Subject:	**Levee Vulnerability Assessment and Flood Risk Management Supplement – Bethany Reservoir Alternative (Final Draft)**
Project feature:	Levees
Prepared for:	California Department of Water Resources (DWR) / Delta Conveyance Office (DCO)
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1. Purpose, Introduction and Background

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 Project and potentially the Central Valley Project conveyance and distribution systems.

Two tunnel alignments have previously been analyzed, the Central and Eastern corridors, and two technical memoranda (TMs) were prepared related to existing levee conditions covering these corridors:

- The *Levee Vulnerability Assessment TM* (DCA 2021a) outlined and defined relative levee vulnerability along the corridors.
- The *Flood Risk Management TM* (DCA 2021b) identified the risk of overland flooding at the tunnel shafts sites during the construction process.

This supplemental TM addresses the analysis of both levee vulnerability and flood risk management for the Bethany Reservoir Alternative alignment (Figure 1-1).

From a levee vulnerability and flood risk management perspective, the Bethany Reservoir Alternative alignment is the same as the Eastern corridor alignment from the intake sites along the Sacramento River to the Lower Roberts Island tunnel shaft site. From there, the Bethany Reservoir Alternative alignment would diverge south from the Eastern corridor with new maintenance shafts on Upper Jones Tract and Union Island, and then a tunnel reception shaft at the Bethany Reservoir Pumping Plant (BRPP) and Surge Basin located on the southern side of Byron Highway near Mountain House Road. The Bethany Reservoir Alternative would not include a Southern Forebay and would instead deliver project water directly to the existing Bethany Reservoir.

The purpose of this TM is to analyze flood risks along the Bethany Reservoir Alternative alignment that are not currently covered by the *Flood Risk Management TM* and *Levee Vulnerability Assessment TM* (DCA 2021b and 2021a, respectively), which focus on the Central and Eastern corridors. This supplemental TM is split into two main sections: Section 2 addresses the levee vulnerability assessment supplement, and Section 3 addresses the flood risk management supplement. This TM provides an overview of the approaches, but additional detail is included in the *Levee Vulnerability Assessment TM* and *Flood Risk Management TM* (DCA 2021a and 2021b, respectively).

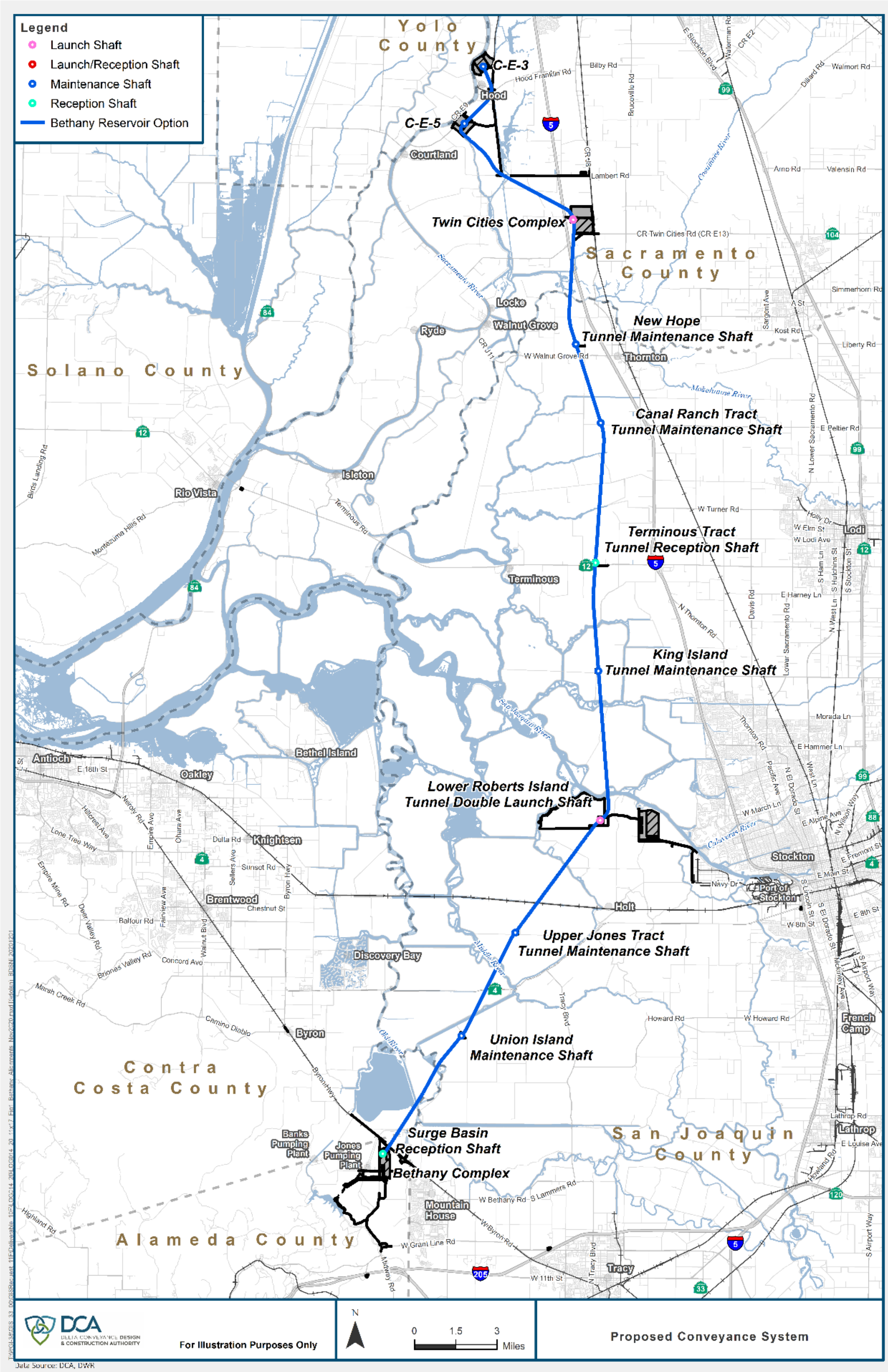

Figure 1-1. Delta Conveyance Project Bethany Reservoir Alignment

1.1 Organization

This TM is organized as follows:

- Levee Vulnerability Supplement
- Flood Risk Management Supplement
- References
- Document History and Quality Assurance

2. Levee Vulnerability Supplement

The levees in the Delta are exposed to many hazards that may damage levees or cause them to fail or breach, resulting in the flooding of the island interior. The most significant hazards are due to hydrologic, hydraulic, and seismic (earthquake) loading, which can lead to seepage, stability or over-topping related failures. A variety of site-specific conditions can also contribute to a levee's vulnerability for failure when subjected to loading, including poor or weak embankment or foundation soils, insufficient levee geometry (height, width, and slope inclination), and various types of damaging animal activity or vegetation growth.

This supplemental TM only addresses the levees that were not addressed in the previous *Levee Vulnerability Assessment TM* (DCA 2021a). These additional areas include the Clifton Court Tract, levees on the southern side of the Delta-Mendota Canal (DMC) (across from the Clifton Court Tract), and levees on the southern side of Old River (across from the Fabian Tract) (Figure 2-1). Project features are planned for the Bethany Reservoir Alternative on Union Island, and Upper Jones Tract which were not a part of the Central and Eastern corridors. However, the levees on these islands were included in the Central and Eastern Corridor *Levee Vulnerability Assessment TM.* The levees analyzed in this supplement were used in rating potential shaft locations as discussed in the *Facilities Siting Study – Bethany Reservoir Alternative* (DCA 2021c).

