be required to shelter in place.	-3.0	2.2	-	12.7	M	Assure all weather access from site to Bacon Island Road. Provide onsite flood refuge for personnel during pad construction such as elevated evacuation route to adjacent levees, a tethered barge or boat, or fixed elevated platform.
Maintenance Shafts at: Union Island	Site located on the northwestern corner of Union Island, adjacent to South Bonetti Road, approximately 0.5 mile south of Victoria Canal. Union Island generally slopes from its high point in the southeastern corner to the low point in the northwest. Main flooding potential comes from Old River, Middle River, Victoria Canal, and Grant Line Canal. Evacuation routes could leverage local roads, or unpaved levee roads to South Tracy Boulevard and then to Highway 4. Union Island has an interior levee separating the east and west sides of the island. Pace and depth of flooding from Victoria Canal during major flood event would be rapid, since site is about 0.5 mile from levee. This analysis considered only the western side of the island as it would take significant time before the eastern side could be impacted by a breach on the west side. If a breach were to occur on the eastern side of the island, it could take a considerable time to breach the dry levee and inundate the western side of the island (if at all), but work at the site should be stopped in case of levee breach on the eastern side of the island.	-4.9	0.6	1.4	14.9	H	Assure all weather escape route to levees adjacent to Victoria Canal Provide onsite flood refuge for personnel during pad construction such as elevated evacuation route to adjacent levees, a tethered barge or boat, or fixed elevated platform

Surge basin shaft location is above the projected floodplain at 38 ft; and therefore, not subject to flood inundation
Notes:
H = high
L = low
M = medium

6. Observations and Conclusions

This TM documents factors contributing to historical flooding in the Delta and presents a summary of potential non-structural and structural flood risk management measures that may be employed during and after construction of launch, reception, and maintenance shafts. The key observations and conclusions from this analysis are listed below.

- A combination of non-structural and structural flood risk management measures can be employed to manage the risk of flooding at the launch, reception, and maintenance shafts.
- Non-structural flood risk management includes several measures such as involving appropriate agencies, flood preparedness, emergency response, and post flood recovery operations. These measures would be employed at all shaft sites and other Project infrastructure within the Delta.
- Flood risk for construction personnel and equipment could be substantially reduced by employing non-structural measures such as coordinating and cooperating with levee maintenance agencies; providing SEMS, FEMS and Delta-specific risk and evacuation training for construction personnel; supplying them with individual emergency kits; and providing facilities for sheltering in place which may include elevated evacuation route to adjacent levees, a tethered barge or boat, or fixed elevated platform. Shaft pads may be considered for on-site refuge after they are constructed.
- Shaft pads constructed to a level above the local 100-year WSEL based upon projected conditions in year 2040 would provide a high ground refuge at each shaft location in the event of a flood; but depending on the level of risk, additional structural measures could be considered to improve worker safety during site construction activities.
- Structural flood risk management options include setback levees, ring levees, geometry repairs, and other remediations such as cutoff walls, pressure relief systems, seepage berms, shallow drainage, and stability berms. For the purposes of this evaluation of flood risk mitigations, structural solutions were generally limited to geometry repairs and ring levees. Site-specific design-level analyses should be performed to determine final recommended structural mitigations.
- A flood effect analysis of the potential ring levee at the Twin Cities Complex was performed so that it could be configured to minimize effects to surrounding flood conditions. The current ring levee configuration would minimize potential flooding affects to the surrounding area during construction based upon projected conditions in year 2040.
- Measures to account for shallow, overland flooding at the intakes should be taken into consideration during final design of these structures. These measures may include upgrading drainage infrastructure, working with local reclamation districts and private landowners, and implementing best management practices to address stormwater runoff.
- A Levee Vulnerability Assessment (CER Appendix F2) was performed as a screening level evaluation to identify levee vulnerabilities under current conditions and was used as a tool to identify where potential repairs may be needed. The levee vulnerability assessment does not replace the need for site-specific evaluations.
- For concept-level engineering, Delta-specific PL 84-99 geometry standard was used as the basis to determine levee improvement extents for launch shaft locations on Lower Roberts Island. This is generally consistent with the reclamation districts' flood risk reduction approach. During the design phase, detailed analyses will be conducted to specifically identify the extent of levee improvements, including consideration of sea level rise and climate change assumptions and any levee improvements completed by the reclamation districts prior to construction of the tunnel launch

shafts. Updated sea level rise and climate change assumptions will also be used to update post-construction tunnel shaft heights.

- The Inundation analysis derived a simple, yet conservative, estimate of the time for rising floodwaters to reach each of the shaft sites by locating the spot on the perimeter levee with the greatest potential hydraulic pressure, assuming an instantaneous 500-foot levee breach there, then treating the breach as a broad-crested weir.
- Based on the inundation analysis, it was estimated that floodwaters from a levee breach would take less than 1 hour to affect the launch shaft sites on Lower Roberts Island.
- Floodwaters from a levee breach would take less than 1 hour to affect the reception shafts located at Intake C-E-3 and the maintenance shafts at New Hope Tract, King Island, and Upper Jones Tract.
- For those shaft access locations where floodwaters would take less than 1 hour to reach shaft site ground elevations, the flood depths 1 hour after the start of the levee breach ranged from 0.2 feet to 3.8 feet.

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Attachment 1
Levee Breaches in the North Delta, February 1986

Attachment 1. Levee Breaches in the North Delta, February 1986

In the two weeks prior to February 18, 1986, heavy rains saturated Northern California watersheds and contributed to high inflows into the North Delta from the Cosumnes River, Dry Creek, and the Morrison Stream Group. The inflows exceeded the conveyance capacity of North Delta channels, resulting in ponding upstream of Franklin Road. A series of levee failures ensued:

- **Glanville Tract:** By February 18 the accumulating water backed up on the east side of the Santa Fe Railroad embankment that comprised the east levee of Glanville Tract, breaching this porous structure at multiple points, to flow across Interstate 5 and flood the tract.
- **McCormack-Williamson Tract:** On the afternoon of February 18 the east levee of McCormack Williamson Tract was overtopped. This 1,600-acre island, which has ground elevations close to sea level, filled in about 2 hours, then began spilling from the western end.
- **Dead Horse Island:** This allowed for a large increase in flows into the channels downstream, putting pressure on the levees of Dead Horse Island. Its levees failed and the 200-acre island filled in less than an hour.
- **Tyler Island:** Several hours later, in the early hours of February 19, the resultant high water in the North Fork Mokelumne River overtopped the east levee of Tyler Island at its midsection, resulting in two breaches. Floodwaters poured through these breaches and flowed southwest to the deepest portion of the island. Over the next 24 hours the 8,600-acre island filled and would have flooded Walnut Grove had not a temporary levee been constructed overnight along Walnut Grove-Thornton Road.
- **New Hope Tract:** On February 20, the east levee of New Hope Tract, near Thornton, failed, allowing floodwaters to flow overland, flood the town, breach the Santa Fe Railroad embankment crossing the island, flow through the Interstate 5 underpass at the Walnut Grove-Thornton Road, and flow overland to accumulate in the deepest southwest end of the island. As the island filled, the Mokelumne River gradually subsided, reducing the rate of inflow. The lower end levee was breached to release accumulated floodwater.

Attachment 2
Flood Inundation Analysis

Attachment 2. Flood Inundation Analysis

A preliminary estimate at each shaft pad site for the time to flood has been completed. To determine the inundation characteristics a conservative and simplified approach was used to get a range of flooding times and elevations at each shaft site. This analysis utilized work performed previously by the Delta Risk Management Strategy study team (URS, 2008b) to determine the average levee breach geometry (depth and width)[3]. Based on review of past Delta levee breaches, it was assumed for this study that a 500-foot levee breach would occur instantly, removing the entire levee cross-section down to the levee toe. Head differential, used to compute the flow through the breach, was estimated as the difference between the 100-year water surface elevation (WSEL) (USACE,1992) and the minimum landside toe elevation based on LiDAR (DWR, 2017). As a conservative approach, the location of the largest head differential along the levee perimeter of each island was chosen as the assumed condition. To estimate the flow into the islands, it was assumed that the levee breach acted as a broad-crested weir, for which the flow could be computed based on the broad-crested weir equation:

$$Q=C*L*H^{(3/2)} \quad \textbf{[1]}$$

where

Q is the flow rate in cfs,
C is the weir coefficient in $ft^{(1/2)}/s$,
L is the width of levee breach in feet, and
H is the head differential in feet).

The weir coefficient, C, is in turn a variable, which for this analysis is estimated based on the discharge coefficient, the gravitational constant, and constants based on geometric properties. $C=\frac{2}{3}C_d\sqrt{2g}$

The discharge coefficient (C_d) is generally dependent upon various factors such as approach velocity, approach water depth, water head above the crest of the opening, width of the approach channel, approach channel bottom slope, height from channel bottom to the crest opening, opening width, length of the floodplain (inundation area), side slope, gravitational acceleration, density of fluid, kinematic viscosity, and time. However, for this inundation analysis, a minimum value of 0.2 and a maximum value of 0.4 was used as the discharge coefficient (C_d) based on Estimates of Discharge Coefficient in Levee Breach Under Two Different Approach Flow Types (Lee, 2019).

In applying this equation, the goal was to determine how quickly the island inundated to the point that the floodwaters reached the ground elevation at the shaft pad locations, and how high the water would be at the pad after 1 hour from the initial breach. Subsequently, it was assumed that the island would continue to flood until the interior inundation depth reached the lowest levee crest elevation along the island perimeter, at which point it was assumed that water would begin spilling out as fast as it was coming into the island.

Based on the 2017 LiDAR data, elevation-capacity curves were estimated for all shaft location sites along the Bethany Reservoir Alignment. The elevation-capacity curves were developed using the Levee Maintenance Areas as boundaries for each of the islands or tracts except for at the intakes. The Intakes are all located within the service area of Maintenance Area 9, but there are areas of high ground

[3] In the DRMS analysis 14 breach scour holes remaining from historic levee breaches were measured from aerial photography. Scour holes ranged in width from 176 feet to 1,018 feet, with an assumed average of 500 feet. The scour holes were generally found to be wider than the levee breaches that caused them, hence the 500-foot breach width is a reasonably conservative estimate for the purpose of the current. Analysis.

between each site. An individual capacity curve was developed for each intake. The basin in which Intake 5 sits is the largest and deepest of the two. Using the elevation capacity curves, the storage capacity was estimated at the existing ground elevation at the shaft pads and the minimum levee crest elevation for the perimeter levee of the island. Storage capacity was also estimated at the inundation depth representing 66% of the head differential (between the 100-year WSEL and interior levee toe). From the time of the initial breach until the flooding reaches this elevation it is reasonable to apply the broad-crested weir elevation because it is fairly accurate up to this point (Hamil, 2010). Beyond this threshold the backwater pressure from the filling island gradually reduces the flow rate until the water level inside the island and outside equalize, at which point flow through the breach would stop.

To determine minimum and maximum estimated inundation times at each shaft pad location, bookend values for the weir coefficient[4] (C_d) of 0.4 and 0.2 were used, respectively (Lee, 2019). If floodwaters would affect shaft locations in under an hour, the floodwater depth was also given to highlight the severity of the flooding. Table A2-1, below, shows the minimum (worst-case) and maximum (best-case) estimated time to cause flooding at the pads at launch, reception and maintenance shafts, applying the high and low values of C_d and the minimum and maximum depth of flooding at one hour after breach initiation.

By this simulation scheme, floodwaters pouring through a 500-foot-wide breach on Lower Roberts Island take less than 1 hour to cause flooding at the tunnel launch pad. Similarly, floodwater pouring through levee breaches on Intake C-E-3, or on King Island and Union Island, Upper Jones Tract, and Union Island would take less than 1 hour to overflow the ground elevations near the shaft pads on these islands. The time to inundate the tunnel shaft locations at New Hope Tract, Canal Ranch Tract, Terminous Tract, and Upper Jones Tract would take at least 3 hours, 2 hours, 1.9 hours, and 3.5 hours, respectively.

For levee breach events that result in rising floodwaters taking less than 1 hour to reach the ground elevation of the pads the inundation depths varied from 0.2 feet to 3.8 feet. Table A2-1 below shows the detailed calculations used to estimate the time to inundate the pad, depth at pad at 1 hour after levee breach and maximum depth at pad for all launch, reception and maintenance shaft locations. Figures A2-1 to A2-10 of this attachment shows the time to inundate the pad and time to reach maximum flooding depth for both best-case and worst-case scenarios for all launch, reception, and maintenance shafts.

[4] Minimal time to effect the pad was computed using a maximum discharge coefficient (C_d) of 0.4. Similarly, maximum time to effect the pads were estimated with a minimum discharge coefficient (C_d) of 0.2.

Table A2-1. Detailed Calculations for Inundation Analysis at Launch, Reception and Maintenance Shafts

Site Description	Ground Elevation at Pad Site (ft)	Elevation for 66% of Total Head	Minimum LS Toe Elevation (ft)	100-year WSEL at Min LS Toe (ft)	Lowest Crest Elevation (ft)	X	C(d) (max)	C(d) (min)	C (max)	C (min)	Head, H(1) (ft)	Head, H(D) (ft)	Length of Breach, L (ft)	Max Flow Rate, Q (cfs)	Min Flow Rate, Q (cfs)	Time to Effect Pad, min (hours)	Time to Effect Pad, max (hours)	Maximum Depth at Pad @ 1 hr (ft)	Minimum Depth at Pad @ 1 hr (ft)	Maximum Depth of Flooding (ft)
Launch Shafts at: Twin Cities	10.0	13.2	1.2	19.4	15.2	5.3	0.4	0.2	1.9	0.8	18.2	12.0	500.0	72753.6	31180.1	5.5	12.8	not applicable	not applicable	**5.2**
Launch Shafts at: Lower Roberts Island	-10.0	0.2	-17.9	9.6	10.5	5.3	0.4	0.2	1.9	0.8	27.5	18.2	500.0	135017.1	57864.5	0.9	2.0	**0.2**	not applicable	**20.5**
Reception Shafts at: Intake C-E-3	6	18.7	13.33	21.5	21	5.3	0.4	0.2	1.9	0.8	8.2	5.4	500.0	21863.7	9370.1	0.2	0.5	**3.0**	**0.8**	**15.0**
Reception Shafts at: Terminous Tract	-8.0	1.2	-16.4	10.2	4.2	5.3	0.4	0.2	1.9	0.8	26.6	17.6	500.0	128516.0	55078.3	1.9	4.3	not applicable	not applicable	**12.2**
Reception Shafts at: BRPP Surge Basin	38.0	not applicable	NA	10.3	NA	no	0.4	0.2	1.9	0.8	not applicable	not applicable	not applicable	not applicable	not applicable	not applicable	not applicable	not applicable	not applicable	not applicable
Maintenance Shafts at: Intake C-E-5	6.5	18.7	13.6	21.3	21	5.3	0.4	0.2	1.9	0.8	7.7	5.1	500.0	20004.4	8573.3	2.9	6.8	**4.7**	**6.1**	**14.5**
Maintenance Shafts at: New Hope Tract	5.0	6.3	-5.9	12.6	13.9	5.3	0.4	0.2	1.9	0.8	18.5	12.2	500.0	74377.7	31876.2	3.0	7.1	not applicable	not applicable	**8.9**
Maintenance Shafts at: Canal Ranch Tract	3.0	2.3	-14.9	11.2	7.6	5.3	0.4	0.2	1.9	0.8	26.2	17.3	500.0	125413.5	53748.6	2.0	4.7	not applicable	not applicable	**4.6**
Maintenance Shafts at: King Island	-12.0	1.6	-13.9	9.6	10.5	5.3	0.4	0.2	1.9	0.8	23.5	15.5	500.0	106521.4	45652.0	0.0	0.0	**3.8**	**1.6**	**22.5**
Maintenance Shafts at: Upper Jones Tract	-3.0	2.2	-12.3	9.7	9.8	5.3	0.4	0.2	1.9	0.8	22.0	14.5	500.0	96610.4	41404.4	3.5	8.2	not applicable	not applicable	**12.8**
Maintenance Shafts at: Union Island	-4.9	4.1	-7.3	10.0	14.4	5.3	0.4	0.2	1.9	0.8	17.3	11.4	500.0	67368.8	28872.3	0.6	1.4	0.2	not applicable	**19.3**

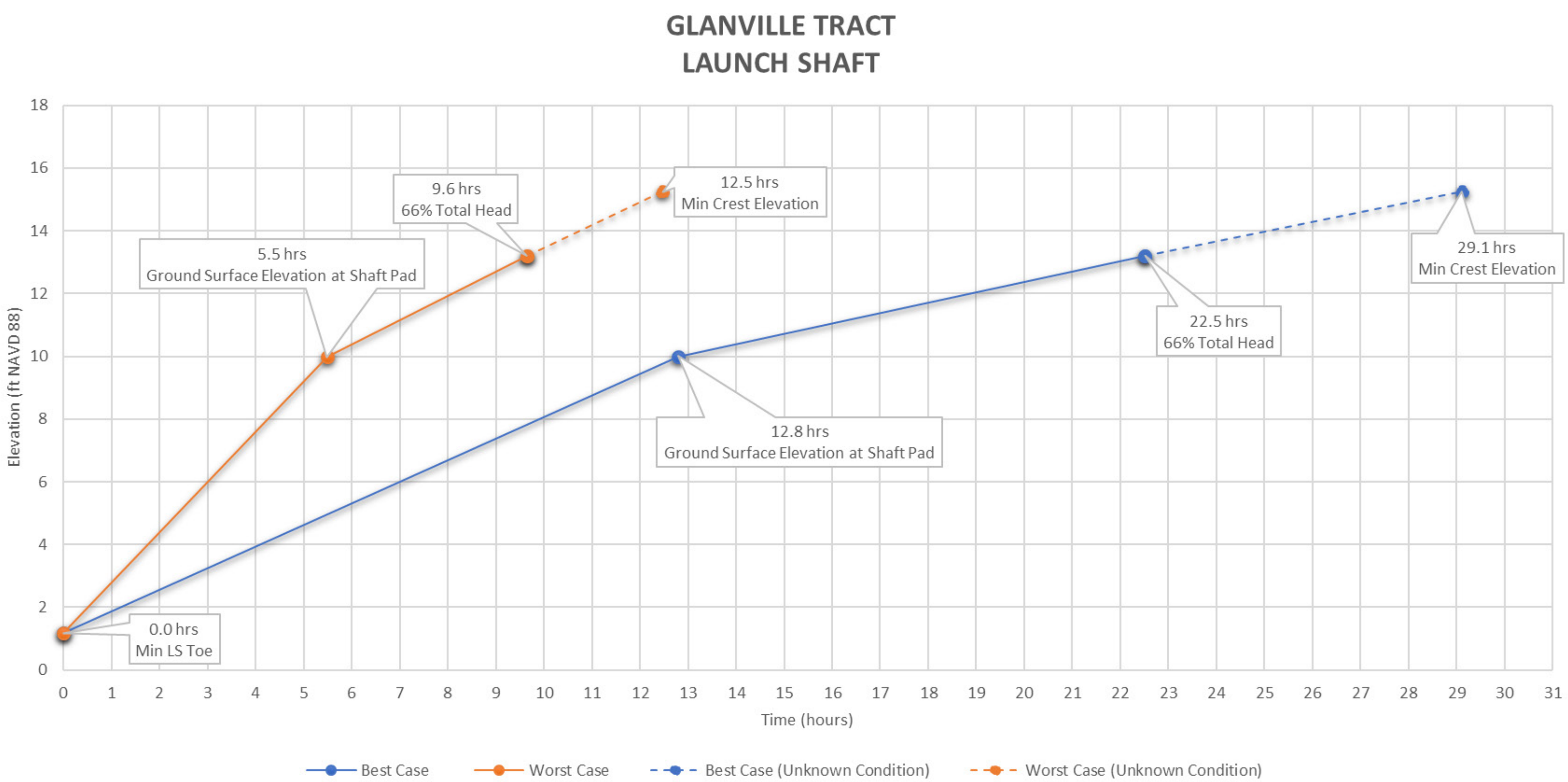


Figure A2-1. Time to inundate pad and time to reach maximum flooding depth for best case and worst-case scenario at Twin Cites (Glanville Tract) Launch Shaft