2.1 Methodology

The methodology for this supplemental TM is the same as those outlined in the *Levee Vulnerability Assessment TM* (DCA 2021a). The methodology for this supplemental TM is summarized here but for additional detail, refer to the *Levee Vulnerability Assessment TM* (DCA 2021a).

The goal of this assessment is to evaluate indicators of levee condition that do not rely heavily on site-specific subsurface data, while providing meaningful results to compare levee vulnerability. The existing levee geometry can provide an indication of how levee systems may perform during different loading conditions, and can provide an even stronger indication of how levees might perform relative to one another. Broader levees with greater freeboard, wide crests, and shallow slopes would inherently be less vulnerable than narrower levees with similar composition, loading, and foundation conditions.

Criteria related to overall levee geometry, freeboard, toe ditches, vulnerability to sea level rise (SLR), and changes in crest elevation were used to assess the relative vulnerability of the levees. The evaluation was performed using cross-sections developed every 500 feet along the levee alignments using light detection and ranging (LiDAR) data collected and provided by DWR (DWR 2017).

The geometric criteria developed for this study do not provide a comprehensive evaluation of a levee system or guarantee levee performance. The results provided here are intended to help locate project infrastructure and better understand potential flood risks within the Bethany Reservoir Alternative project area. This vulnerability assessment does not replace the need for site-specific investigations, testing, and analyses.

2.1.1 Information Sources

Numerous programs and supporting studies have been performed to evaluate the conditions of existing levees in the Delta and potential vulnerabilities due to flooding and seismic events. The programs included assessments on the Delta's ecosystem and habitat, water quality, water availability, natural hazards impact, land use, and economic impact, among others. Those levee studies were performed with various objectives and levels of detail, and culminated in providing a range of data compilations and levee performance evaluations with varying approaches and spatial distributions. Refer to the *Levee Vulnerability TM* (DCA 2021a) for complete listing of studies and references. Data used for this levee vulnerability assessment include:

- **Flood Elevation Data**: The 100-year and 300-year design flood elevations used for the assessment are based on the *Hydrology Study for the Sacramento San Joaquin Delta* developed by the U.S. Army Corps of Engineers (USACE) (1992). These flood elevations are used for the freeboard requirements for Delta levee geometry Standards. 100-year flood elevations are used in Hazard Mitigation Plan (HMP), and Public Law (PL) 84-99 (PL 84-99) levee standards. 300-year flood elevations are used in the DWR Bulletin 192-82 (192-82) levee standard. Additional detail is provided in Section 2.1.2.
- **Levee Stationing**: Assigned to levees within the study area using the station points identified in the USACE 1992 study; however, these have been adjusted to new levee centerlines where levee alignments have been modified since the 1992 report.
- **Organics/Peat Thickness**: The thickness of organic material in the levee foundation as an input to define the required landside slope geometry to meet Delta levee geometry standards. Contours of organics/peat thickness were previously developed and digitized for the *Delta Risk Management Strategy* (URS 2008a) and were used in this levee vulnerability assessment.
- **LiDAR Survey Data**: The cross-sectional data used in this vulnerability assessment were developed using LiDAR data obtained by DWR in December 2017 and January 2018. The vulnerability assessment uses a simplified slope geometry defined by crest elevation, crest width, landside levee height, waterside and landside slope, landside berm height and slope if present, and landside toe ditch location if present. 2007 LiDAR data collected by DWR were also incorporated into this levee vulnerability study by comparing changes in crest elevation between 2007 and 2017.

The levee vulnerability study was performed using cross-sections developed along levee systems protecting the Clifton Court Tract and on the southern side of the DMC and Old River. The cross-sections are typically spaced every 500 feet along the levee centerline. Internal levees that are not intended to provide flood protection are typically not included in the dataset. The Clifton Court Tract has 52 analysis cross-sections and the DMC and Old River areas have 51 analysis cross-sections.

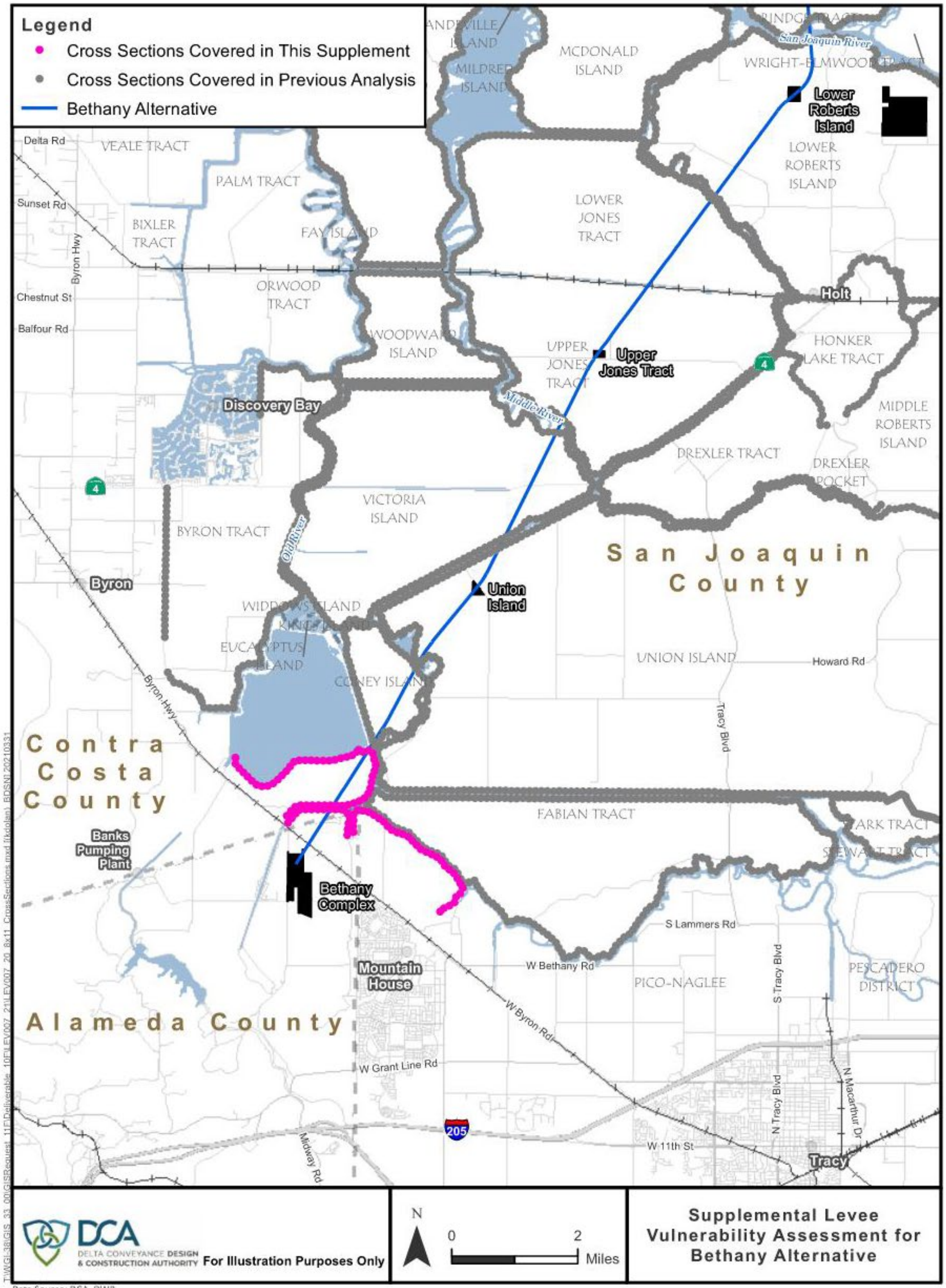

Figure 2-1. Supplemental Levee Vulnerability Assessment for Bethany Alternative

2.1.2 Levee Geometry and Freeboard Standards

Delta levees are held to different standards, based on whether they are part of the Federal Flood Control Project or not, and whether they protect urban locations. These requirements were not considered as part of this assessment because the intent of this supplemental TM is to limit the potential risk to the project infrastructure irrespective of what else those levees protect, or how they were originally constructed. The three levee geometry standards considered for this assessment include Hazard Mitigation Plan, PL 84-99, and DWR Bulletin 192-82. The HMP requires the smallest levee prism of the three while 192-82 is the largest. Table 2-1 summarizes levee geometry design standards that apply to the Delta levees.

Table 2-1. Delta Levee Design Standards

Standard	Freeboard	Crown Width	Waterside Slope	Landside Slope
HMP	1 ft above 100-year WSE	16 ft	1.5:1 (H:V)	2:1 (H:V)
PL 84-99	1.5 ft above 100-year WSE	16 ft	2:1 (H:V)	3:1 to 5:1 (H:V)[a]
DWR Bulletin 192-82	1.5 ft above 300-year WSE (rural) 3 ft above 300-year WSE (urban)	16 ft	2:1 (H:V)	3:1 to 7:1 (H:V)[b]

[a] Landside slope that ranges from 3H:1V to 5H:1V depending on height of levee and thickness of peat (See Attachment 1)

[b] Landside slopes without a berm range from 3H:1H to 7H:1V. Landside levee slopes with a berm are 3H:1V and include berms that are ½ the levee height with slopes that range from 3H:1V to 13H:1V (See Attachment 1).

Notes:

ft = foot (feet)

H:V = horizontal to vertical

WSE = water surface elevation

2.1.3 Relative Levee Vulnerability Criteria

The relative levee vulnerability criteria presented here were developed internally by the DCA team and through feedback on the approach provided by the DCO. The criteria used in this TM to evaluate relative levee vulnerability are as follows:

- Criterion 1 – Levees meeting levee geometry standards
- Criterion 2 – Freeboard against the 100-year flood elevation
- Criterion 3 – Proximity of toe ditch (if present) to landside toe of levee or berm
- Criterion 4 – Vulnerability to SLR
- Criterion 5 – Change in levee crest elevation between 2007 and 2017 LiDAR

Each criterion was evaluated using a rating score that varied from 1 to 4 (1 being unfavorable, 4 being favorable) and was assigned an importance (weighting) factor ranging from 1 to 5 (1 being of little importance, 5 being very important). The rating scores and importance factors were multiplied together for each criterion, and the cumulative sum of all criteria provides a levee vulnerability score. The vulnerability scores can then be grouped and compared to provide a relative levee vulnerability rating (Levee Vulnerability Rating). This section further discusses the rating score for each criterion, as well as the levee vulnerability score and levee vulnerability rating. Table 2-2 summarizes the criteria, rating scores, and importance factors used in this assessment.

Table 2-2. Levee Vulnerability Evaluation Criteria and Vulnerability Ratings

Criterion	Importance Factor	Rating Score (Lower Numbers = Worse Conditions)			
		1	2	3	4
Levees Meeting Geometry Standards	5	Does Not Meet HMP	Meets HMP	Meets 84-99	Meets 192-82
Freeboard Against 100-year Flood Elevation	3	less than 0 ft	0 to less than 1 ft	1 to less than 1.5 ft	Greater than or equal to 1.5 ft
Toe Ditches	2	Ditch is present within 1 levee height from landside levee toe and no berm is present	Ditch is present within 1 to 2 levee heights from landside levee toe or 1 levee height from berm toe	Ditch is present within 2 to 4 levee heights from landside levee toe or 1 to 2 levee heights from berm toe	No ditch or ditch is present beyond 4 levee heights from landside levee toe and beyond 2 levee heights from berm toe
Vulnerability to Sea Level Rise	2	less than 0 ft	0 to less than 1 ft	1 to less than 1.5 ft	Greater than or equal to 1.5 ft
Change in Levee Crest Elevation	2	>1.0-ft decrease in crest elevation from 2007 to 2017	0.75- to 1.0-ft decrease in crest elevation from 2007 to 2017	0.5 to 0.75-ft decrease in crest elevation from 2007 to 2017	Less than 0.5-ft decrease in crest elevation from 2007 to 2017

The vulnerability criteria used in these analyses are based on LiDAR data and are therefore limited by the level of accuracy associated with the source dataset. Allowable tolerances in the assessment were incorporated so small deficiencies that are within the vertical accuracy tolerance of the source data do not flag a levee section as deficient. A tolerance of +/- 0.1 foot was applied to levee crown elevations, and +/- 1.0 foot was applied to levee toe elevations to account for accuracy in vegetated areas.

The rating scores and importance factors for the relative levee vulnerability criteria were multiplied together and summed to provide a levee vulnerability score that ranged from 14 to 56 for each levee cross-section. The lowest possible vulnerability score (highest relative vulnerability) a cross-section could receive is 14, which results from receiving a rating score of 1 for each of the five criteria. Similarly, the highest possible vulnerability score (lowest relative vulnerability) a cross-section could receive is 56, which is obtained by receiving a rating score of 4 for each of the five criteria.

The levee vulnerability scores provide a single metric that can be used to compare the relative vulnerability of one levee cross-section to another. By combining the levee vulnerability scores for all cross-sections and binning the results into quartiles, relative vulnerability ratings (Levee Rating) were developed. The data quartiles from the *Levee Vulnerability Assessment TM* (DCA 2021a) were applied directly to this supplemental analysis rather than recalculating the quartiles given the small dataset this supplement represents. The levee scores associated with each Levee Rating quartile are:

- High relative vulnerability: levee vulnerability score range is from 14 to 38
- Medium relative vulnerability: levee vulnerability score range is from 39 to 47
- Low relative vulnerability: levee vulnerability score range is from 48 to 52
- Very Low relative vulnerability: levee vulnerability score range is from 53 to 56

2.1.4 Assumptions

The levee vulnerability assessments provided here are based on available topographic data, subsurface data (peat/organics thickness), and existing target WSEs (100-year and 300-year). The results of the analyses are therefore influenced by the accuracy of available data, as discussed. Assumptions used to perform the relative vulnerability assessment include the following:

- The simplified cross-section consisting of points at the waterside contact between the slope and river level, levee crest hinges, levee toe, berm toe (if present), and ditch hinges (if present) adequately represents existing levee geometry for the purposes of this study.
- Source topography is based on LiDAR and does not include bathymetry. The LiDAR dataset used for this vulnerability assessment does not define the waterside toe elevation or waterside slope below the waterline at the time of the survey. These unknowns are acknowledged and assumed to be negligible.
- Inaccuracies and uncertainty present within source data would affect sections similarly throughout the Delta.