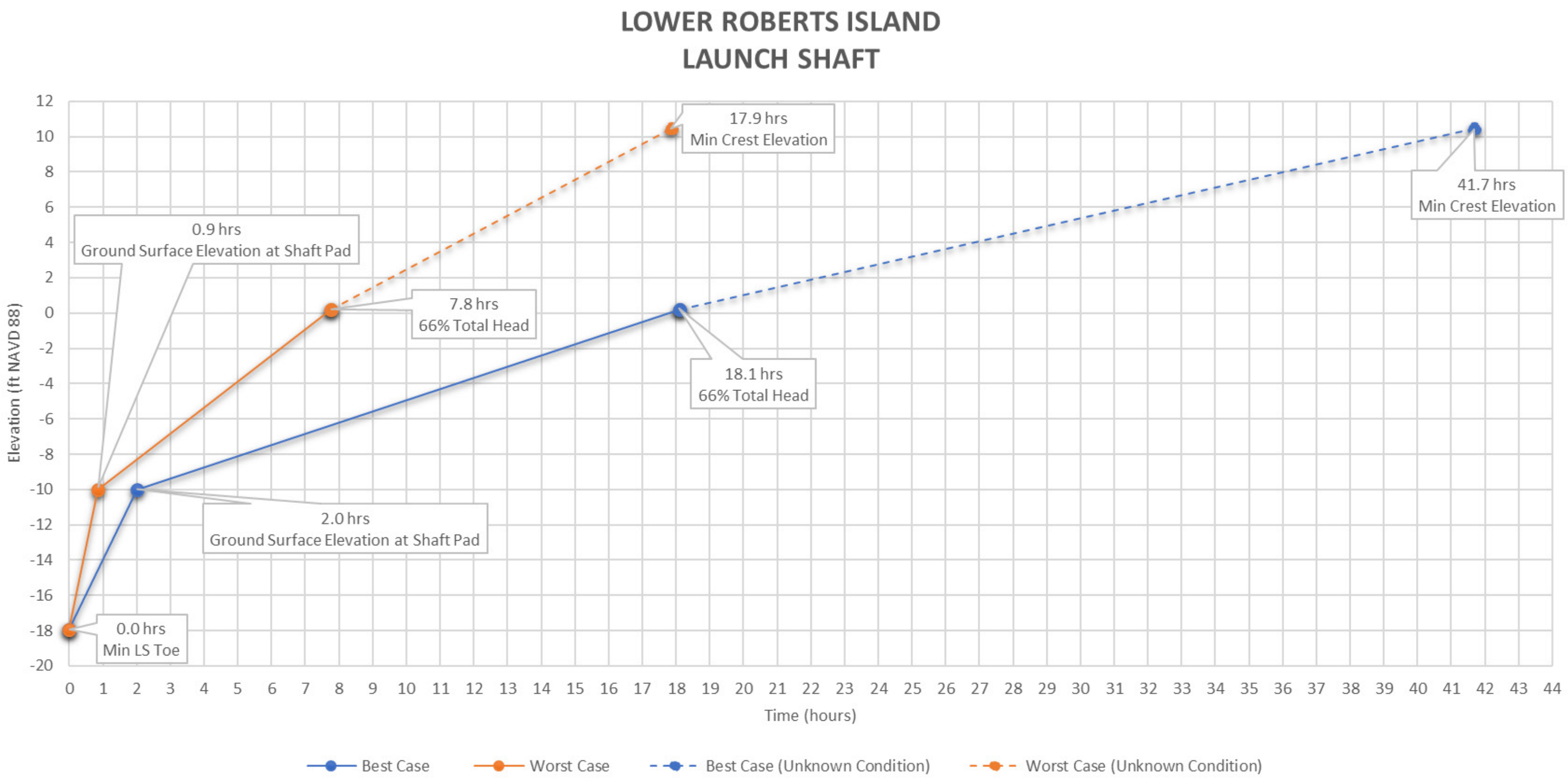


Figure A2-2. Time to inundate pad and time to reach maximum flooding depth for best case and worst-case scenario at Lower Roberts Island Shaft

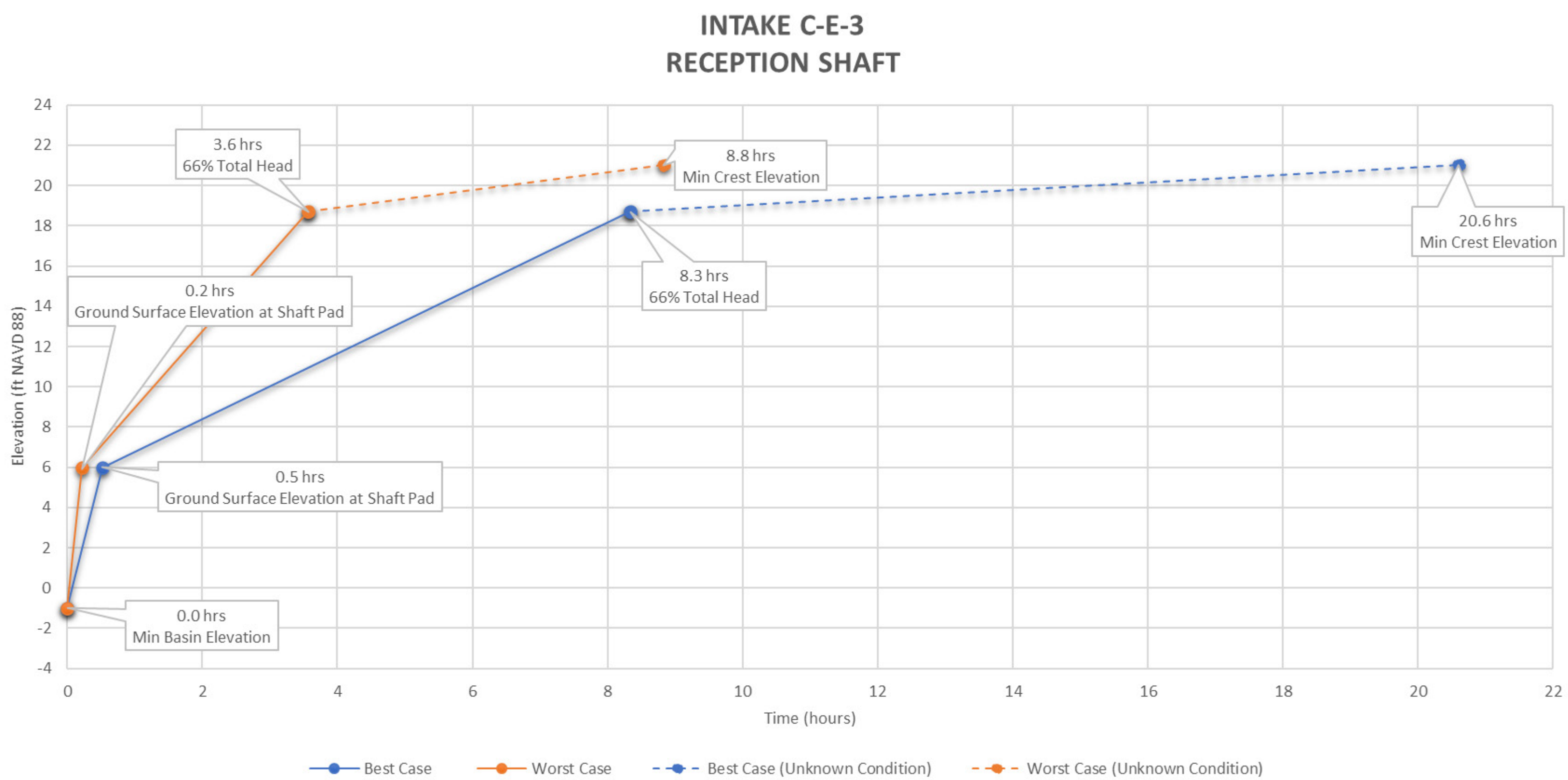


Figure A2-3. Time to inundate pad and time to reach maximum flooding depth for best case and worst-case scenario at Intake C-E-3 Reception Shaft

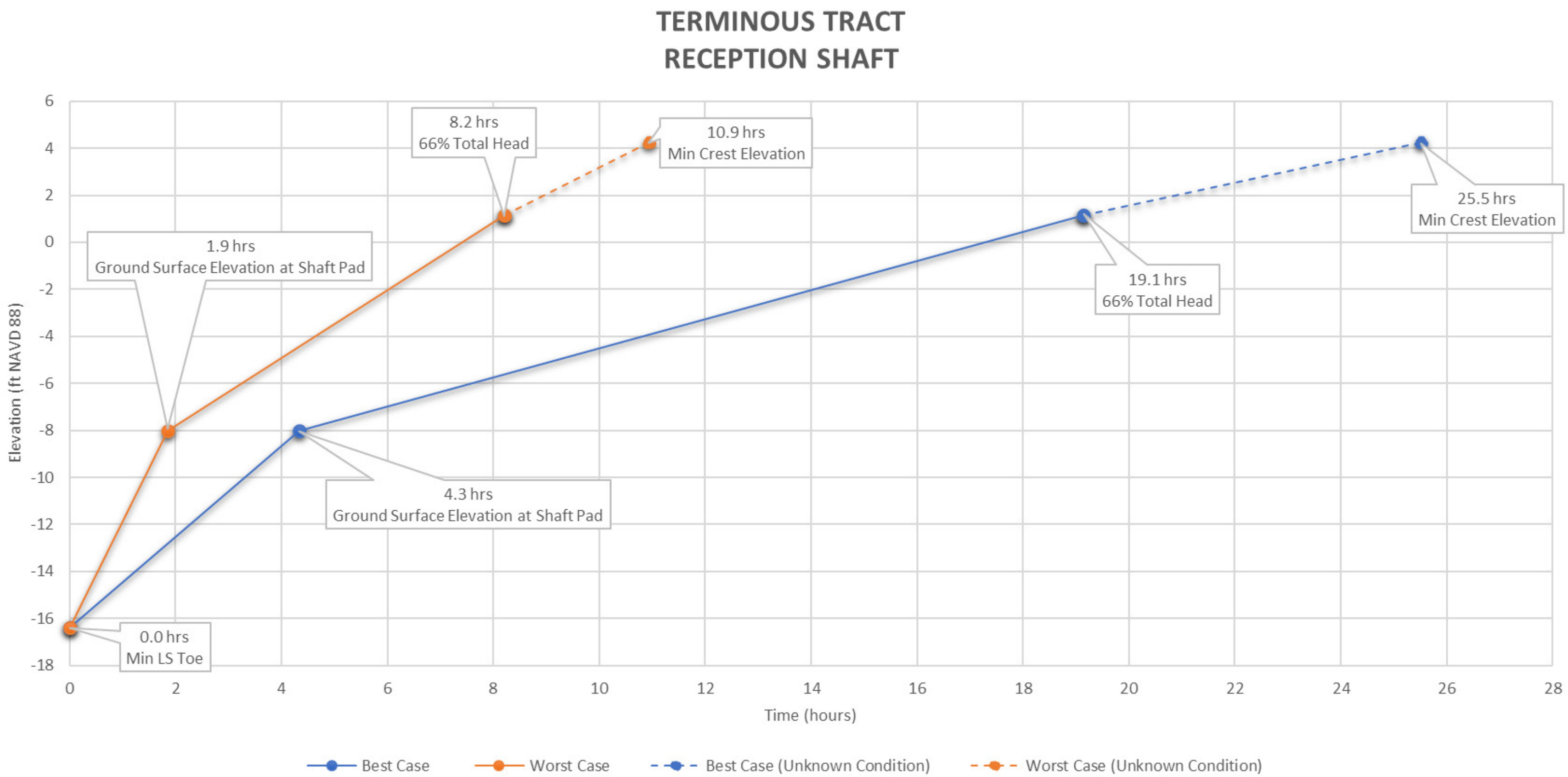


Figure A2-4. Time to inundate pad and time to reach maximum flooding depth for best case and worst-case scenario at Terminous Tract Reception Shaft

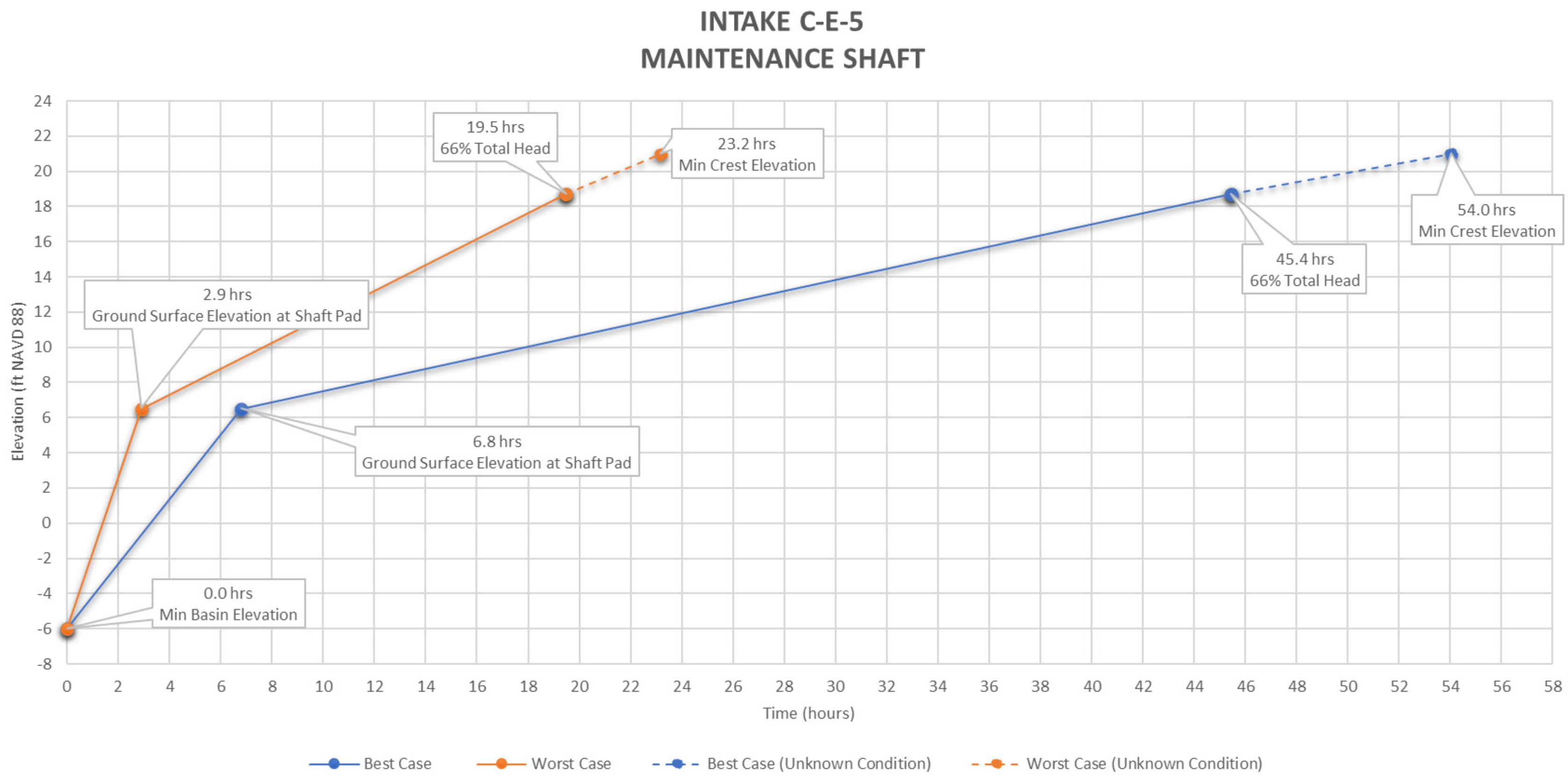


Figure A2-5. Time to inundate pad and time to reach maximum flooding depth for best case and worst-case scenario at Intake C-E-5 Maintenance Shaft