2.2 Analysis and Results

The results of the levee vulnerability evaluation are summarized as a percentage of cross-sections within a levee system that received a rating score of 1, 2, 3, or 4 (Table 2-3). Table 2-4 provides overall relative vulnerability ratings. Figures A2-1 through A2-6 (Attachment 2) provide a graphical presentation of the results.

Table 2-3. Levee Vulnerability Criteria Results

	Levee Geometry Standards	**Clifton Court Tract (%)**	**South of DMC and Old River (%)**
1:	Does not meet minimum HMP geometry	0	0
2:	Meets HMP geometry but not PL84-99	0	0
3:	Meets PL84-99 geometry but not 192-82	0	0
4:	Meets Bulletin 192-82 geometry	100	100
Freeboard			
1:	Freeboard against the 100-year WSE less than 0 ft	0	0
2:	Freeboard against the 100-year WSE is greater than or equal to 0 ft but less than 1 ft	0	0
3:	Freeboard against the 100-year WSE is greater than or equal to 1 ft but less than 1.5 ft	0	0

Table 2-3. Levee Vulnerability Criteria Results

	Levee Geometry Standards	Clifton Court Tract (%)	South of DMC and Old River (%)
4:	Freeboard against the 100-year WSE is greater than or equal to 1.5 ft	100	100
Toe Ditch Proximity			
1:	Toe ditch is present within 1 levee height form the landside levee toe and no berm is present	0	0
2:	Toe ditch is present within 1 to 2 levee heights from the landside levee toe or 1 levee height from the berm toe	0	0
3:	Toe ditch is present within 2 to 4 levee heights from the landside levee toe or 1 to 2 levee height from the berm toe	0	0
4:	No toe ditch is present, or the ditch is present beyond 4 levee heights from the landside levee toe and beyond 2 levee height from the berm toe	100	100
Vulnerability to Sea Level Rise			
1:	Freeboard against the 100-year SLR WSE less than 0 ft	0	0
2:	Freeboard against the 100-year SLR WSE is greater than or equal to 0 ft but less than 1 ft	0	0
3:	Freeboard against the 100-year SLR WSE is greater than or equal to 1 ft but less than 1.5 ft	0	0
4:	Freeboard against the 100-year SLR WSE is greater than or equal to 1.5 ft	100	100
Change in levee crest elevation			
1:	Crest elevation decrease greater than 1.0 ft between the 2007 and 2017 LiDAR	12	16
2:	Crest elevation decrease equal to 0.75 ft to less than 1.0 ft between the 2007 and 2017 LiDAR	2	2
3:	Crest elevation decrease equal to 0.5 feet to less than 0.75 ft between the 2007 and 2017 LiDAR	4	8
4:	Crest elevation decrease less than 0.5 ft between the 2007 and 2017 LiDAR	83	75

Table 2-4. Overall Relative Levee Vulnerability Rating

Relative Levee Vulnerability Rating	Clifton Court Tract (%)	South of DMC and Old River (%)
High relative vulnerability: vulnerability score range is from 14 to 38	0	0
Medium relative vulnerability: vulnerability score range is from 39 to 47	0	0
Low relative vulnerability: vulnerability score is from 48 to 52	13	18
Very low relative vulnerability: vulnerability score range is from 53 to 56	87	82

2.3 Levee Vulnerability Observations and Conclusions

This supplemental TM presents the results of a relative levee vulnerability assessment performed for levees potentially impacted by the Bethany Reservoir Alternative alignment that were not analyzed as part of the study of the Central and Eastern corridors (DWR 2021a). The results of this TM are intended as a screening-level assessment to identify potential vulnerabilities within the Delta levee systems, and not to be interpreted as design-level analyses. However, it should be noted that screening existing levee geometry as a means for prioritizing levee upgrades is common practice within the Delta and supported by DWR Delta Special Projects program managed by the Delta Levees group within DWR. Key observations and conclusions from the assessment include:

- The relative vulnerability ratings of Very Low, Low, Medium, or High are a metric to compare one levee cross-section and system to another. Levee cross-sections may meet relatively stringent current standards (that is, 192-82 geometry and freeboard) but may still be characterized as not meeting the lowest relative vulnerability due to potential impacts from subsidence.
- When siting project infrastructure, the relative vulnerability ratings and levee geometry standards should be considered as part of the selection siting process. Levee locations with higher vulnerability rankings may require more robust mitigations and/or repair footprints.
- The results provided here should be used in conjunction with sound engineering judgement when selecting the locations of project infrastructure. This analysis provides an indication of levee relative vulnerability at discrete cross-section locations. The higher relative vulnerability rankings serve as an indicator of levee locations within the tunnel corridors that may be deficient and require further evaluation and possible mitigation. Future repairs should consider type, magnitude, and extent of deficiencies.
- As project components progressed from feasibility- and planning-level studies to design-level studies, it would be necessary to obtain site-specific subsurface data and testing and to conduct site-specific engineering analyses. In order to protect the levees and the work, during the design and construction phases, special provision such as emergency response plans would be made to protect the levees within the vicinity of the associated work.
- The levees along the Clifton Court Tract and on the southern side of the DMC and Old River appear to be relatively robust compared to other Delta levees, based on the variables considered in this assessment. These levees would likely require few repairs if any to protect project infrastructure.

3. Flood Risk Management Supplement

This portion of the supplemental TM focuses on the consequences of flooding at the tunnel launch, reception, and maintenance shaft sites during 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 case of a flood event during construction. A proportional level of flood protection for the infrastructure and equipment at each construction site is also an important consideration.

This supplement provides an evaluation of flood risks and considers mitigation options only for sites not included in the *Flood Risk Management TM* (DCA 2021b). The analyses are specific to conditions at each shaft location, so this supplement includes maintenance shafts at Upper Jones Tract and Union Island, and the reception shaft on the southern side of Byron Highway. This supplement only provides updates for analysis at these sites and does not repeat discussions of historical flooding conditions in the Delta. Additional details and discussions can be found in the *Flood Risk Management TM* (DCA 2021b).

3.1 Flood Risk and Proportional Response

For this supplemental 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 including expensive tunnel equipment, such as a tunnel boring machine (TBM), effects on the functioning of important infrastructure, damage to fisheries and wildlife, and other significant measures.

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, where the extent of investment in flood risk reduction measures is proportional to the flood risk, as described here. Thus, a large investment in flood risk management may be warranted even if the chance of flooding is small but the consequences are great, 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.

3.2 Flood Risk Management Measures

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

The nonstructural measures would involve fully integrating the project construction team with the existing Delta flood preparation, response, and recovery system. 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. The options considered for managing flood risk at the launch, reception, and maintenance sites during construction must be evaluated in the context of the existing Delta flood risk management system, which in turn is embedded in statewide, national, and international systems that are documented elsewhere, as summarized in the *Flood Risk Management TM* (DCA 2021b).