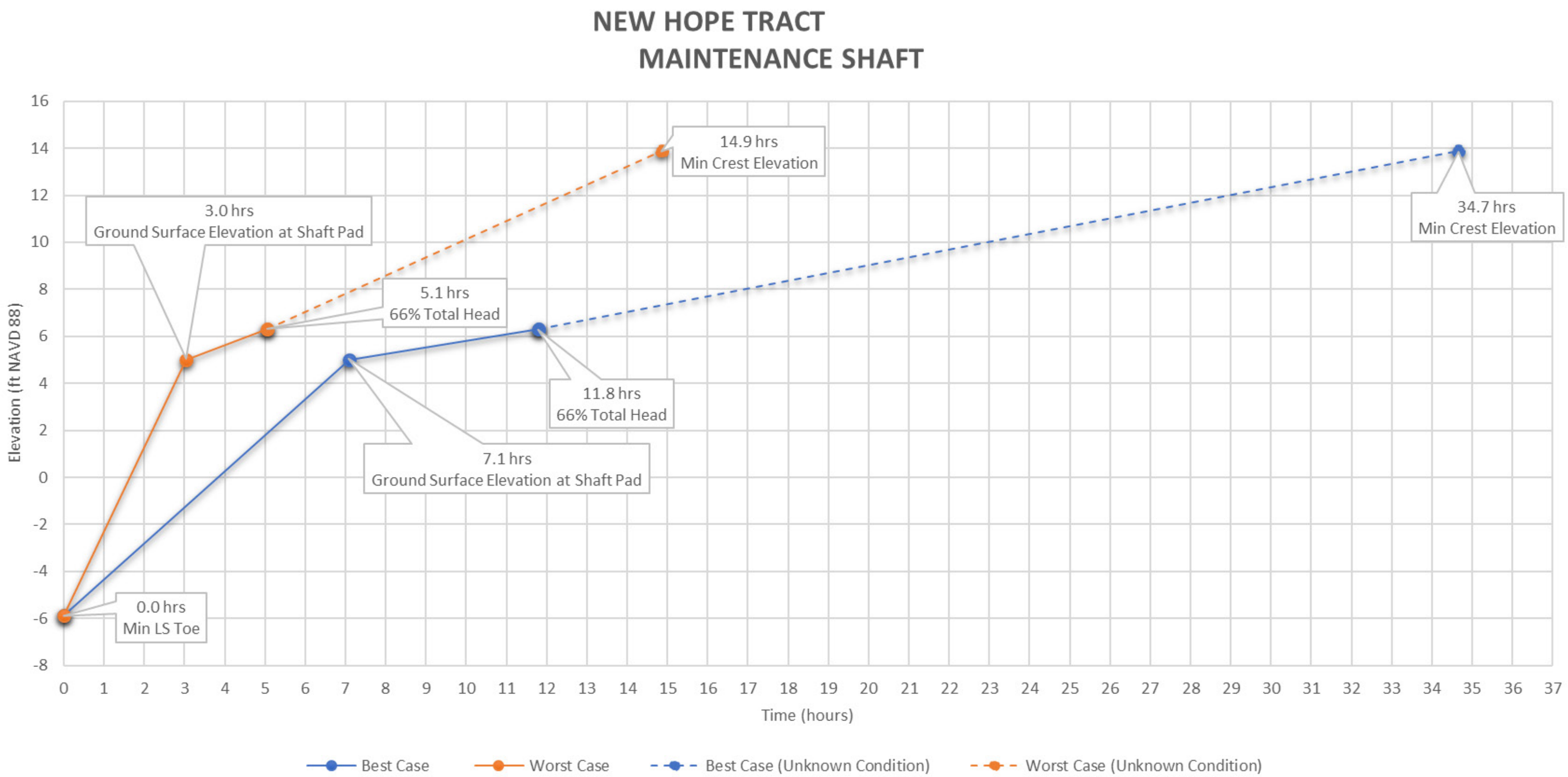


Figure A2-6. Time to inundate pad and time to reach maximum flooding depth for best case and worst-case scenario at New Hope Tract Maintenance Shaft

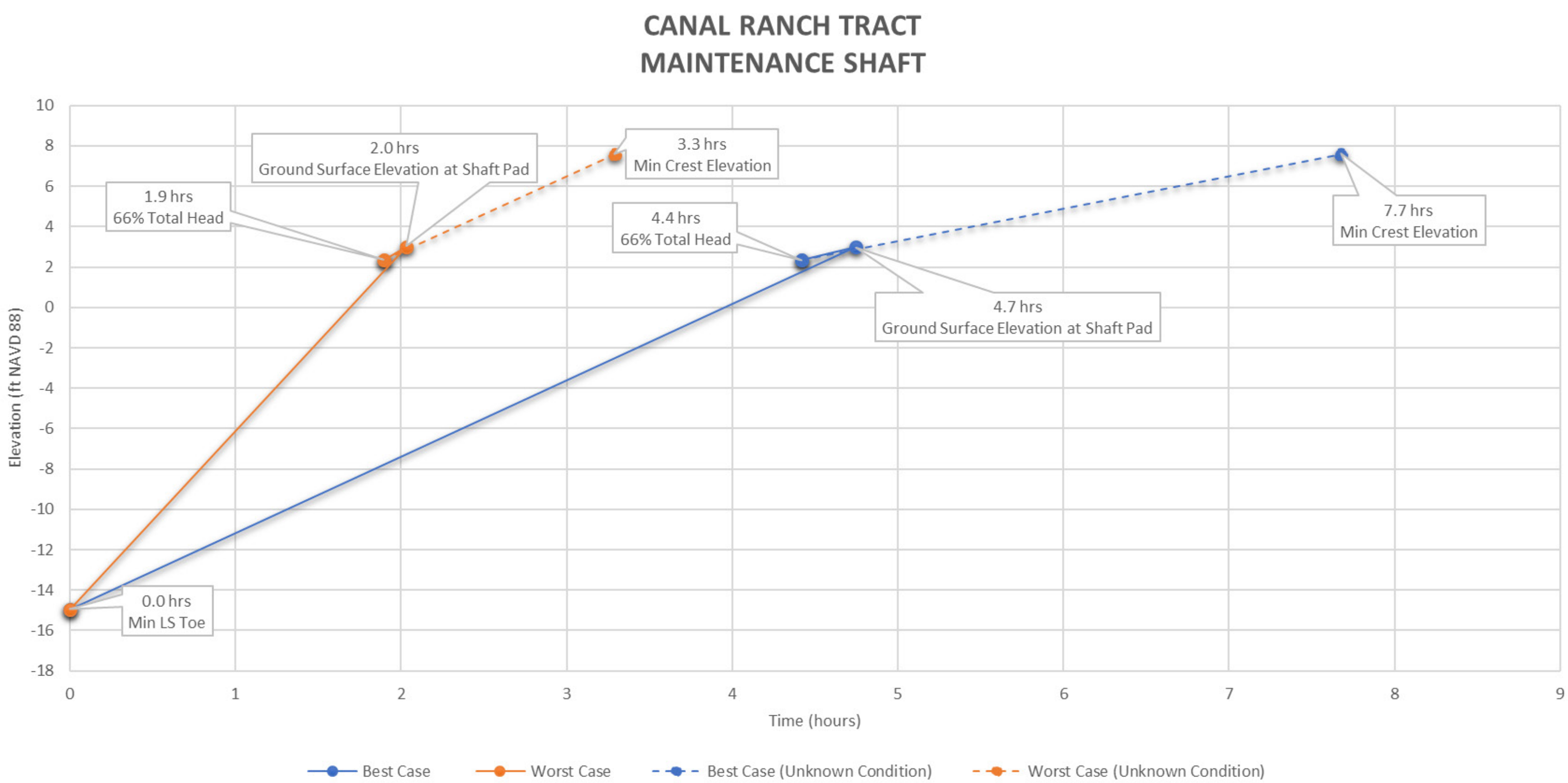


Figure A2-7. Time to inundate pad and time to reach maximum flooding depth for best case and worst-case scenario at Canal Ranch Tract Maintenance Shaft

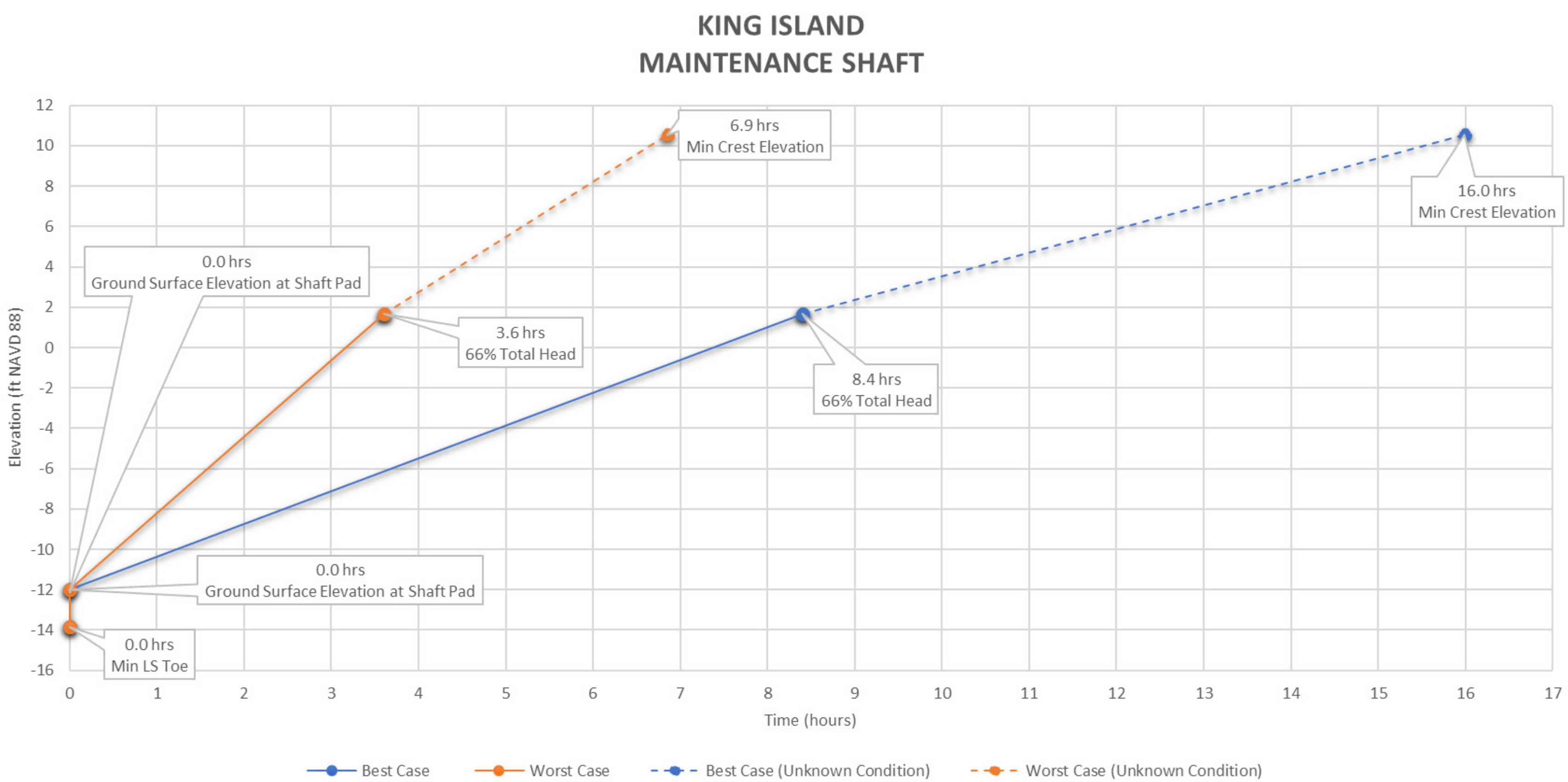


Figure A2-8. Time to inundate pad and time to reach maximum flooding depth for best case and worst-case scenario at King Island Maintenance Shaft

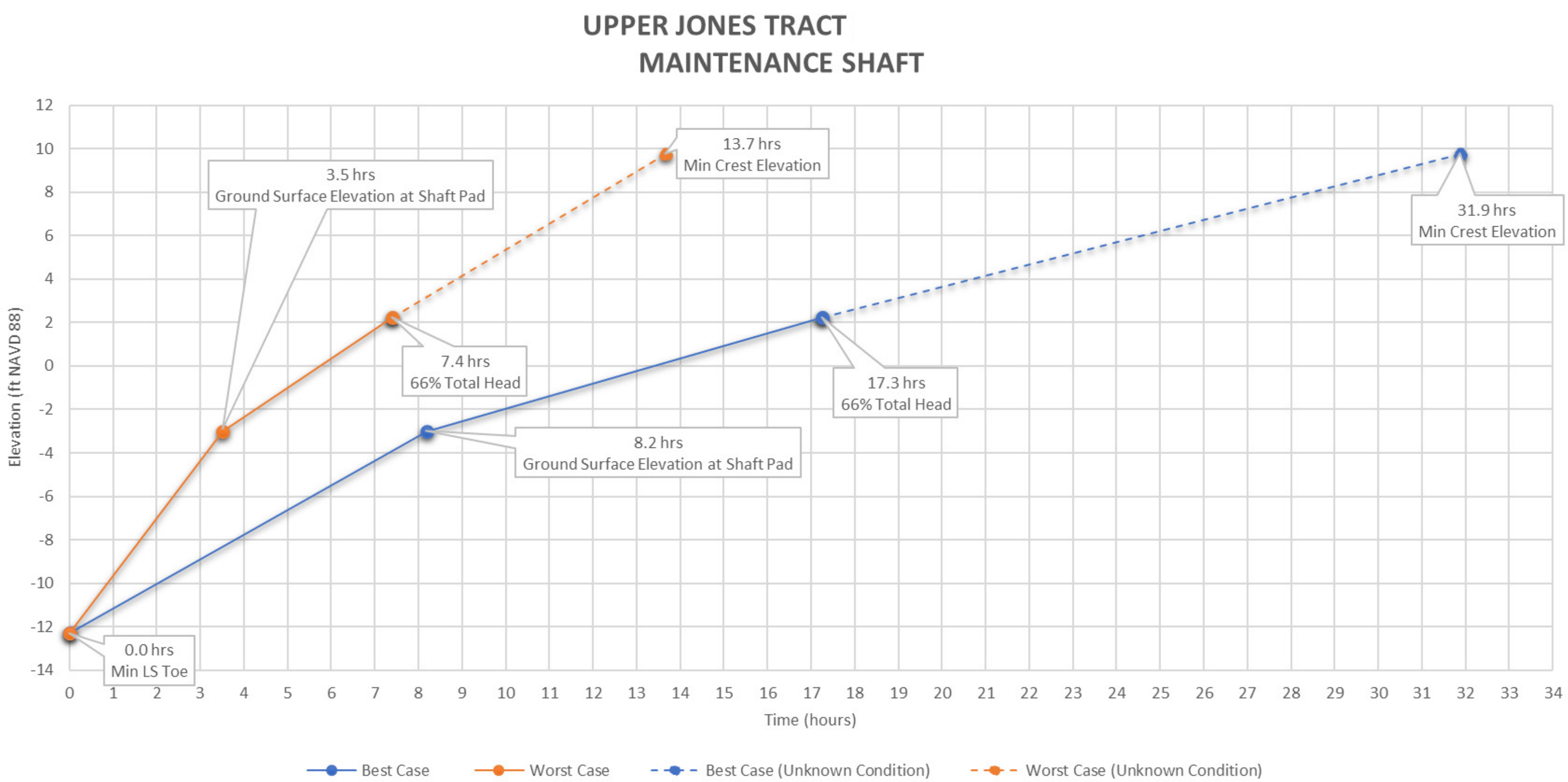


Figure A2-9. Time to inundate pad and time to reach maximum flooding depth for best case and worst-case scenario at Upper Jones Tract Maintenance Shaft

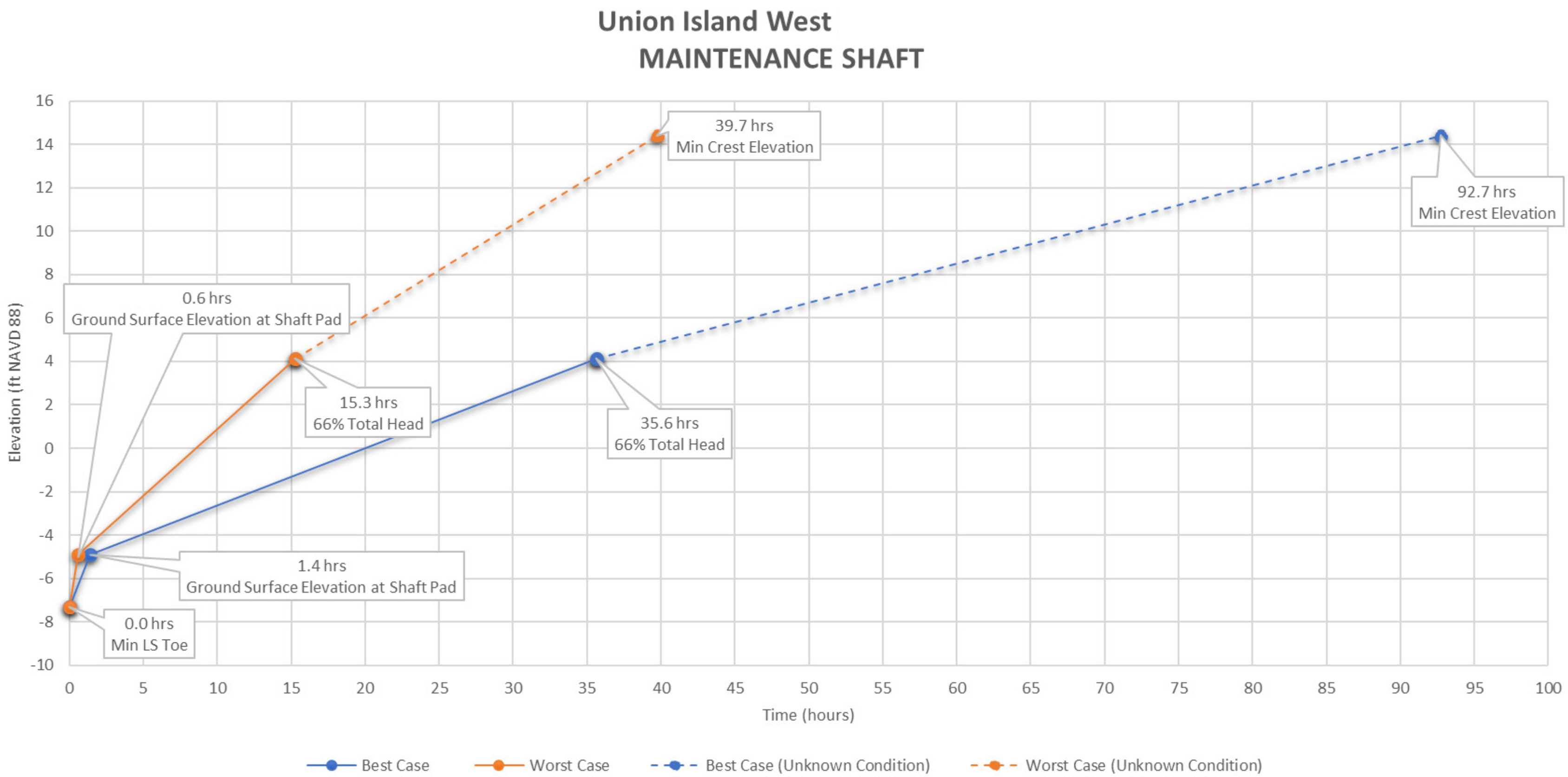


Figure A2-10. Time to inundate pad and time to reach maximum flooding depth for best case and worst-case scenario at Union Island Maintenance Shaft

Attachment 3
Twin Cities Complex Flood Analysis

Attachment 3. Twin Cities Complex Site Flood Analysis

The purpose of Attachment 3 is to present the 100-year storm frequency flood results that would be due to the construction of a ring levee to protect the Twin Cities Complex site. The Project ring levee location at the Twin Cities Complex, as shown on Figure A3-1, would be bounded by Interstate 5, Twin Cities Road, Lambert Road and Franklin Blvd. The ring levee provides flood protection to the Twin Cities complex in the event of a levee failure on Glanville Tract. More details related to the location of the ring levee are presented in the conceptual engineering drawings. The pre- and post-Project hydraulic model evaluation was based on the Sacramento County North Delta hydraulic model described below. The hydraulic model scenario evaluation used the United States Corps of Engineers Hydrologic Engineering Center River Analysis program HEC-RAS version 5.0.7.

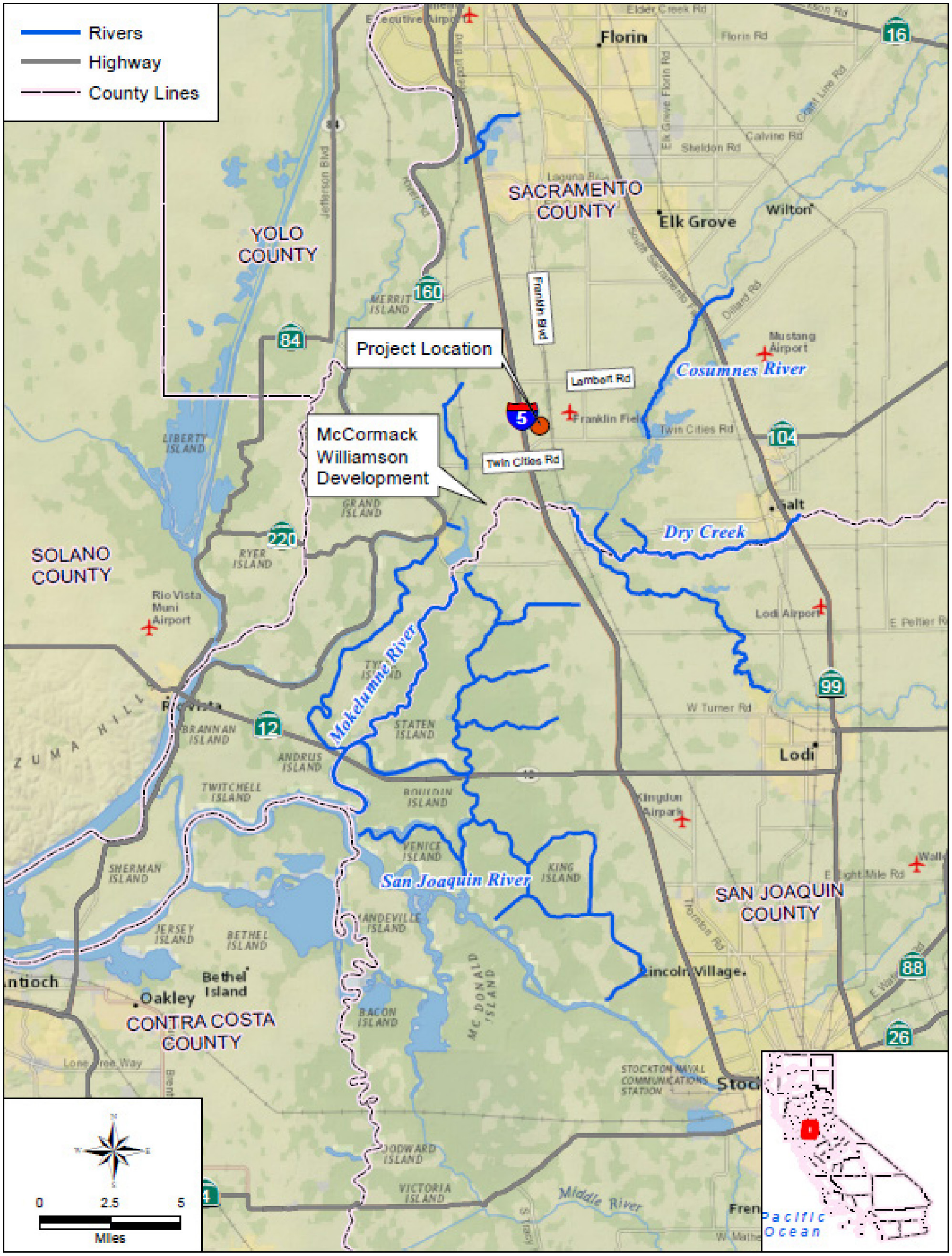


Figure A3-1. Location of Ring Levee at the Twin Cities Complex

3.1 Background

Due to the unregulated Cosumnes River 724 square mile (sq.mi.) watershed, limited Mokelumne River channel conveyance, and tidal conditions downstream of the Cosumnes River, the Glanville Tract and the area around the Project area has a history of flooding. The Mokelumne River watershed combined with the Cosumnes River watershed at the Project location is 2188 sq. mi. Flows in the Mokulumne River are regulated through the Pardee and Camanche Reservoirs. The combined Mokelumne and Cosumnes River watershed is presented on Figure A3-2.

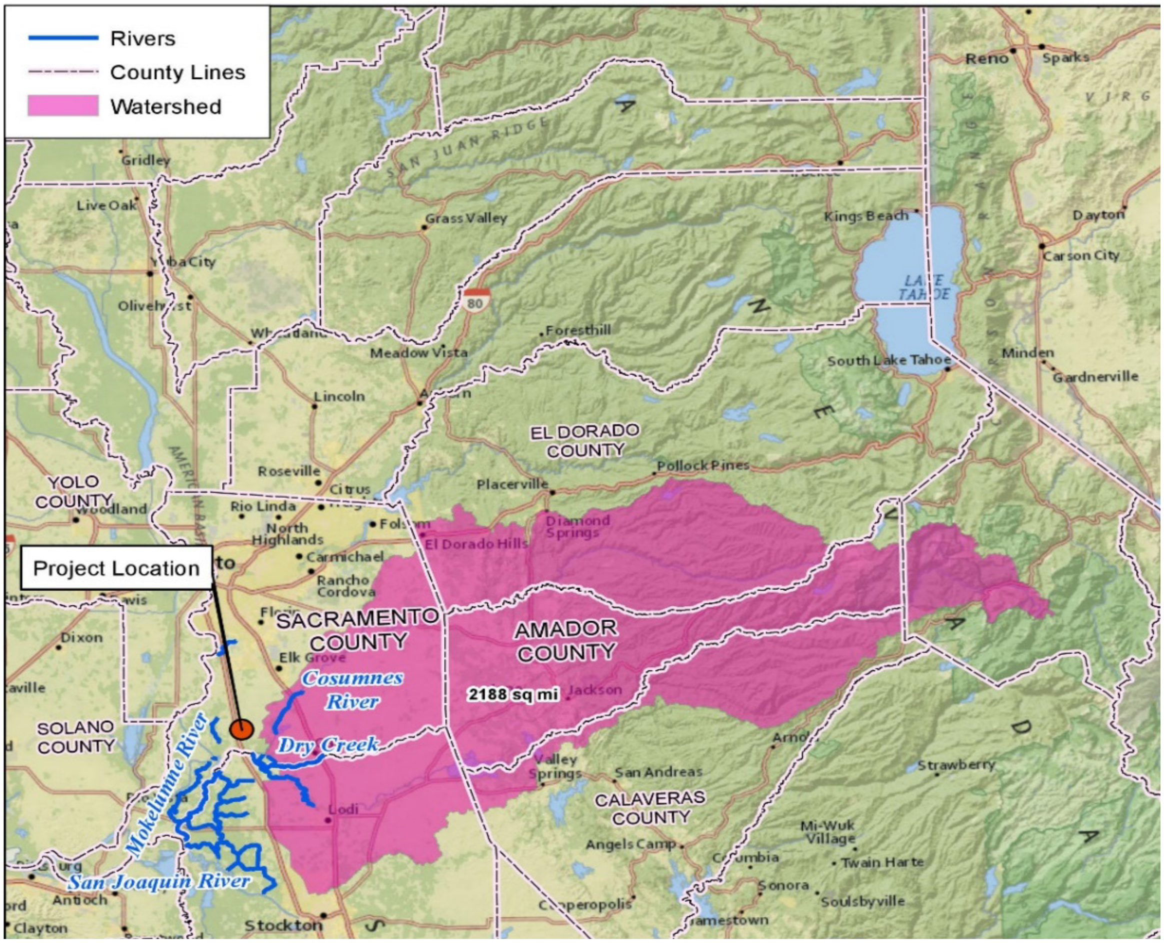


Figure A3-2. Cosumnes River and Mokelumne River Watersheds

3.2 Topography

A seamless terrestrial-bathymetric digital terrain was created from a variety of data sources to develop the coupled 1D/2D HEC-RAS model (CBEC, 2020). Primary model geometry was derived from the 2007 Delta Light detection and ranging (LiDAR) topography and bathymetry collected by DWR between 2007 and 2016 using single and multi-beam surveys (DWR, 2007, 2016). Supplementary bathymetric data was used from the 2011 DWR multi-beam surveys for the North and South Mokelumne Rivers, and short portions on the downstream ends of Snodgrass Slough, Dead Horse Cut, and the Mokelumne River. Nearest-neighbor interpolation was used to fill data gaps which occur at the channel margins where the LiDAR and multi-beam surveys were unable to resolve the topography, due to shallow water (DWR, 2012). Breaklines, obtained with the CVFED LiDAR dataset, were used to define channel edges, in-channel islands, and ditches. Local levees were analyzed for topographic anomalies (e.g. artificially high levee heights), created by dense vegetation, where true bare earth LiDAR returns were not obtained. The North Delta hydraulic model geometry shown in Figure A3-3 and model results are presented on North American Vertical Datum of 1988 (NAVD 88).

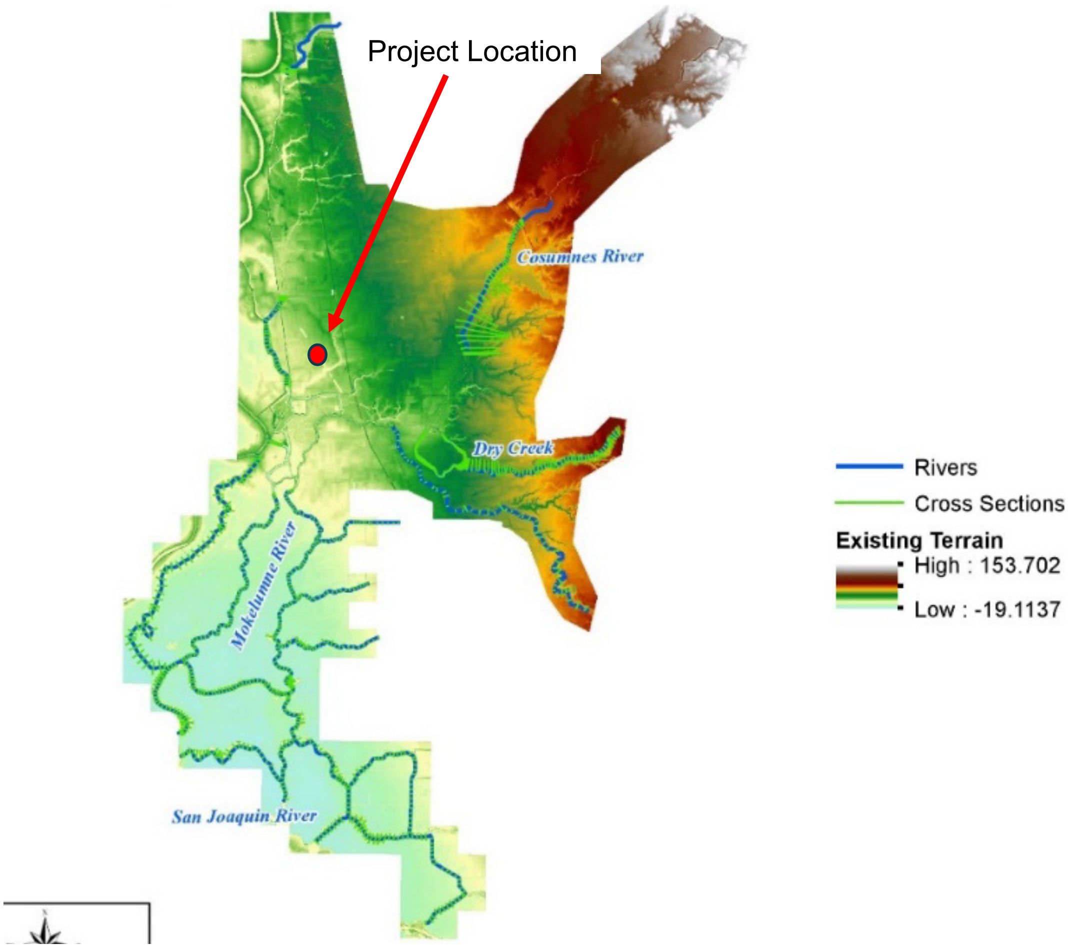


Figure A3-3. Model Terrain

3.3 Hydrologic and Hydraulic Model Development

The North Delta hydraulic model was created for Sacramento County to evaluate flood extents and the flood depth in the surrounding areas and on the local Delta levees. Recently, the North Delta hydraulic model was used to evaluate the McCormack-Williamson Tract Levee Modification Project which is adjacent to the Mokelumne River. After the McCormack-Williamson Tract Project is completed, the hydraulic profile would be reduced approximately 1-1.5 ft. within the adjacent floodway, which reduces the likelihood of flooding within Glanville Tract which includes the Twin Cities Complex. However, at the time of this TM, the McCormack-Williamson Tract Project completion schedule has not been identified; therefore, the proposed McCormack-Williamson Tract Project improvements to lower water surface elevations was not included in the hydraulic model evaluation. The major effect from not including the McCormack-Williamson Tract Project with this evaluation is that flood waters appear to overtop the existing railroad embankment on the southeast side of Glanville Tract leading to shallow overland flooding in the vicinity of the ring levee location at the Twin Cities Complex. Overtopping of the railroad embankment does not appear to occur in the model after the McCormack-Williamson Tract Project is in place.

The North Delta hydraulic model extent and boundary condition hydrograph locations are shown on Figure A3-4. Table A3-1 presents the 100-year peak flows corresponding to the node locations utilized for this evaluation. The 100-year storm frequency hydrology was based on the rainfall-runoff models described in the report “Cosumnes and Mokelumne River watersheds – Design storm runoff analysis” prepared for Sacramento County Department of Water Resources (David Ford Consulting Engineers, February 6, 2004). The hydrologic models developed by Ford Consulting validated the hydrologic models based on historic precipitation and snow gage data. Statistical analysis to develop the theoretical the 100-year 10-day storm hydrographs in the hydraulic model were extended to a 20-day duration to fully route the upstream hydrographs and downstream tidal through the 1D/2D hydraulic model. The downstream tidal boundary conditions were based on historical gage data and there were no adjustments to account for sea level rise or climate change. Background on the hydraulic model calibration and validation based on the 1997, 2006 and 2017 storm events are presented in the report “McCormack-Williamson Tract Levee Modification and Habitat Development Project” (CBEC, 2020).

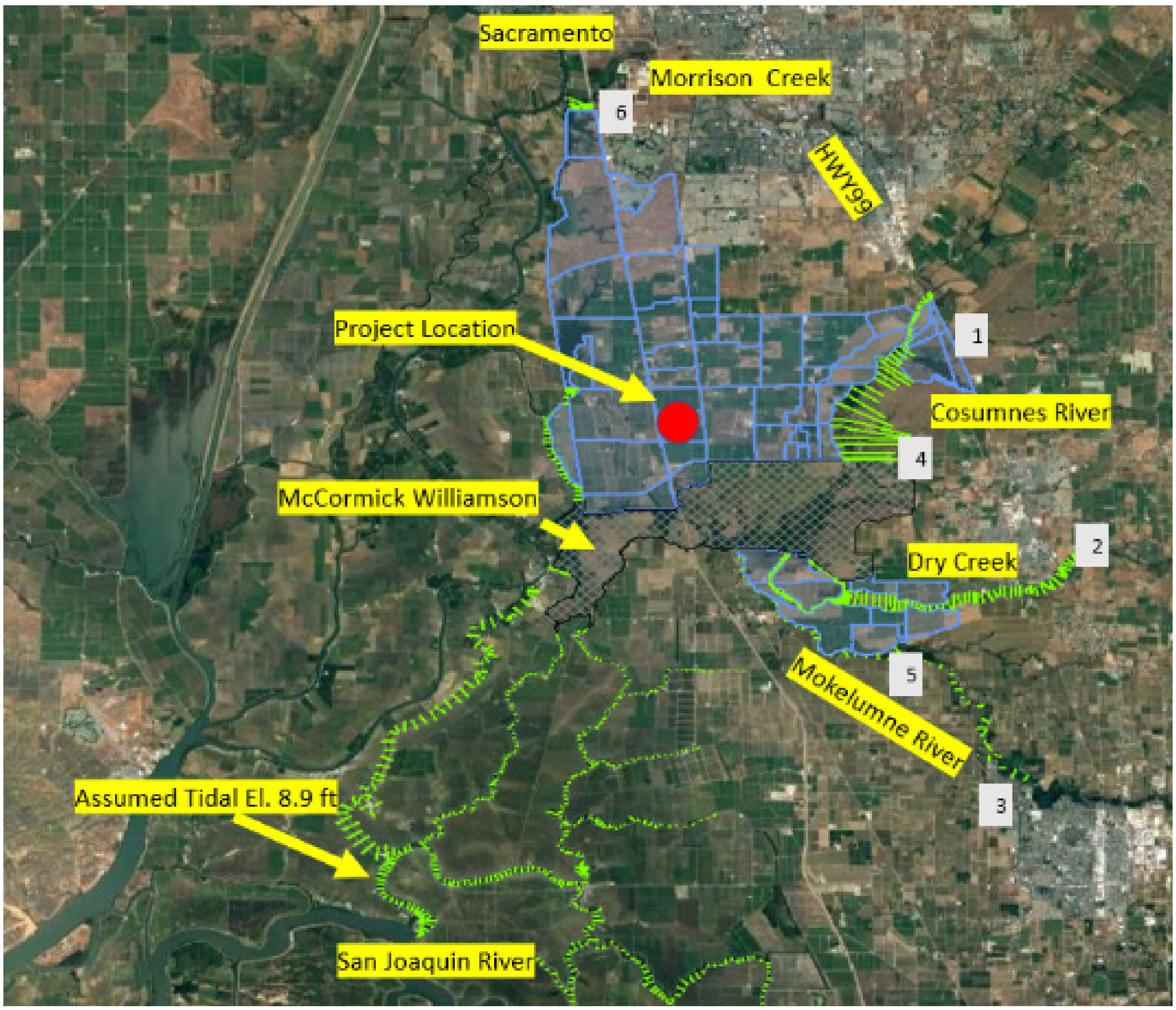


Figure A3-4: North Delta Model Domain and Hydrograph Boundary Condition Locations

Note: The green lines represent model cross sections, the blue lines represent the model 2-d computational mesh. Numbers 1-6 represent the upstream boundary hydrographs routed through the hydraulic model.