Structural measures would also be integrated with the design of the tunnel infrastructure so the short-term risks during construction, and the long-term risks during the subsequent operations of the facility were minimized. The most significant hazards would be due to hydrologic, hydraulic, and seismic (earthquake) loading. Several structural remediation measures (such as setback levees, ring levees, and geometry repairs) would be available to reduce the flood risk, based on the identified hazard. These measures are summarized in the *Flood Risk Management TM* (DCA 2021b).

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

As discussed, this supplemental TM addresses portions of the Bethany Reservoir Alternative alignment not addressed in the *Flood Risk Management TM* (DCA 2021b). As such, flood risks on portions of the project from the Intakes to Lower Roberts Island are not discussed, as they are unchanged from a flood risk management perspective. This supplement only addresses the maintenance shaft sites on Upper Jones Tract and Union Island, and the reception shaft site on the southern side of Byron Highway at the BRPP and Surge Basin.

It is anticipated that the elevated earthen pads for construction at the shaft sites could be constructed relatively quickly, within a time frame of about 1 to 2 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 above the 100-year flood elevation. Subsequently, a vertical shaft would be constructed for launching or accessing the tunnel. In case of a sudden and unforeseen levee breach, workers at the shaft sites would generally be protected at pad elevations but would face the potential chance 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 case rapid site evacuation is needed. In extereme cases, the use of helicopters to rescue the workers and project staff may be implemented.

Based on review of conditions at the Upper Jones Tract and Union Island, maintenance shaft sites, repairs to existing levees do not appear warranted. The need for levee repairs to mitigate flood risks should be further evaluated before construction. If repairs to existing levees are ultimately recommended, the levee improvements should be initiated at the beginning of project construction activities, and may overlap to some extent the initiation of shaft construction at the shaft sites. However, if critical weaknesses were identified in these levee systems, remediation would be completed before shaft sites were constructed. 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 reception shaft site on the southern side of Byron Highway is at an elevation approximately 25 to 30 ft higher than the flood stage of the surrounding channels, and thus no levees repairs would be required.

3.3.1 Flood Inundation Analysis

Levees in the Delta are exposed to many hazards that may damage them or cause failure, resulting in flooding. The unique geographical, topographical, and hydrological characteristics of each shaft site affect the level of flood risk at that site. Important determinative factors are: the likely speed of flooding (and thus escape and rescue time windows), the likely depth of flooding, available evacuation routes, and the extents to which the flood risk varies over the seasons. Details about how these variables are evaluated are summarized here.

3.3.1.1 Speed of Flooding

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 (refer to the *Flood Risk Management TM* [DCA 2021b]).

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 will depend on 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 including the surface area being flooded.

A preliminary estimate of the time to flood for each of the shaft sites has been completed, as described in Attachment 3, Flood Inundation Analysis. To determine the inundation characteristics, a simplified approach was used to get a range of impact times and elevations at each shaft site. This analysis used work performed previously by the DRMS study team (URS 2008b) to determine the average levee breach geometry (depth and width)[1]. The 100-year WSEs 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 WSE. However, the timing of water flowing through a levee breach is not anticipated to change substantially. Therefore, the historical values for WSE have been used in this analysis.

3.3.1.2 Maximum Depth of Flooding

The flood danger at a shaft site generally increases 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 flood elevation 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.

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 one hour. This range is shown in Table 3-1 to reflect the substantial uncertainty involved in estimating the progression of future levee breaches. Shaft pad elevations were developed based on the methodology identified in the *Flood Risk Management TM* (DCA 2021b). This table only includes shaft sites not studied in the *Flood Risk Management TM* (DCA 2021b).

[1] In the DRMS analysis, 14 breach scour holes remaining from historical 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.

Table 3-1. Flood Inundation Analysis Summary for Bethany Shaft Sites

Site Description	Ground Elevation at Shaft Site (ft)	Top of Shaft Pad Elevation (ft)	Time to Impact Shaft Site		Depth at Shaft Site at 1-hr		Maximum Flood Elevation (ft)	Maximum Depth of Flooding (ft)
			Min (hr)	Max (hr)	Max (ft)	Min (ft)		
Reception Shafts at:								
Surge Basin shaft	40.0[a]	40.0	-	-	-	-	-	-
Maintenance Shafts at:								
Upper Jones Tract	-3.0	13.0	3.5	8.2	-	-	9.7	12.7
Union Island West	-4.9	12.0	0.6	1.4	0.2	-	10.0	14.9

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

Notes:

- = not applicable

hr = hour(s)

Max = maximum

Min = minimum

3.3.1.3 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 be preferable if it was safe to do so. 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 effected 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 itself, because the site of the breach poses 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 impact upon the site, regardless of the rate of the island-filling, simply due to the overland flow of the floodwaters from the breach.

3.4 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 one 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 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.

All of the risk factors taken together are considered in arriving at the risk rating shown in Table 3-2, along with site-specific observations and risk mitigation recommendations.

The risk factors for the Twin Cities Complex launch are described in the *Flood Risk Management TM* (DCA 2021b). As with the Central and Eastern alignments, 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. A flood impact 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 4.

Table 3-2. Site-specific Cumulative Safety Rating for Shaft Pad Sites

Site Description	Shaft Type	Site Characteristics	Ground Elevation at Shaft (ft)	Minimum Time to Impact Pad (hr)	Maximum Depth at Pad at 1-hr (ft)	Maximum Depth of Flooding (ft)	Flood Risk Rating	Recommended Structural Flood Risk Mitigations (Nonstructural Flood Risk Mitigations Apply to all Shaft Sites)
Upper Jones Tract	Maintenance	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
Union Island	Maintenance	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	Reception	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.

Notes:

H = high

L = low

M = medium

3.5 Flood Risk Management Observations and Conclusions

This section of the TM supplements the *Flood Risk Management TM* (DCA 2021b) to include relatively small portions of the Bethany Reservoir Alternative alignment that were not previously evaluated. Key observations and conclusions from this supplemental TM are listed here:

- A combination of nonstructural and structural flood risk management measures can be employed to manage the risk of flooding at the maintenance shaft sites.
- Nonstructural flood risk management includes several measures, such as involving appropriate agencies, flood preparedness, emergency response, and post-flood recovery operations. These measures should be employed at the maintenance shaft sites.
- Flood risk for construction personnel and equipment can be substantially reduced by employing nonstructural measures such as coordinating and cooperating with levee maintenance agencies; providing Standardized Emergency Management Systems (SEMS), National Incident Management Systems (NIMS), 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.
- A flood impact 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 minimizes potential adverse flooding impacts to the surrounding area.
- Upper Jones Tract maintenance shaft location was rated as having moderate flood risk relative to other potential project sites throughout the Delta, as described in the Flood Risk Management TM (DCA 2021b). The shaft site does have the potential for deep flooding, but the probable time to impact is relatively high due to the large size of the island and the relative high elevation of the shaft site. This is a maintenance shaft site, so it would not be occupied for periods as extended as the reception and launch shafts. Recommended structural flood risk mitigation measures include:
  - Assure all weather access from site to Bacon Island Road to allow for early evacuations from the site.
  - Provide onsite flood refuge for personnel to shelter in place during pad construction as described.
- Union Island levees appear to be fairly robust compared to levees throughout the Delta as discussed in the *Levee Vulnerability Assessment TM* (DCA 2021a). However, the potential shaft site is near a levee, and thus may be subject to extreme flows in the event of a levee breach on an adjacent levee. The Union Island shaft site could be inundated in under an hour. As a result of potential inundation time and depth, the Union Island shaft site was rated has having high flood risk. This is a maintenance shaft site, so it would not be occupied for periods as extended as the reception and launch shafts. Recommended structural flood risk mitigation measures include:
  - Assure all weather escape route to levees adjacent to Victoria Canal to allow for early evacuations from the site.
  - Provide onsite flood refuge for personnel to shelter in place during pad construction as described.
- The surge basin reception shaft is approximately 25 to 30 ft higher in elevation than the 100-year base flood elevation; thus, riverine flooding is not possible in the return intervals considered and

accordingly rated as having low flood risk. Normal inclement weather precautions for major construction sites would be developed during design and construction.