Table A3-1. Cosumnes River System 100-Year Flows

Hydrograph Boundary Condition Location	Location Description	Estimated 100-Year Peak Flow (cfs)
1	Cosumnes River at Highway 99	26,531
2	Dry Creek at Galt	30,985
3	Mokelumne River at Woodbridge	5,000
4	Local Cosumnes River watershed inflows below Highway 99	14,000
5	Local Mokelumne River watershed inflows	12,930
6	Morrison Creek at UPR	10,631

3.3.1 Hydraulic Model Scenarios

The North Delta hydraulic model was modified to include two conditions described below for comparison to existing conditions. The resulting findings are provided in this TM.

- **Scenario 1)** Hydraulic model Scenario 1, shown on Figure A3-5, presents Bethany Reservoir Alignment (6,000 cfs Project design capacity) temporary ring levee alignment in red. The temporary levees would be constructed to Elev. 20.5 ft around the north, west, and south sides, and 21.0 ft on the west side to prevent flood water from entering the construction site.
- **Scenario 2)** Hydraulic model Scenario 2, shown on Figure A3-6, shows the permanent stockpile for the Bethany Reservoir Alignment (6,000 cfs Project design capacity) once the ring levee has been removed. The stockpile would be elevated above the existing 100-year floodplain.

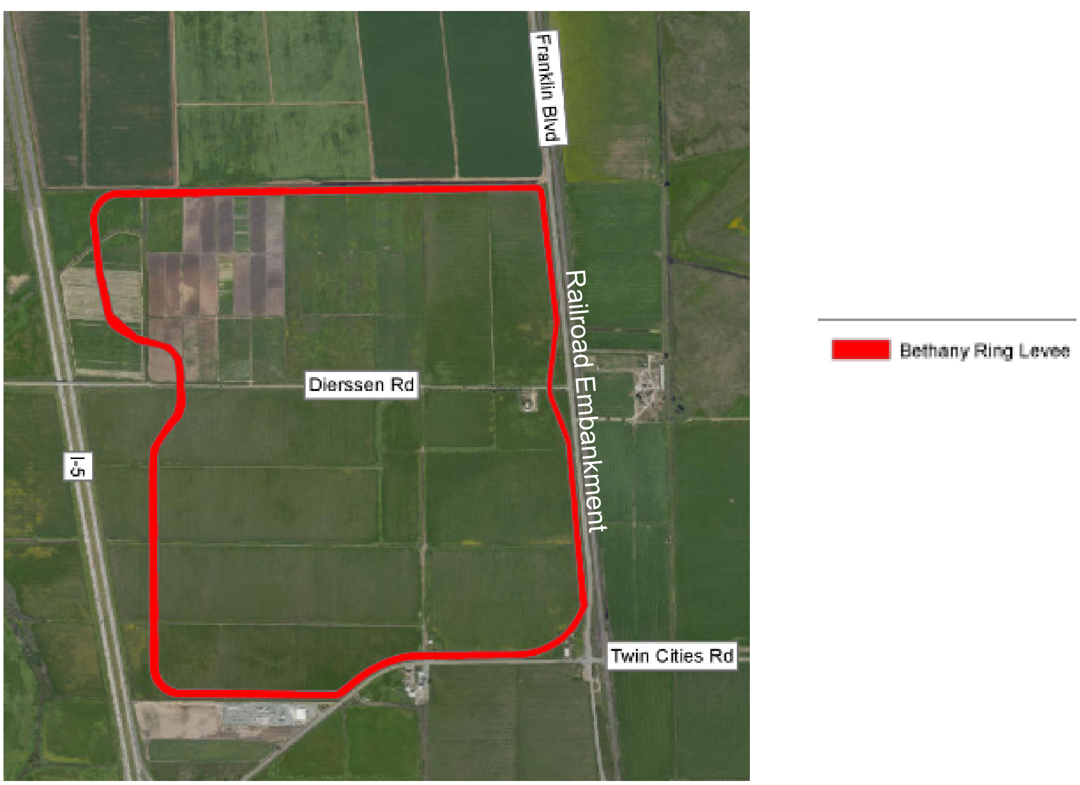


Figure A3-5. Scenario 1: Ring Levee at the Twin Cities Complex Site

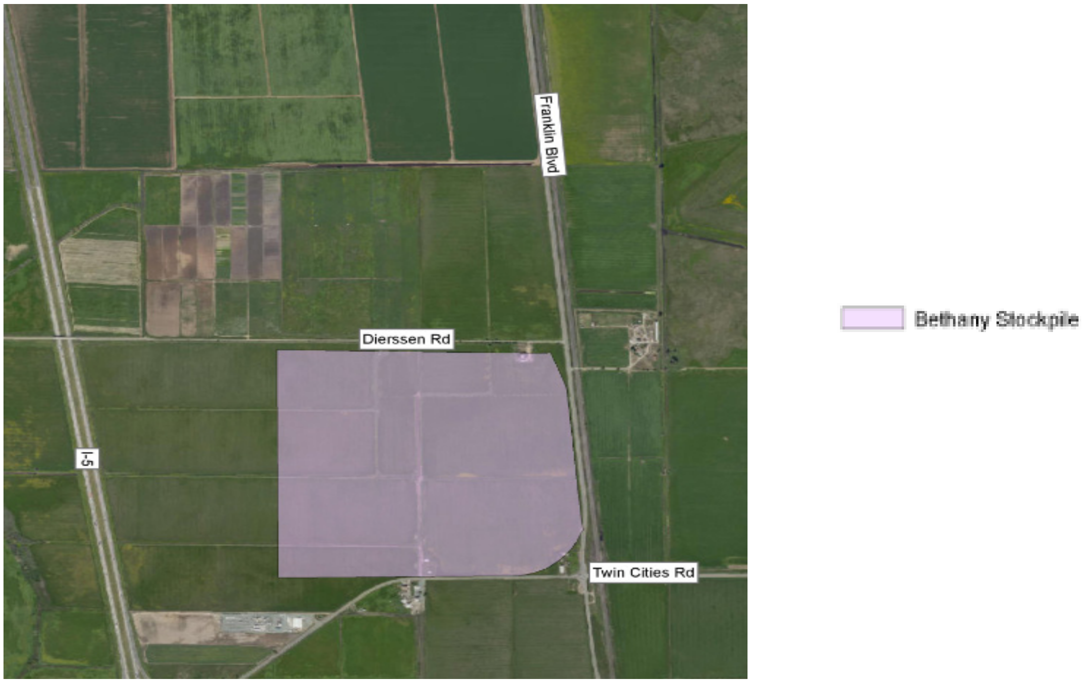


Figure A3-6. Scenario 2: Permanent Stockpile at the Twin Cities Complex Site

3.4 Hydraulic Model Findings

The following results comparing conditions with Scenarios 1 and 2 are based on the 20-day storm duration.

3.4.1 Existing Conditions Results

The existing condition 100-year floodplain is presented on Figure A3-7. The existing 100-year flood depths reach up to 15 ft in the river channel but within the Project area flood depths range from 1 to 3 ft. The large floodplain as shown on Figure A3-7 is the result of limited Mokelumne River channel conveyance adjacent to the McCormack-Williamson Tract Project site as shown on Figure A3-8a. The limited channel capacity causes flood water to backwater up through Snodgrass Slough and overtop the existing railroad embankment on the east side of Glanville Tract leading to inundation of local channels adjacent to Interstate 5, Lambert Road, Twin Cities Road, and Dierssen Road which passes through the center of Twin Cities Complex. Figure A3-8b provides a cross section of the width of McCormack-Williamson Tract compared to the Mokelumne River which has limited channel conveyance (shown on the right). Figure A3-8c presents the routed 100-year flow hydrograph just upstream of McCormack-Williamson Tract reaching a peak flow of approximately 63,000 cubic feet per second (cfs). The downstream Mokelumne River was estimated to be approximately half the required channel capacity.

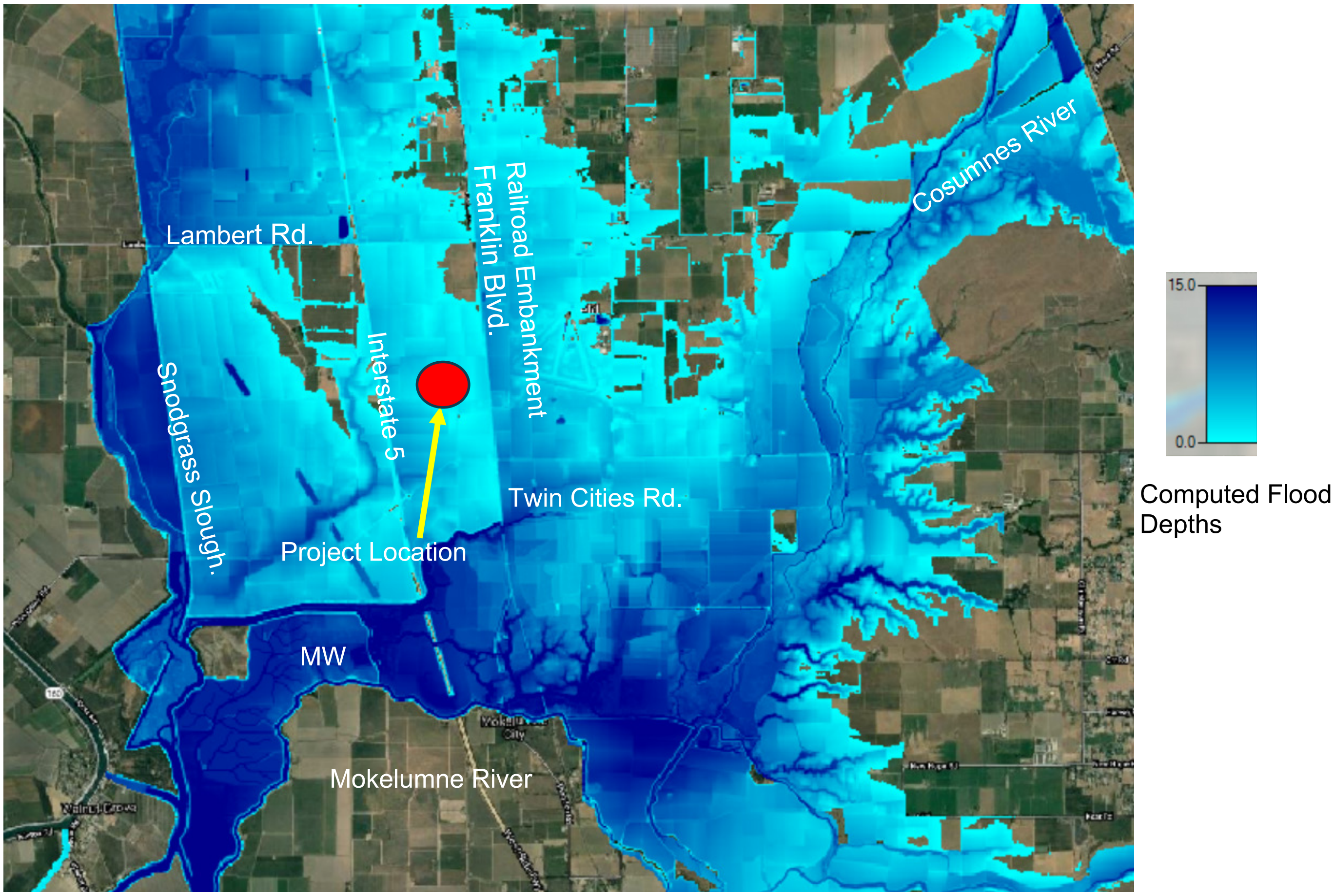


Figure A3-7. Existing 100-Year Floodplain near Twin Cities Complex Site

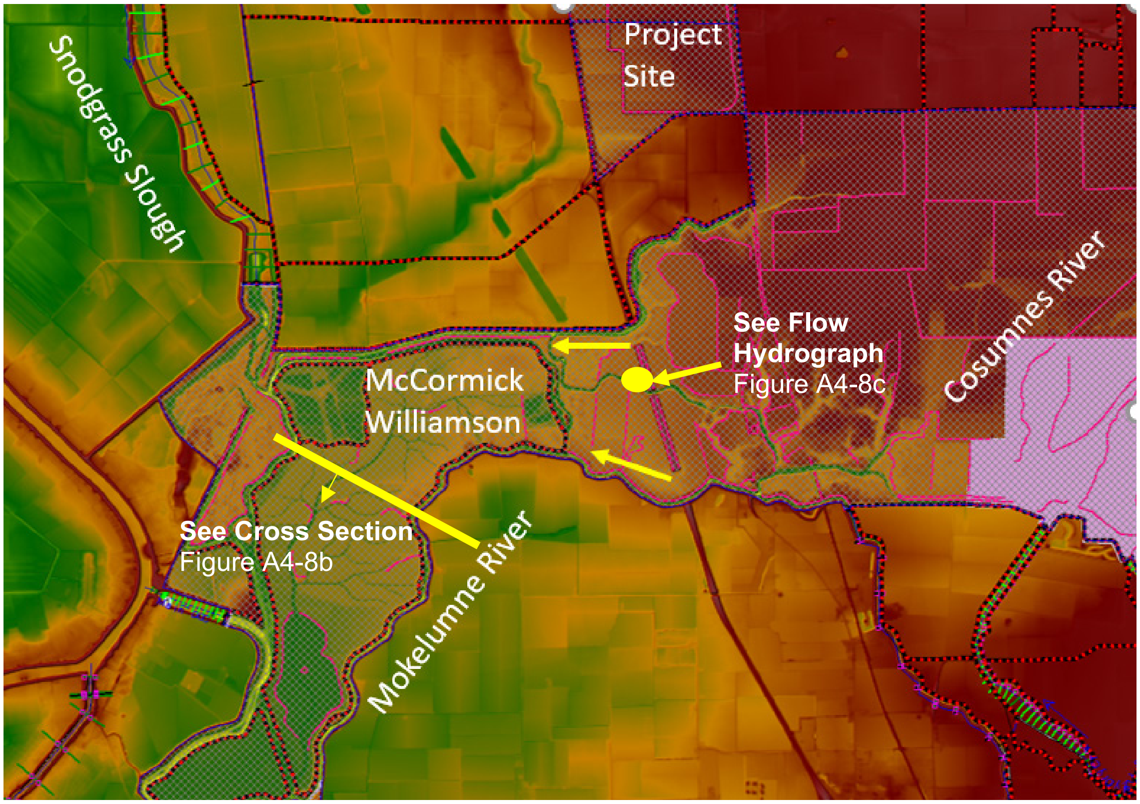


Figure A3-8a. Mokelumne River Channels located to the south of the Twin Cities Complex Site

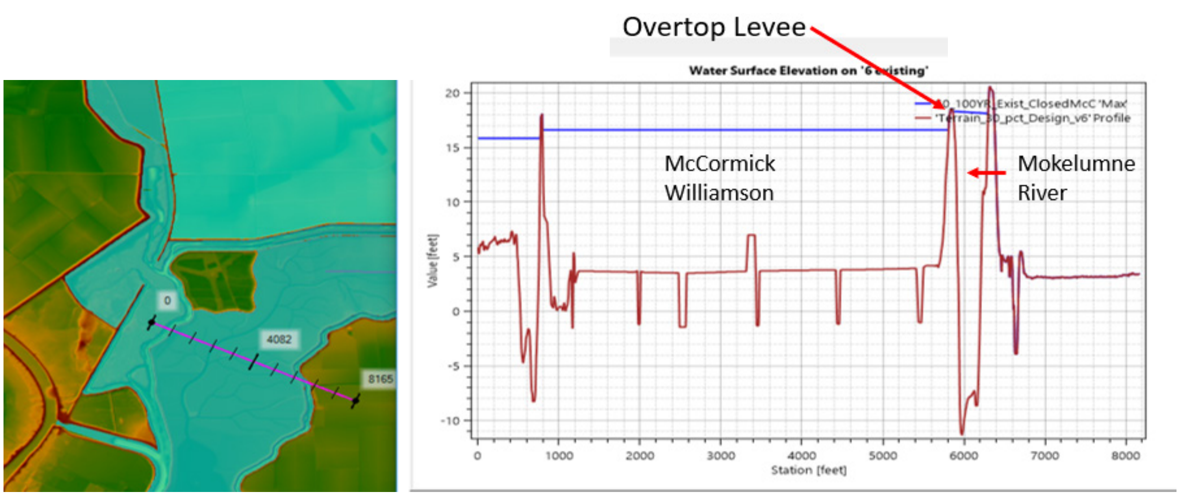


Figure A3-8b. McCormack-Williamson Tract Topographic Cross Section

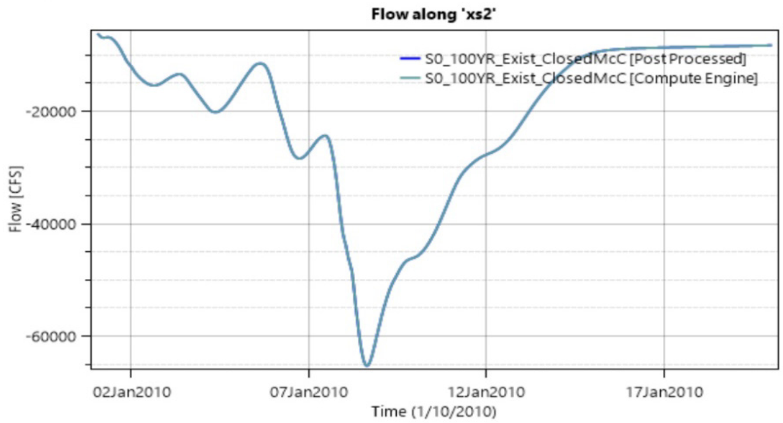


Figure A3-8c. Mokelumne River Hydrograph upstream of McCormack-Williamson Tract

The following shows the existing roadways effected by the 100-year floodplain without the ring levee. Figure A3-9a shows the 100-year floodplain would overtop the north bound lane of Interstate 5, Figure A3-9b shows the 100-year floodplain would overtop Franklin Boulevard and Figure A3-9c shows Dierssen Road would overtop because the road profile is located at existing grade.

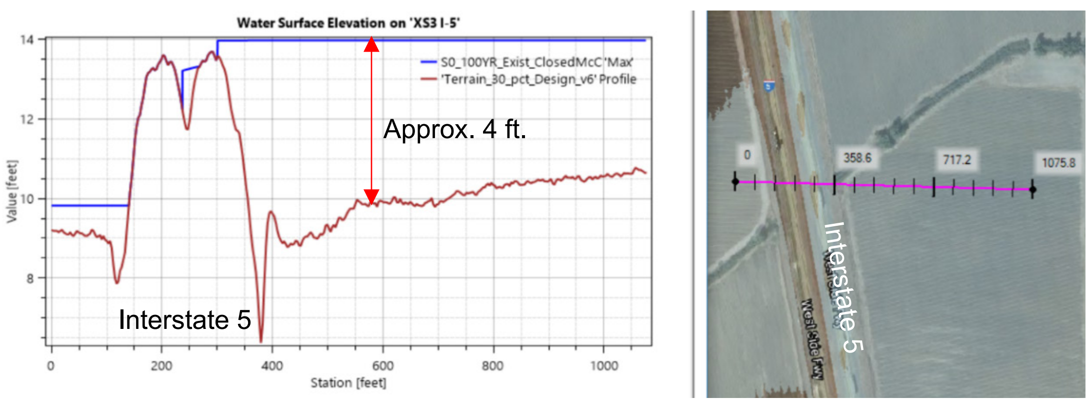


Figure A3-9a. 100-Year Flood Depths under Existing Conditions Hydraulic Model at Interstate 5 North of Dierssen Road Intersection

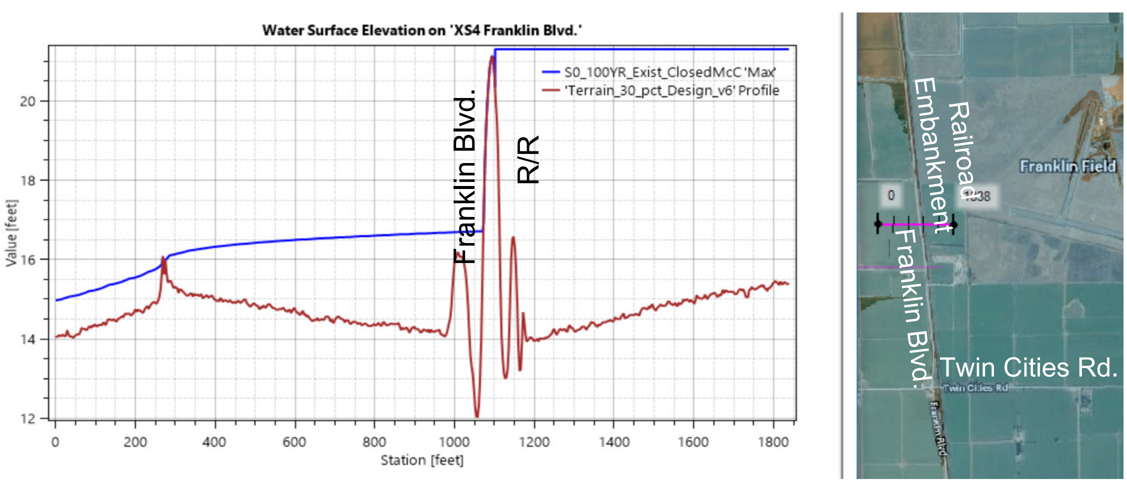


Figure A3-9b. 100-Year Flood Depths under Existing Conditions Hydraulic Model at Franklin Boulevard North of Dierssen Road

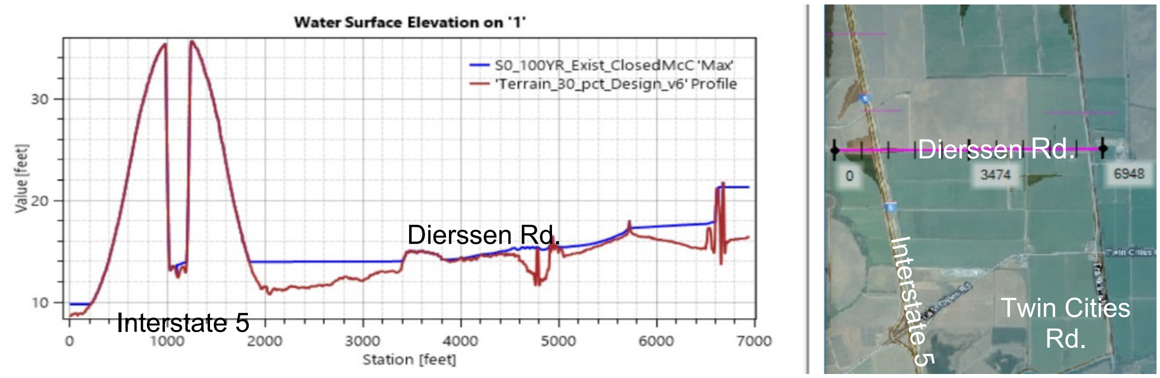


Figure A3-9c. 100-Year Flood Depths Existing Conditions Hydraulic Model at Dierssen Road between Interstate 5 and Franklin Boulevard

3.4.2 Bethany Reservoir Alignment Ring Levee Hydraulic Model Scenario 1 Results

A key element to minimizing the flood effect to the surrounding properties from the ring levee would be to setback the levee from Interstate 5 to allow the 100-year storm frequency flood water to flow overland from north to south between Lambert and Twin Cities roads. The grade of Dierssen Road would not be affected by the Project in the area shown on Figure A3-10a and Figure A3-10b, which allows overland flood flows to inundate Dierssen Road just to the east of I-5 consistent with existing conditions (also refer to Figure A3-9 flood profile along Dierssen Road). Under the existing conditions and conditions under Hydraulic Model Scenario 1, depths of flow over the low point on Dierssen Road would be the same at approximately 3.5 ft.

The flood depth between the ring levee and the existing railroad would be approximately 3.0 ft higher than existing condition because the 100-year floodplain overtops the railroad embankment and is contained in the constricted area between the relocated Franklin Blvd/eastern portion of the ring levee and the existing railroad embankment. Without the ring levee, flows from this overtopping spread west along the Project site. The total volume of this flow is fairly low and increased depths are due to the limited space between Franklin Blvd and the railroad embankment and impacts are localized to this area. The flood elevations in this area would decrease rapidly towards the north and south sides of the ring levee. Note that overtopping of the existing railroad embankment would not occur after the McCormack-Williamson Tract flood project is implemented.

Figure A3-10a. 100-Year Flood Depths under Hydraulic Model Scenario 1 at the Twin Cities Complex Site

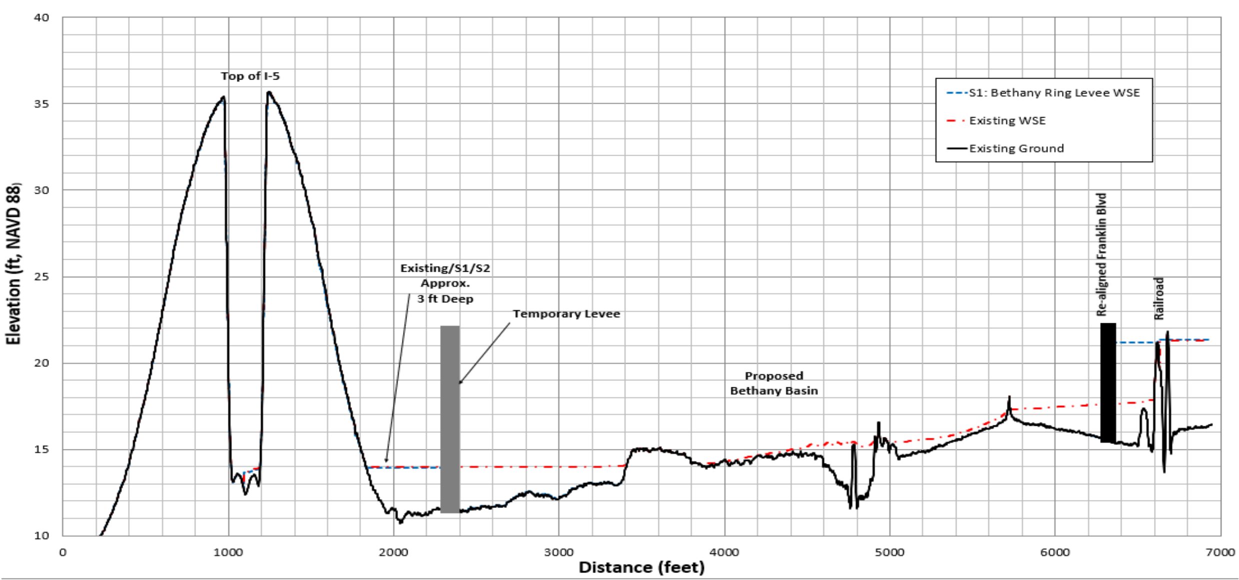


Figure A3-10b. 100-Year Flood Depths Along Dierrsen Road under Hydraulic Model Scenario 1

Figure A3-11 shows the ring levee would increase water levels approximately 0.4 ft. in the 100-year storm event compared to existing condition. This effect location north of the ring levee at the Twin Cities Complex site is currently open space. Review of the modeling results in the surrounding area west of Interstate 5, east of the railroad embankment and south of the Twin Cities Complex site area shows there would be negligible to no change in the inundation depts due to the ring levee. The model simulation shows the flood inundation north of the ring levee would be impacted from approximately 2.5 days.

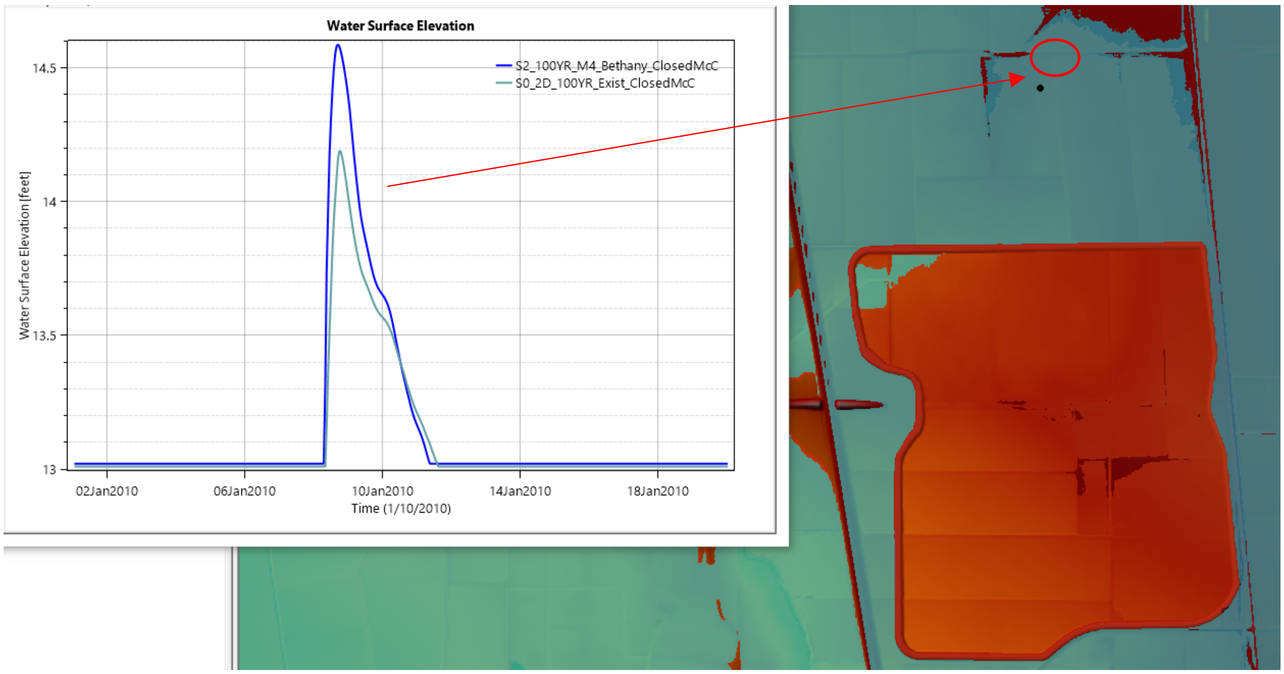


Figure A3-11. 100-Year Flood Depths under Existing Conditions and Hydraulic Model Scenario 1 to the North of Twin Cities Complex Site

Note: The green line is the Existing Conditions water surface elevations and the blue line is the water surface elevations under Hydraulic Model Scenario 1

Figure A3-12 presents the 100-year existing condition floodplain (grey) compared to the Hydraulic Model Scenario 1 condition floodplain (blue). The Hydraulic Model Scenario 1 condition flood extent would increase the 100-year floodplain by approximately 10 acres concentrated in the open area between the north side of the ring levee and Lambert Road.

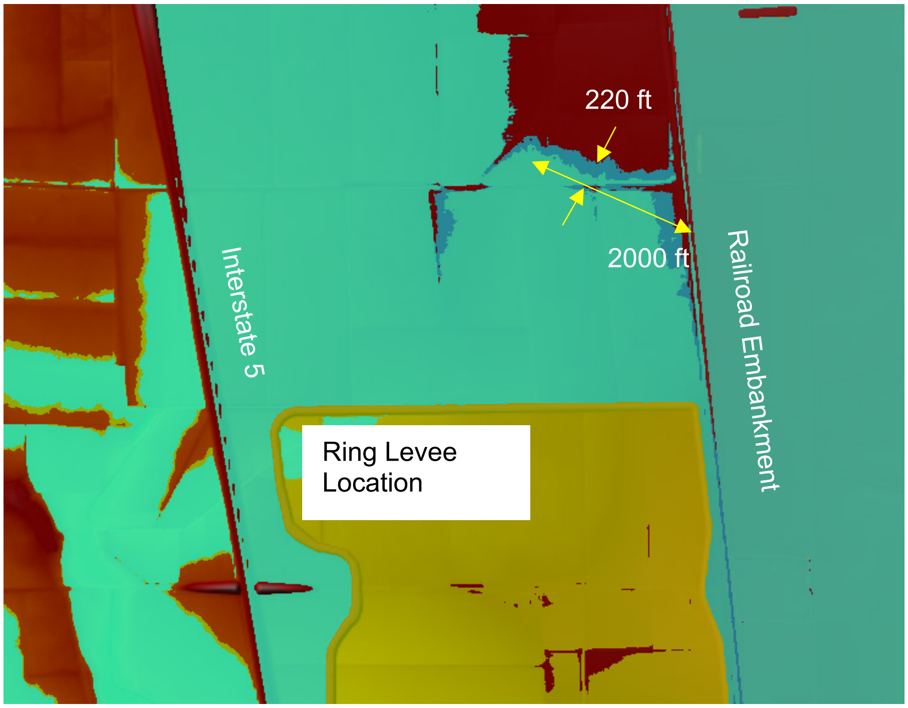


Figure A3-12. 100-year Floodplain under Existing Conditions and Hydraulic Model Scenario 1 north of Twin Cities Complex Site

Figure A3-13 present cross sections from the 2008 LiDAR showing the existing grades between the Twin Cities Complex ring levee location and Interstate 5 may include minor agricultural mounds which may impede the overland shallow flows to Dierssen Road. The green line is the existing ground profile and the red line represents the recommended depth to degrade the area north of Dierssen Rd to allow the shallow 100-year flood flows to easily flow overland over Dierssen Road.

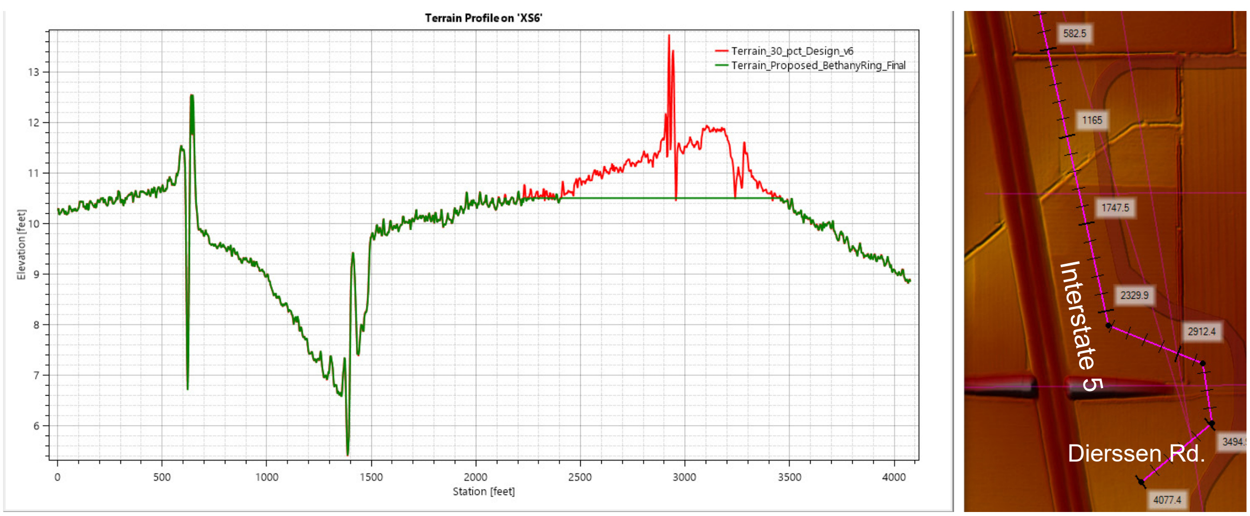


Figure A3-13. Proposed Grading North to South Profile North of Dierssen Road and East of Interstate 5 to Facilitate Flood Flows from North to South towards Dierssen Road

Note: The green line is existing ground surface; red line is proposed ground surface to facilitate flow patterns

Figure A3-14 presents the 100-year existing conditions and Hydraulic Model Scenario 1 water surface elevations comparison in the hydrographs for one location adjacent to Interstate 5 (circled in red) north of Dierssen Road. Due to the limited channel conveyance of the Mokelumne River downstream of Interstate 5 as previous discussed, the 100-year floodplain will begin to backwater through Snodgrass Slough and the local drainage system and store shallow flood water in areas on both the west and east of Interstate 5 near the Project area between Twin Cities Road and Lambert Road. The drainage system on the east and west side of Interstate 5 between Lambert Rd. and Twin Cities Road is connected with culverts under Interstate 5 to allow the shallow flood water from the east side of Interstate 5 to drain to the west side and recede back into Snodgrass Slough as the backwater reduces in elevation. The backwater in Snodgrass Slough would peak and stabilize several days (day 9) after the start of the model simulation and the shallow water inundation in the overbank area west of Interstate 5 would peak on approximately day 12 after the start of the simulation because flood water is slow moving in the range of 0.5-1.0 ft/s.