4. References

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URS Corporation (URS) and Jack R. Benjamin and Associates. 2008b. Technical Memorandum: Delta Risk Management Strategy (DRMS) Phase 1, Topical Area: Levee Vulnerability, Final. Prepared for the California Department of Water Resources.

URS and Benjamin and Associates. 2008c. Technical Memorandum: Delta Risk Management Strategy (DRMS) Phase 1, Topical Area: Levee Vulnerability, Final. Prepared for the California Department of Water Resources.

5. Document History and Quality Assurance

Reviewers listed have completed an internal quality review check and approval process for deliverable documents that is consistent with procedures and directives identified by the Engineering Design Manager (EDM) and the DCA.

Approval Names and Roles			
Prepared by	**Internal Quality Control review by**	**Consistency review by**	**Approved for submission by**
Michael Conant / EDM Senior Engineer	Graham Bradner / DCA Executive Director	Gwen Buchholz / DCA Environmental Consultant Phil Ryan / EDM Design Manager	Terry Krause / EDM Project Manager

This interim document is considered preliminary and was prepared under the responsible charge of Ernest Michael Conant, California Professional Engineering License C79228.

Attachment 1
Levee Geometry Standards

Attachment 1. Levee Geometry Standards

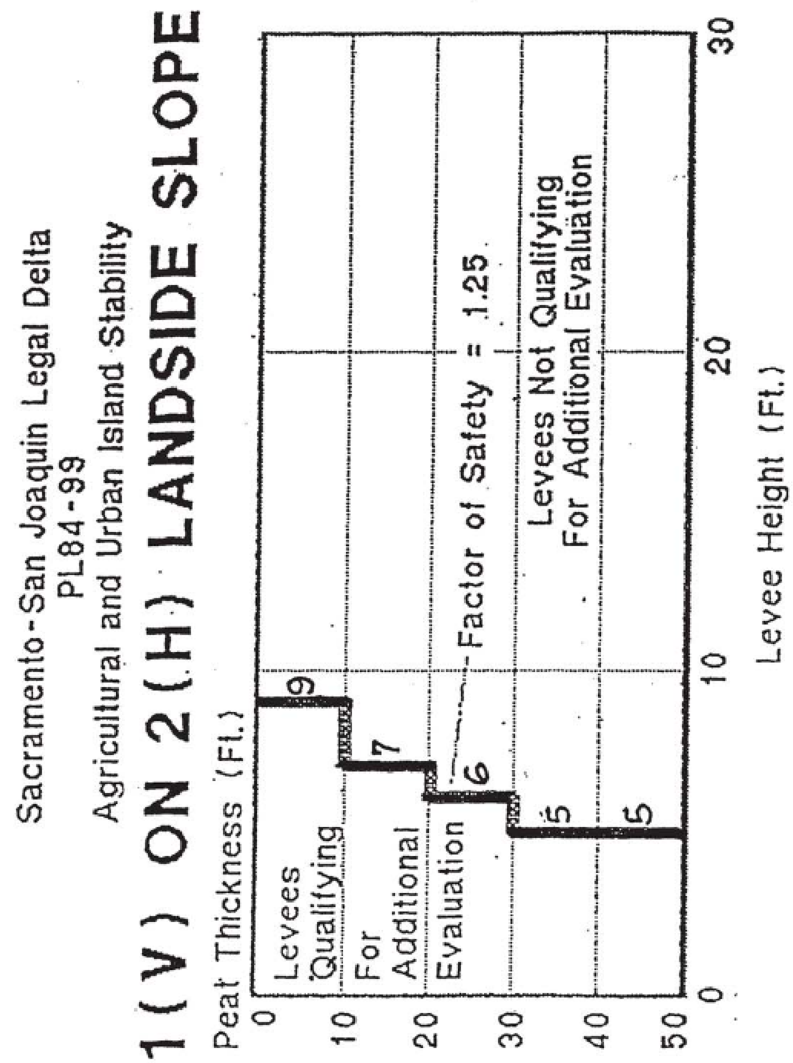
This attachment is a presentation of levee geometry standards relating peat/organics thickness and levee height to allowable landside levee or berm slopes for Public Law 84-99 (USACE 1987) and DWR Bulletin 192-82 (DWR 1982) and (DWR 1989). Content excerpted from these standards includes:

- Public Law 84-99 – Four charts showing the required landside height versus peat thickness for a specific landside levee slope. Each chart is developed for a different landside slope which include 2H:1V, 3H:1V, 4H:1V, and 5H:1V. This geometry standards applies to both urban and non-urban levee systems.
- DWR Bulletin 192-82 – Four charts which present the minimum landside slope or berm slope based on levee height, presence of berm, contours of peat thickness and land use. Figures on page Att 1-6 and Att 1-8 are for urban tracts and presented for completeness. Figures on page Att 1-7 and Att 1-9 present the reference standards for agricultural (non-urban) tracts which were used for the geometry assessment in this TM.

D-031946

D-031946

PL 84-99



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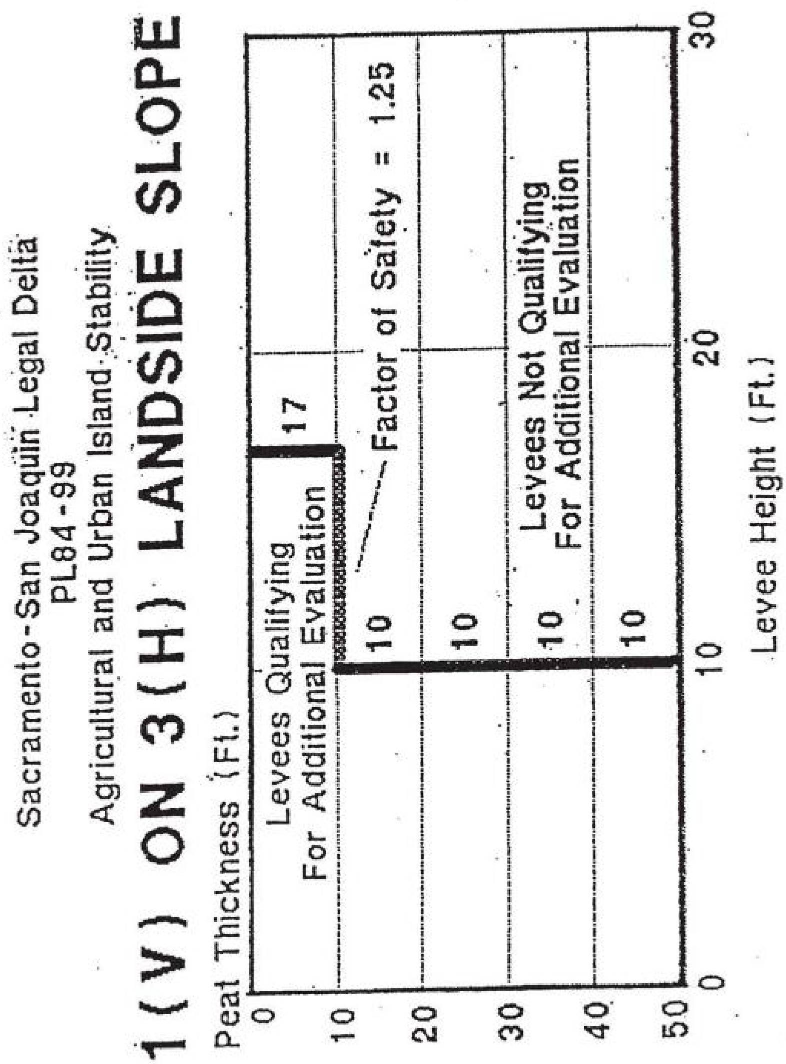
D – 0 3 1 9 4 7