Within the hydraulic model domain, the 20-day simulation is adequate to evaluate a rising and falling limb of the flow and stage hydrograph impacts, however, the flood inundation west of Interstate 5 between Lambert Rd. and Twin Cities Rd. receives the flood inundation on day 12 which is later in the model simulation. The inundation west of Interstate 5 stores the shallow flood water for a significant amount of time because there is a levee barrier adjacent to Snodgrass Slough which prevents the floodwater to drain freely drain back to the channel. The backwater elevations would reach a peak stage in the range of El. 10.0 ft. The hydraulic model comparison between existing and Hydraulic Model Scenario 1 condition shows there would be negligible effects to the west of Interstate 5.

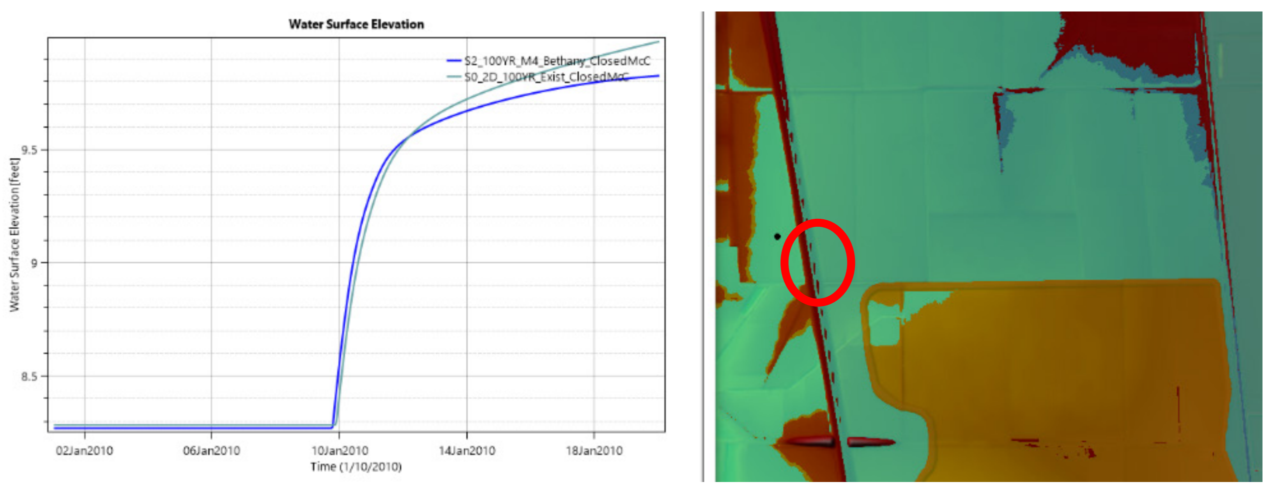


Figure A3-14. 100-Year Flood Condition under Existing Conditions and Hydraulic Model Location **West of Interstate 5 to the North of Dierssen**

Note: The blue line is the Existing Conditions and the green line is the Hydraulic Model Scenario 1 conditions

3.4.3 Bethany Reservoir Alignment Stockpile Scenario 2 Results

Figure A3-15 shows the stockpile storage area would increase water levels approximately 0.15 ft. in the 100-year storm event compared to existing conditions. The effect would be located north of the Twin Cities Complex site is currently open space. The modeling shows that the surrounding areas west of

Interstate 5, east of the railroad embankment and south of the Twin Cities Complex site would not have an effect from the stockpile placement.

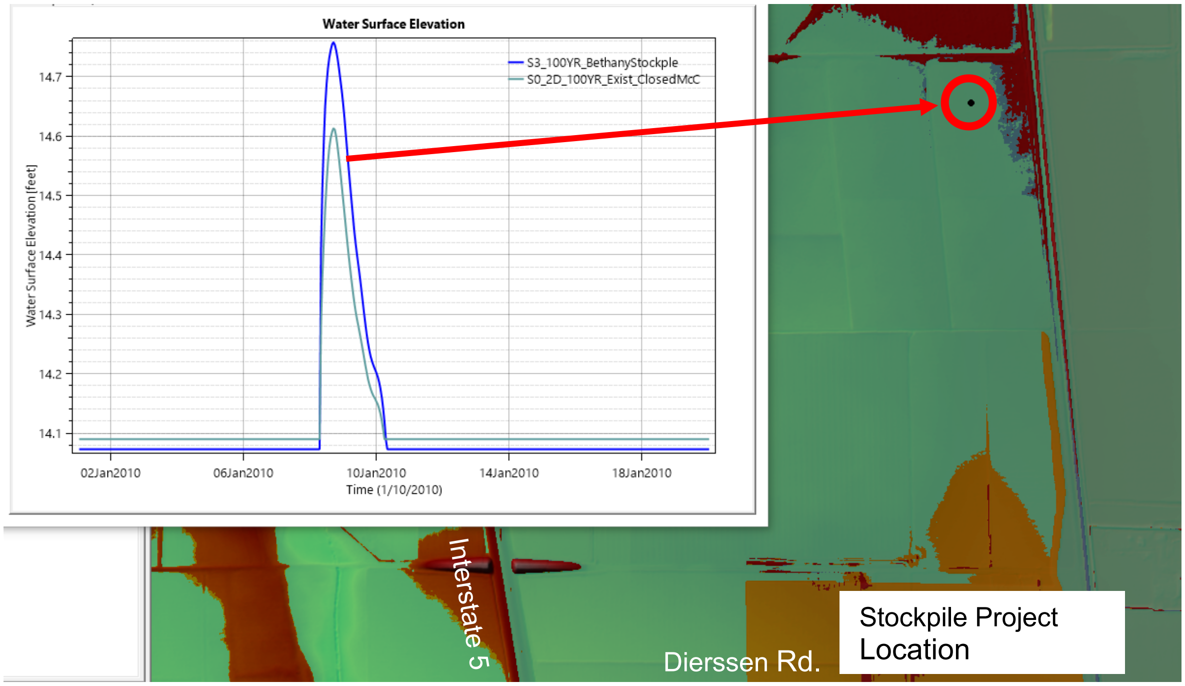


Figure A3-15. 100-Year Flood Depths under Existing Conditions and Hydraulic Model Scenario 2 North of Twin Cities Complex Site

Note: The green line is the Existing Conditions water elevations and the blue line is the water elevations under Hydraulic Model Scenario 2

Figure A3-16 presents the 100-year existing condition floodplain (grey) compared to the Hydraulic Model Scenario 2 condition floodplain (blue). The flood extent would increase the 100-year floodplain surface area by approximately 4 acres.

Figure A3-16. 100-year Floodplain under Existing Conditions and Hydraulic Model Scenario 2 north of Twin Cities Complex Site

Note: The green line is the Existing Conditions water elevations and the blue line is the water elevations under Hydraulic Model Scenario 2

3.5 Observations and Conclusion

- The North Delta hydraulic model was used for this evaluation because the model was calibrated to historical flood event gage data and high-water marks for floods at this geographical location.
- Glanville Tract has a history of flooding along the local levees and the surrounding roadways of Interstate 5, Highway 99, Twin Cities Road and Lambert Road.
- The ring levee and stockpile storage areas would increase 100-year water levels approximately 0.4 ft and 0.15 ft, respectively, compared to existing conditions, but the flood effect is confined to an open space area north of the Twin Cities Complex site with no effect to residential development and/or critical facilities.
- The ring levee and stockpile storage areas would increase the 100-year floodplain approximately 15 acres and 4 acres, respectively, in the open space to north of the Twin Cities Complex.
- The ring levee location was setback from Interstate 5 to allow floodwater to travel in the same direction along Interstate 5 as under existing flood conditions. The depth of flow for both existing and future conditions with the Project would overtop Dierssen Road by approximately 3.5 ft.
- Modeling results show that the ring levee and stockpile storage areas would not change water surface elevation to the west of Interstate 5.

Attachment 4
Shallow Flooding at Intakes

Attachment 4. Shallow Flooding at Intakes

Periodic shallow overland flooding was identified as a concern during community outreach efforts at the Town of Hood. Hood residents indicated areas of shallow ponding north of the community and expressed concern over the proposed intakes being placed in the same general area. The purpose of Attachment 4 is to provide an overview of existing conditions within, and adjacent to, the potential temporary and permanent footprints for Intakes C-E-3, and C-E-5 to assess areas that are susceptible to localized, shallow flooding, and consider how these areas might be impacted by the construction and operation of the intakes. Recommended considerations for final design to reduce potential impacts are also provided.

4.1 Background

Existing conditions within, and adjacent to, the potential temporary and permanent footprints for Intakes C-E-3 and C-E-5 are described below. A location map of the Intakes is shown in Figure A4-1. This map shows Project features including the three intake footprints and the tunnel alignment, as well as the various maintenance agencies in this area.

Agencies with involvement in flood control or drainage within the vicinity of the Intakes includes DWR's Maintenance Area (MA) 9, Reclamation District (RD) 744, and RD 813. MA 9 is responsible for maintaining the entire Sacramento River left levee from the Little Pocket down to Intake C-E-5. RD 744 has responsibility for agricultural lands, flood control, and levee maintenance within its service area not on the Sacramento River (SacLAFCO, 2016). RD 813 has responsibility for maintaining agricultural drainage within its service area (SacLAFCO, 2016).

The Project location for the analyses considered in this Attachment is bounded on the west and north by the Sacramento River, on the east by the abandoned Southern Pacific Railroad (SPRR) embankment, and on the south by the RD 551 Borrow Canal.

4.1.1 Topography

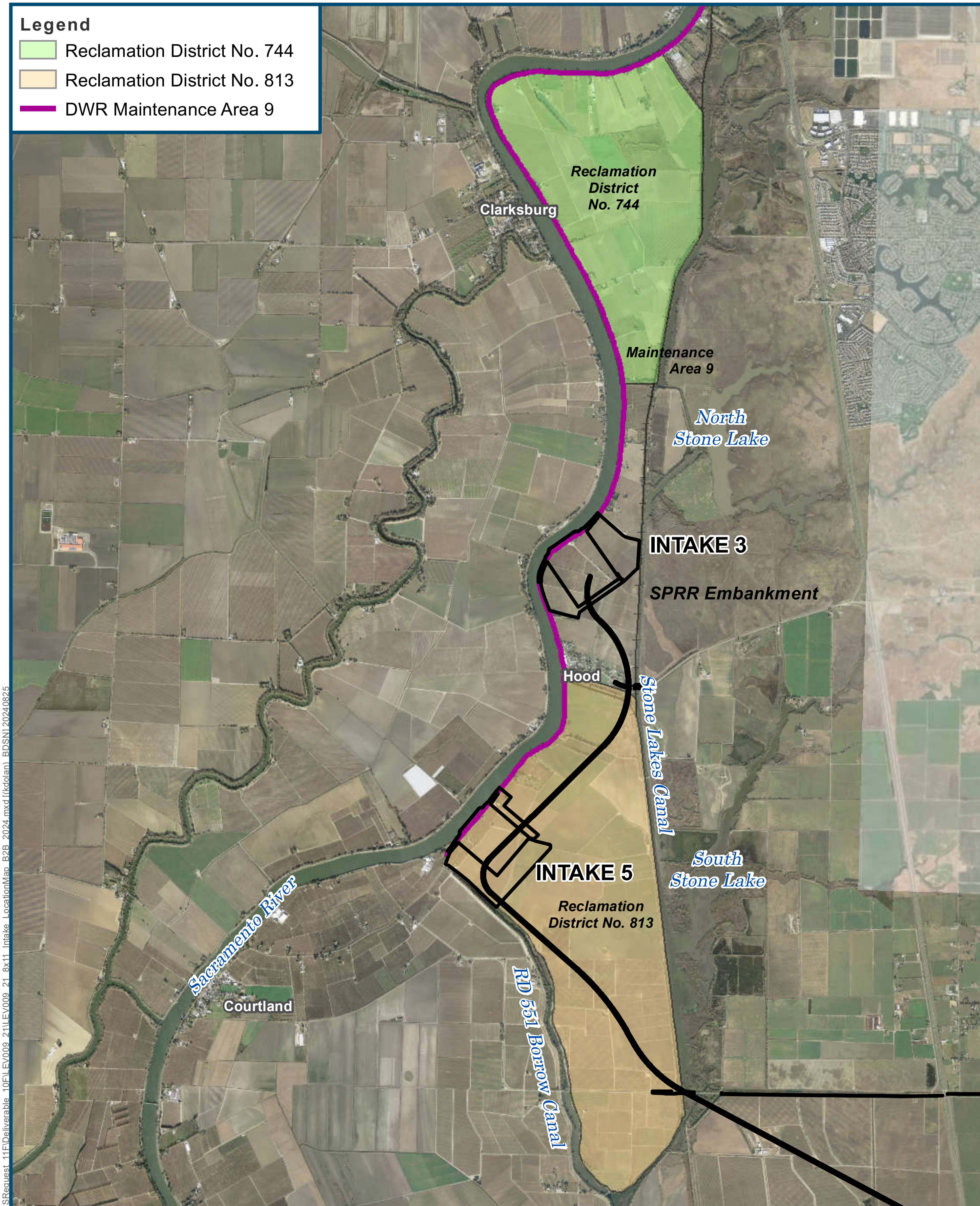
Ground elevations within, and adjacent to, the Project boundaries for Intake C-E-3 and C-E-5 based on 2017 Light Detection and Ranging (LiDAR) are depicted in Figure A4-2. Generally, in the Project area, ground elevations are highest directly adjacent to the Sacramento River Levee and slope away within about 500 feet. Levee crest elevations at the intakes range from approximately 27 feet and 31 feet (all elevations are referenced to NAVD 88). Crest elevations along the abandoned railroad embankment range from approximately 22 feet to 27 feet. Within the Project area, there are three localized depressions, separated by areas of relative high ground which when accounting for local drainage creates separate shallow "runoff basins". Each intake is within a different basin and will be considered separately.

Within the Project construction extent for Intake C-E-3, ground elevation is generally highest immediately adjacent to the landward toe of the Sacramento River levee (at approximately 14 feet). From the landward toe of the Sacramento River levee, ground elevations generally decrease towards the SPRR embankment. The lowest elevation within the temporary footprint of C-E-3 is about 2 feet which includes the lowest point in the basin. The lowest existing ground surface in the permanent footprint is about 3 feet. This area is responsible for runoff from approximately 650 acres.

Within the Project construction extent for Intake C-E-5, ground elevation is generally highest immediately adjacent to the landward toe of the Sacramento River levee (approximately 14 feet). From the landward toe of the Sacramento River levee, ground elevations generally decrease towards the SPRR embankment. The lowest elevation within the temporary footprint of C-E-5 is about 0 feet and it is about 2 feet within the permanent footprint. A localized low point (at approximately -3 feet) which would collect runoff from C-E-5 and surrounding areas is located approximately 6,500 ft southeast of Intake C-E-5. This area is responsible for runoff from approximately 2,400 acres.

4.1.2 Soils and Groundwater

The construction extents for Intake C-E-5 and a portion of Intake C-E-3 primarily lie adjacent to the landward toe of the left bank of the Sacramento River, are within areas with a shallow groundwater table (Jones & Stokes Associates, Inc., 1991). Local landowners routinely use artificial drainage to ensure these areas remain suitable for agriculture. Areas within and adjacent to the locations of Intakes C-E-3 and C-E-5 are comprised of poorly drained soils which can encourage ponded water. Hydric soils, which is considered a criterion for wetland designation by the USACE, are also documented within the Project construction extents for Intakes C-E-3 and C-E-5.



DCA DELTA CONVEYANCE DESIGN & CONSTRUCTION AUTHORITY

For Illustration Purposes Only

N

0 1 Miles

Figure A4-1. Location map

Data Source: DCA, DWR

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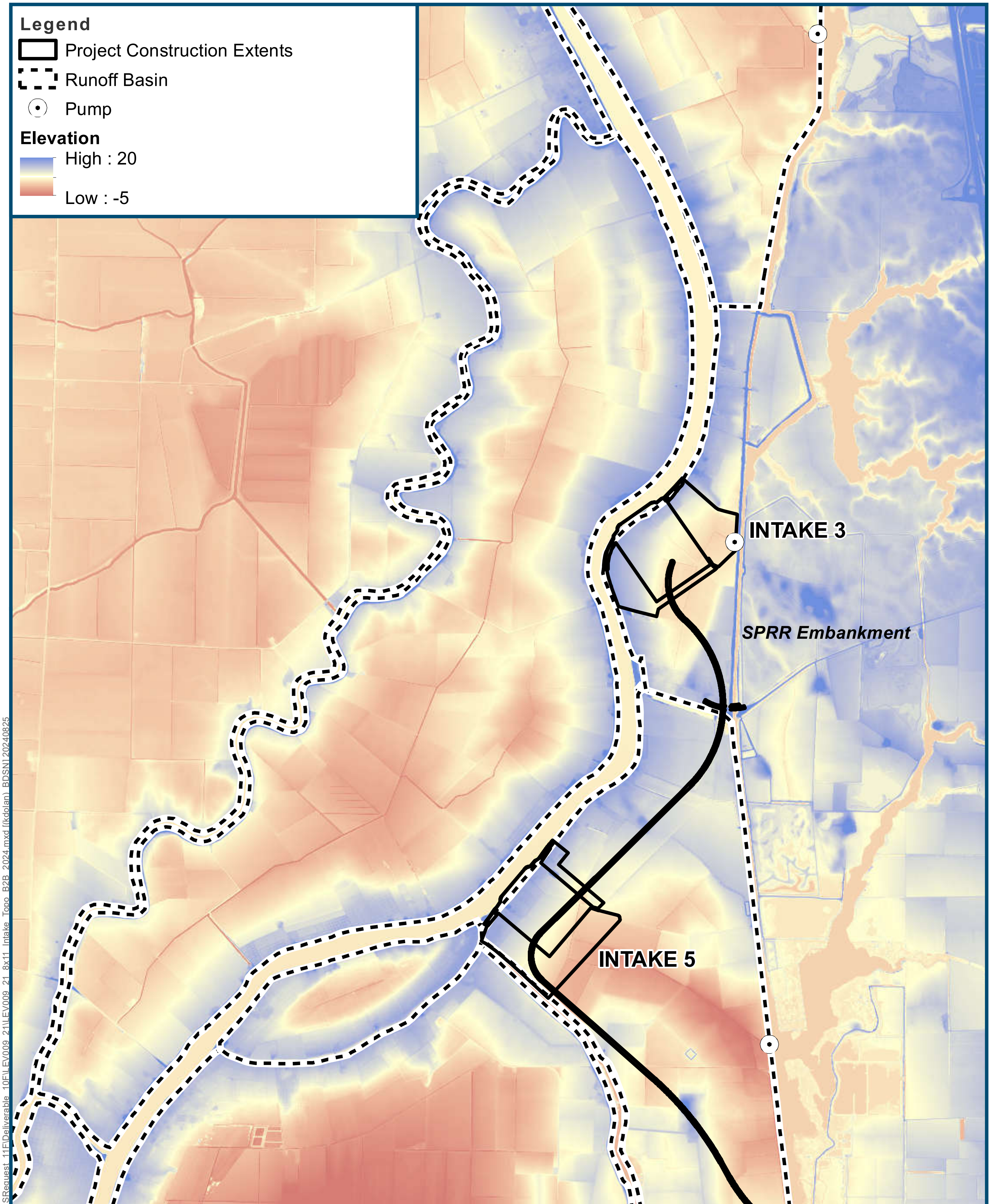

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DCA DELTA CONVEYANCE DESIGN & CONSTRUCTION AUTHORITY

For Illustration Purposes Only

Figure A4-2.
Local Topography
Adjacent to Intakes C-E-3,and C-E-5

Data Source: DCA, DWR

4.1.3 Drainage

Most of the area within, and adjacent to, the construction extents for the intakes consists of agricultural lands, primarily comprised of permanent vineyard and orchard crops, pasture, and seasonal row or field crops. The poorly drained soils within these areas require agricultural drainage to encourage crop development. Agricultural drainage within the construction extents for the intakes is managed by local RDs and landowners. As shown in Figure A4-1, Intake C-E-5 is within the boundaries of RD 813. Based on review of the existing topography, a series of agricultural drainage ditches drain water away from west to east to the low point near Intake C-E-3 where a pump station, location shown in Figure A4-2, lifts the water through the abandoned railroad embankment and into the Stones Lake National Wildlife Refuge Canal.