PL 84-99



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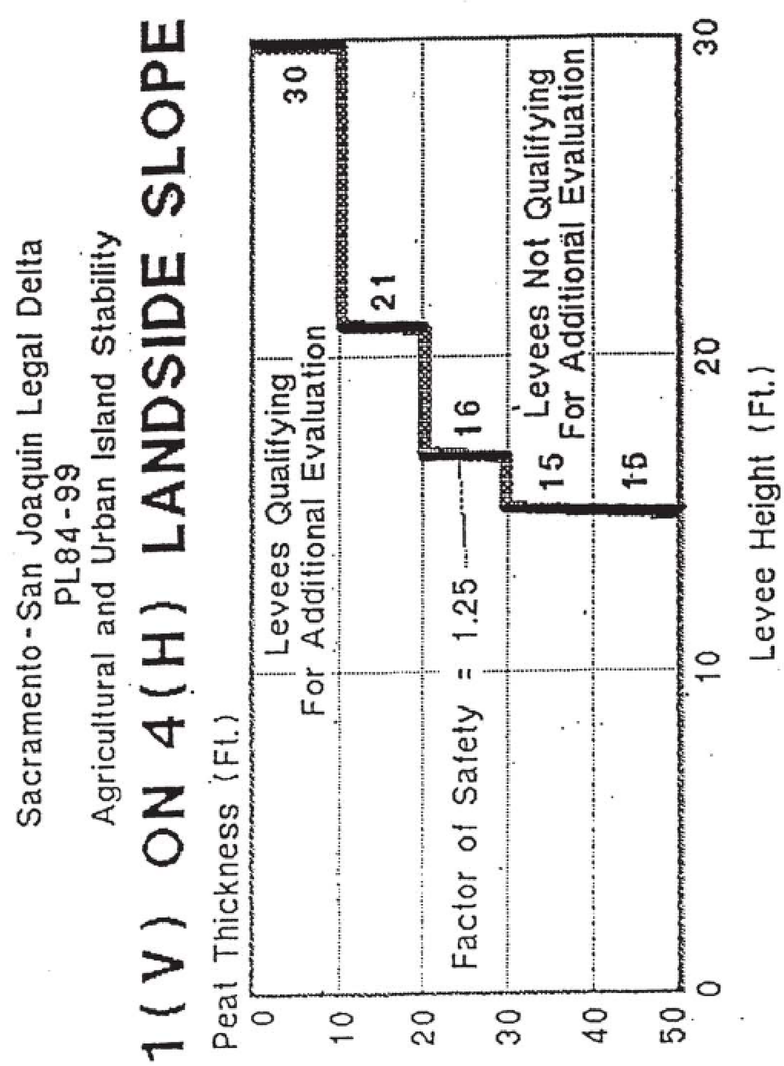
PL 84-99



D-031949

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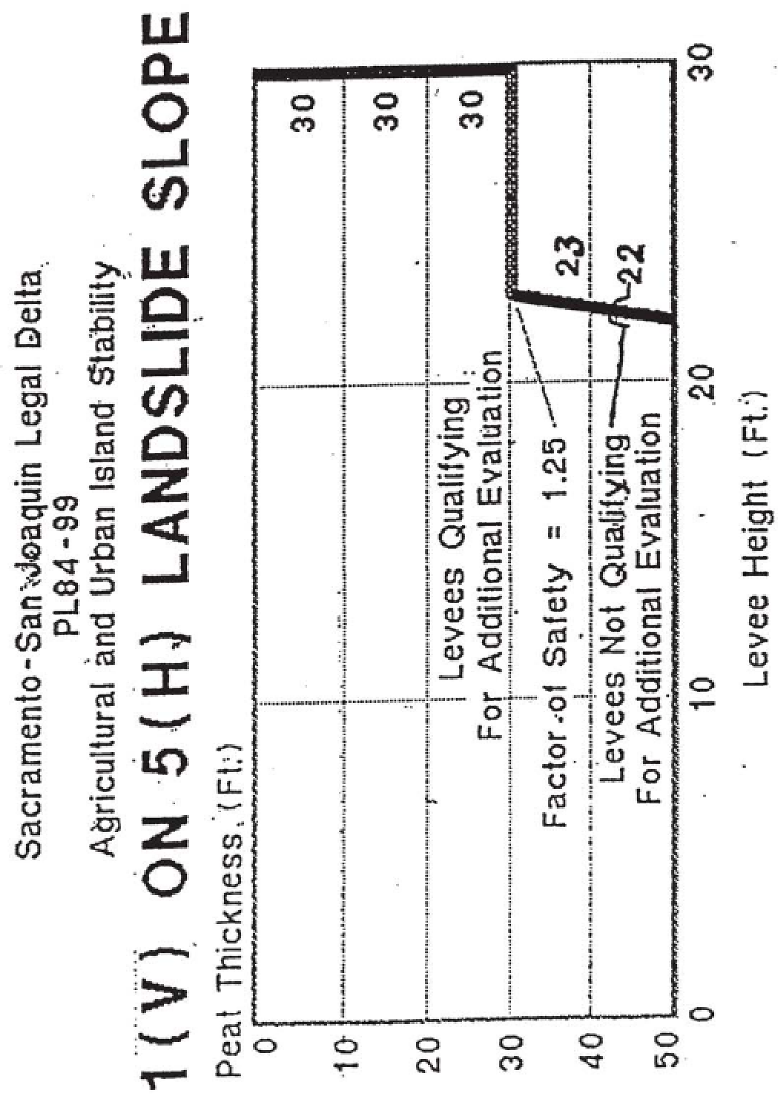
PL 84-99