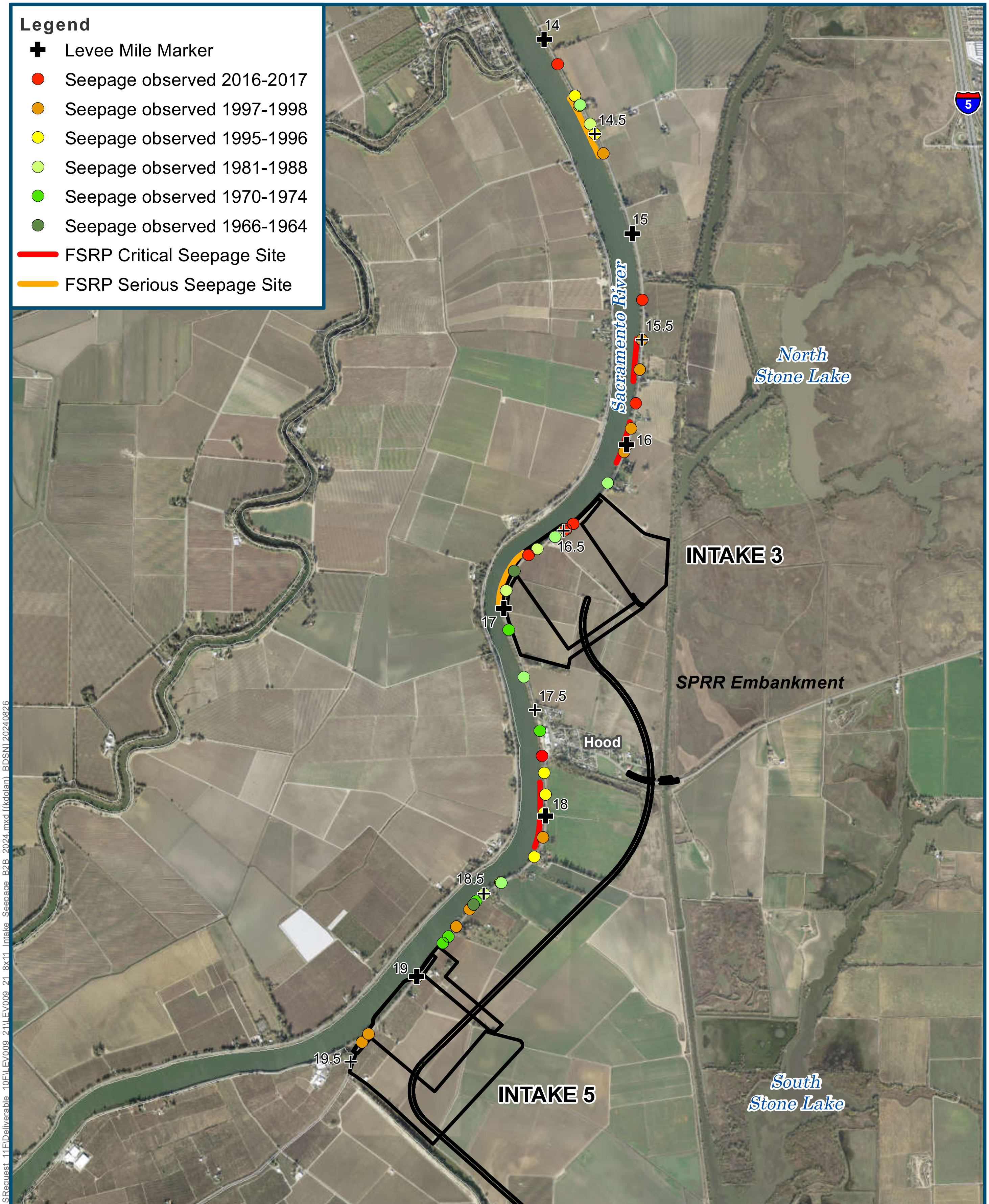
Drainage within the area from north of Intake C-E-3 to the northern boundary of RD 813, inclusive of the construction extent for Intake C-E-3, is managed by local landowner(s) as there is no RD assigned to this area. Within this area, drainage is managed through a series of agricultural ditches and a pump station located on the west side of the SPRR embankment less than one mile northeast of Hood. The pump station, location shown in Figure A4-2, lifts the water through the abandoned railroad embankment and into the Stones Lake National Wildlife RefugeCanal/North Stone Lake area.

Agricultural parcels, including those at the intake sites, generally include agricultural and/or stormwater drains, plus irrigation diversions and ditches. Agricultural drains may be constructed as subsurface drains and are not identified on publicly available records and cannot be fully located without access to the properties. Pump stations associated with irrigation and drainage are not accessible at this time, so their condition and functionality cannot be described. The irrigation diversions and ditch network cannot be fully evaluated without access to the properties, but open-air ditches were observed from aerial photography and LiDAR. The existing pump stations and drainage systems appear to be critical to managing shallow flooding in the low-lying areas within each of the basins.

4.1.4 Past Levee Performance

Instances of past seepage or boils have been documented along the Sacramento River levees between levee mile 14.0 north of Intake C-E-3 and levee mile 19.6 near the downstream boundary of Intake C-E-5, as shown in Figure A4-3 (DWR 1981-2000, DWR 2017,2020). Figure A4-3 also shows the known critical and serious seepage sites documented under DWR's Flood System Repair Project (FSRP) (URS, 2013). It should be noted that conceptual design of the proposed intake facilities includes construction of seepage cutoff walls along the Sacramento River, as well as around the perimeter of the intake facilities. Any levee-related past performance issues that occur within the limits of proposed construction would be remediated by construction of the intake facilities.

The abandoned railroad embankment acts as a levee in this area, protecting the area from overland flooding that occurs east of the embankment. While this embankment acts as a levee, there is no official agency responsible for maintaining, monitoring, or responding to problems along the embankment. Because of this, there are no documented instances of embankment issues which could be identified for this study. However, it is assumed that the abandoned railroad embankment would present many of the same seepage problems identified along the Sacramento River.



DCA DELTA CONVEYANCE DESIGN & CONSTRUCTION AUTHORITY

For Illustration Purposes Only

N

0 3,200 Feet

Figure A4-3.
Past Performance Issues along Sacramento River

Data Source: DCA, DWR

4.2 Outreach With Levee and Drainage Districts.

As described above, entities with involvement in flood control or drainage within the vicinity of the Intakes includes MA 9 and RD 813. Each of these entities were contacted to attempt to confirm the findings described above and clarify the function of the current drainage infrastructure. MA 9 staff met with DCA on October 18, 2021 and confirmed the existing seepage issues along the Sacramento River. However, MA 9 is only responsible for operation and maintenance of the Sacramento River levee and does not necessarily document landward instances of overland flooding if it does not impact operations or monitoring at the levee. Because of this, MA 9 was not able to provide specifics about past instances of overland or related shallow flooding in the Project area.

As part of this evaluation, DCA staff contacted RD 813 representatives to discuss their infrastructure and operations. The RD declined the invitation to meet with DCA staff.

4.3 Sources of Shallow Flooding Near Intakes

Based upon review of the background information summarized above, there are three primary sources of water which may contribute to localized shallow flooding near the intakes. These include stormwater runoff, increased groundwater levels during high precipitation periods and high-water events in the adjacent river/ water bodies, and through-seepage in the Sacramento and abandoned levee embankments. Due to the lack of rigorous documentation in this area and lack of direct input from the local RDs, it is difficult to quantitatively determine the contributory effects of each of these sources.

Stormwater runoff within each area drains to existing low points within each basin. A 50 year-recurrence interval storm with a duration of 24 hours is approximately 0.19 inches/hour, or 4.71 inches over 24-hours, when considering historical rainfall in the nearby Town of Clarksburg (NOAA, 2014). While most of the land within each basin is considered undeveloped, agricultural land, the potential volume of runoff that could be generated from this rainfall within each basin is highly dependent upon soil type, crop type, and antecedent moisture conditions, along with rainfall intensity. Final designs of the intakes would need to account for conveyance and accumulation of stormwater, depending on the intake site.

As indicated in the sections above, the areas around the intakes are prone to high groundwater which may be exacerbated due to high water events in the Sacramento River and Stone Lakes National Wildlife Refuge. Construction of the intakes are not anticipated to cause additional flooding from underseepage, since the intakes include cutoff walls beneath all levees, which would decrease the potential for underseepage-related high groundwater to cause shallow flooding. Loss of flood storage in the landward areas would be unaffected by this potential condition, since upward gradients associated with high groundwater and underseepage are not governed by volume, but rather by differential heads and subsurface gradients.

The Sacramento River levees in the Project area are understood to be composed of sandy course-grained materials which are prone to through-seepage. Additionally, the SPRR embankment is expected to be susceptible to the same conditions as the Sacramento River. This seepage can accumulate and contribute to any existing ponding from the other sources.

4.4 Intake Considerations

4.4.1 Intake C-E-3

Because the drainage collection point in this area is within the temporary and permanent footprint areas, Intake C-E-3 would likely have a storage impact on the existing floodplain within its drainage basin. The footprint includes a large portion of the lowest elevation ground in the vicinity, as well as, existing drainage infrastructure.

Drainage and agricultural infrastructure would require modifications to accommodate the construction of the intake based on available data. These changes would be accommodated within the footprint currently identified. During the design phase, field work would be completed to identify locations of agricultural diversions, ditches, and drains and define the function and character of these features. If the diversions, ditches, and drains serve adjacent properties, modifications to these features would occur to preserve this service as part of site preparation, prior to construction. During construction, site specific structural and operational BMPs would be implemented to prevent and control impacts on stormwater runoff, including monitoring by visual and/or analytical means. These BMPs would be implemented as necessary before storm events and inspected/maintained on a regular basis.

Based on available information, a local land owner currently operates a drainage pump to clear runoff into the adjacent Stone Lakes National Wildlife Refuge Canal. Because this pump is within the footprint of Intake C-E-3, the design phase would assess the needs of the pump station and include upgrades, as necessary.

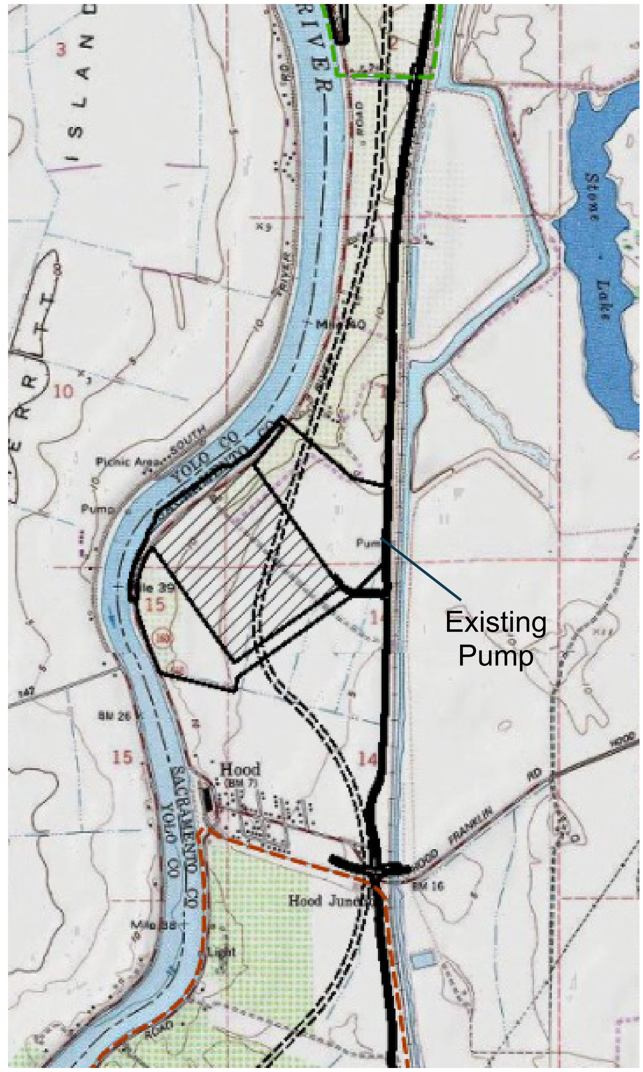


Figure A4-4. Intake C-E-3 USGS Topo

4.4.2 Intake C-E-5

Intake C-E-5 would likely have a minimal storage impact on the existing floodplain within RD 813. This is because the site is located generally on higher elevation ground and thus would not take up volume within the lowest lying areas within this basin.

Drainage and agricultural infrastructure appear to be relatively unaffected by construction of C-E-5 based on available data. During the design phase, field work would be completed to identify locations of agricultural diversions, ditches, and drains and define the function and character of these features. If the diversions, ditches, and drains serve adjacent properties, modifications to these features would occur to preserve this service as part of site preparation, prior to construction. During construction, site specific structural and operational BMPs would be implemented to prevent and control impacts on stormwater runoff, including monitoring by visual and/or analytical means. These BMPs would be implemented as necessary before storm events and inspected/maintained on a regular basis.

Based on available information, RD 813 appears to operate a drainage pump to clear runoff into the adjacent Stone Lakes National Wildlife Refuge Canal. During the design phase, the RD would be coordinated with to confirm the reliability of the existing infrastructure and work with the RD if improvements need to be implemented.

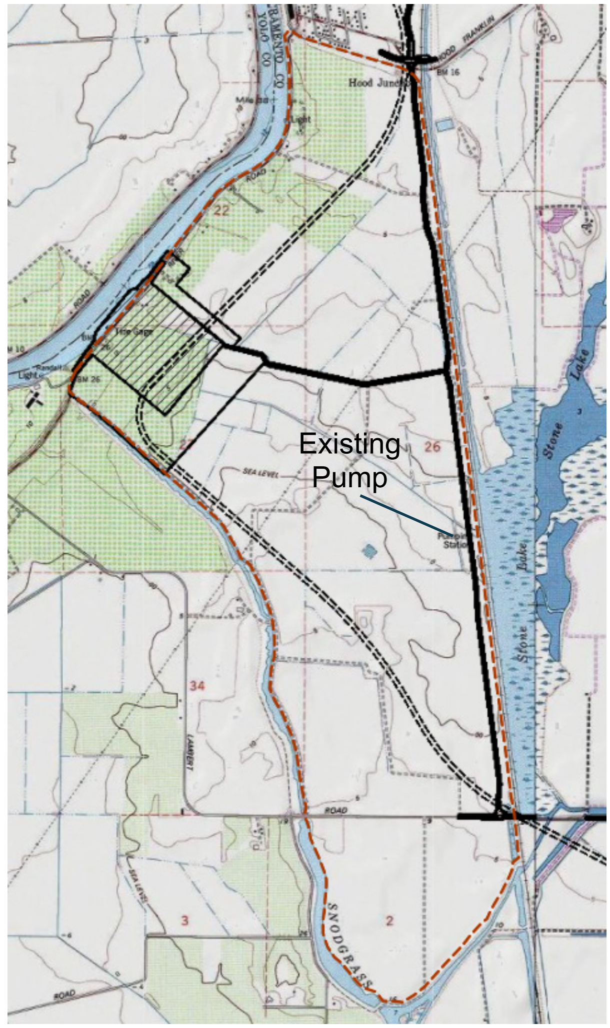


Figure A4-5. Intake C-E-5 USGS Topo

4.5 Observations and Conclusions

- The area around the intakes is prone to shallow groundwater tables and has poorly drained soils which can encourage ponded water
- The topography in the area surrounding the intake sites forms three separate basins, with Intakes C-E-3 and C-E-5 each in a different basin. Intake C-E-5 is located at a relatively high elevation. The permanent footprint for Intake C-E-3 extends to approximately the low point of the basin.
- Existing drainage ditches and pump systems are used to manage flooding in each of the basins by discharging water into the Stone Lakes canal. The exact source of flooding cannot be determined without detailed observation at the site and cooperation from local landowners. Local Reclamation Districts were contacted for this study but declined requests to meet with DCA staff to confirm site conditions and observations during past storm events. DWR Maintenance Area 9 is responsible for maintaining the Sacramento River Levee along the intakes, but not the interior areas.

- The Sacramento River levee has experienced a number of past performance issues related to seepage. Some of these potential seepage sites will be remediated with construction of the intakes. There is no past performance documentation related to the SPRR embankment.
- IntakeC-E-5 is at fairly high elevations within its respective basin. The construction of the intake is unlikely to make a significant impact on existing flooding and drainage, but design of drainage features would be done in coordination with the local Reclamation Districts. The footprint for Intake C-E-3 includes the existing drainage pump for this area and any necessary upgrades would be assessed during the design phase.