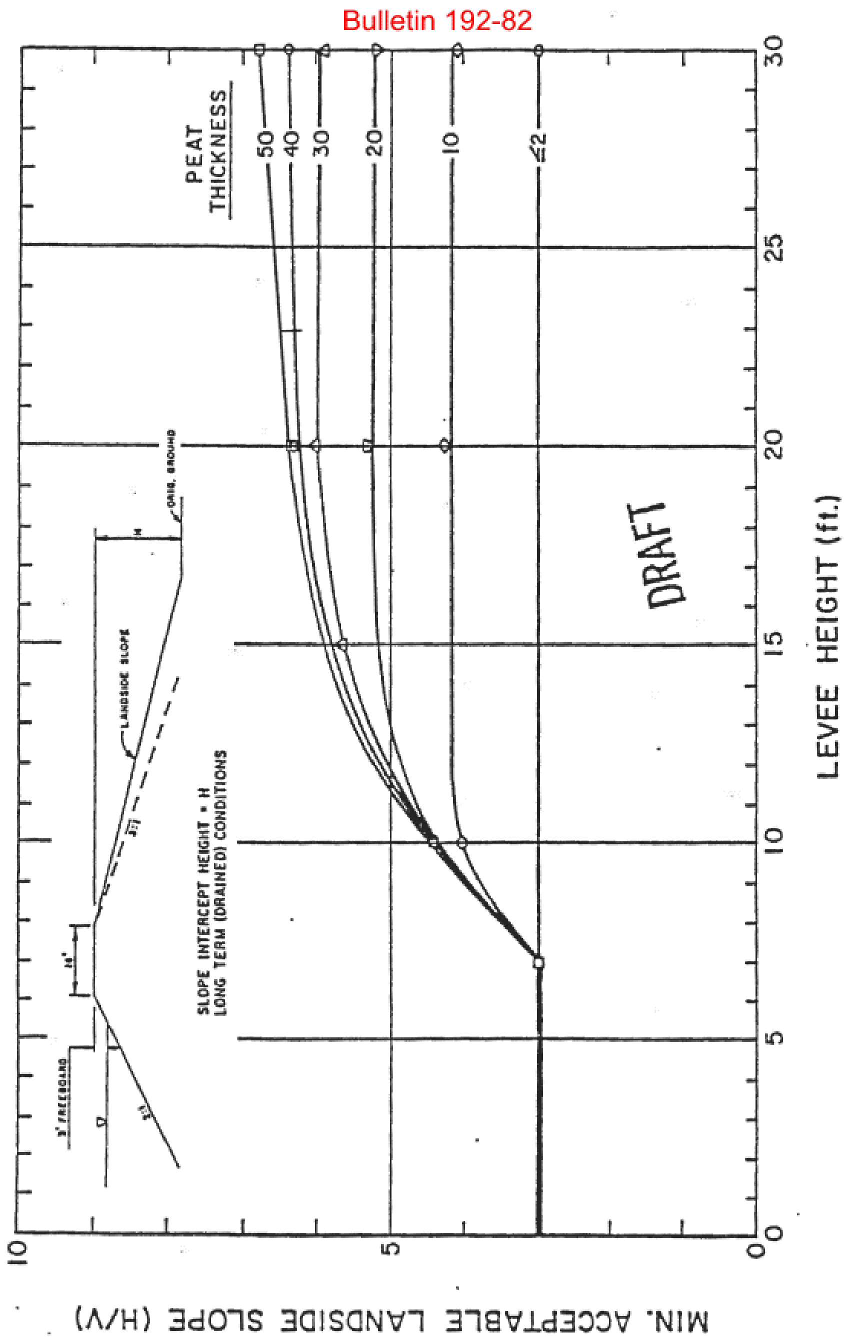


FIGURE 1: MINIMUM ACCEPTABLE LANDSIDE SLOPES FOR LEVEES PROTECTING URBAN TRACTS

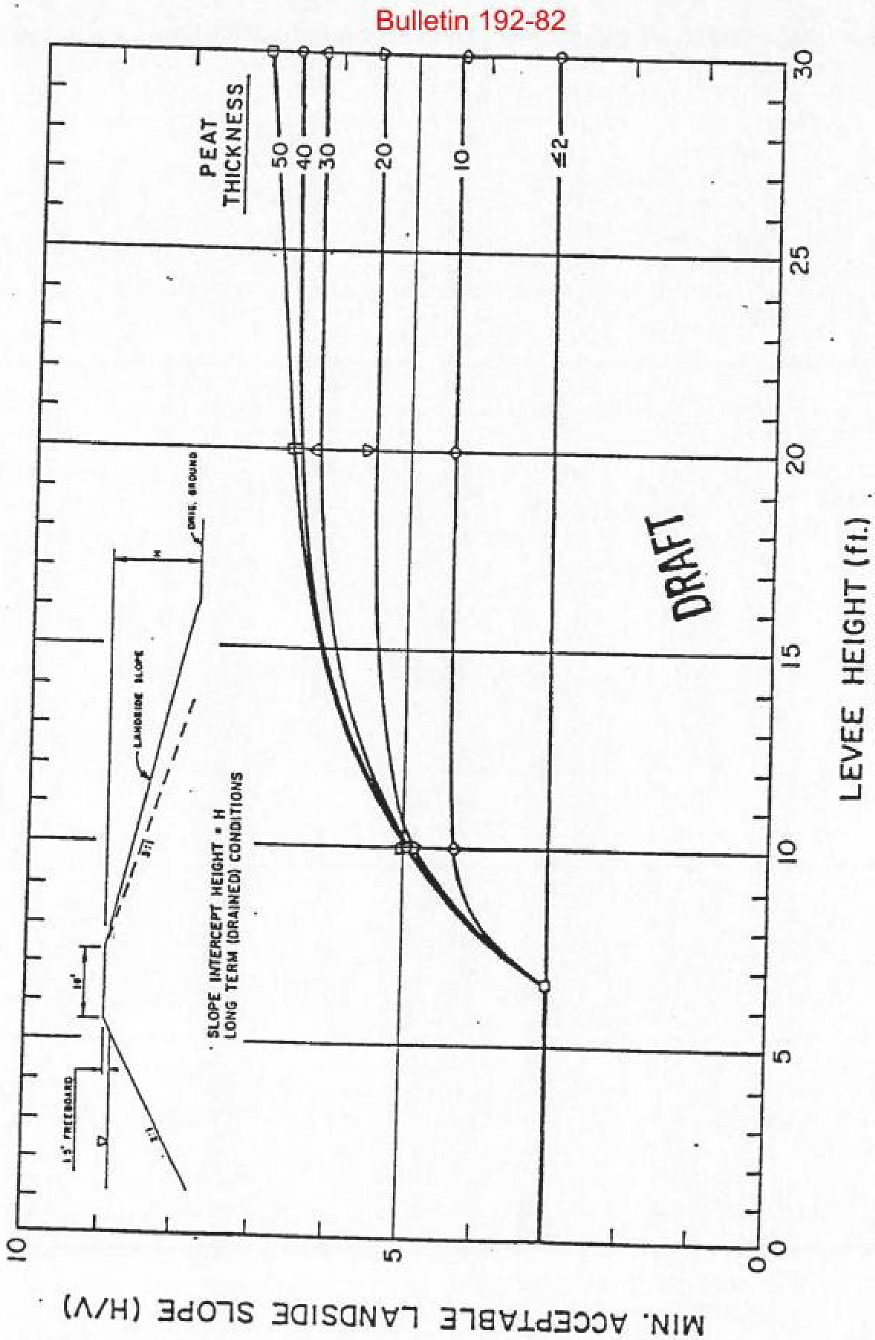


FIGURE 2: MINIMUM ACCEPTABLE LANDSIDE SLOPES FOR LEVEES PROTECTING AGRICULTURAL TRACTS

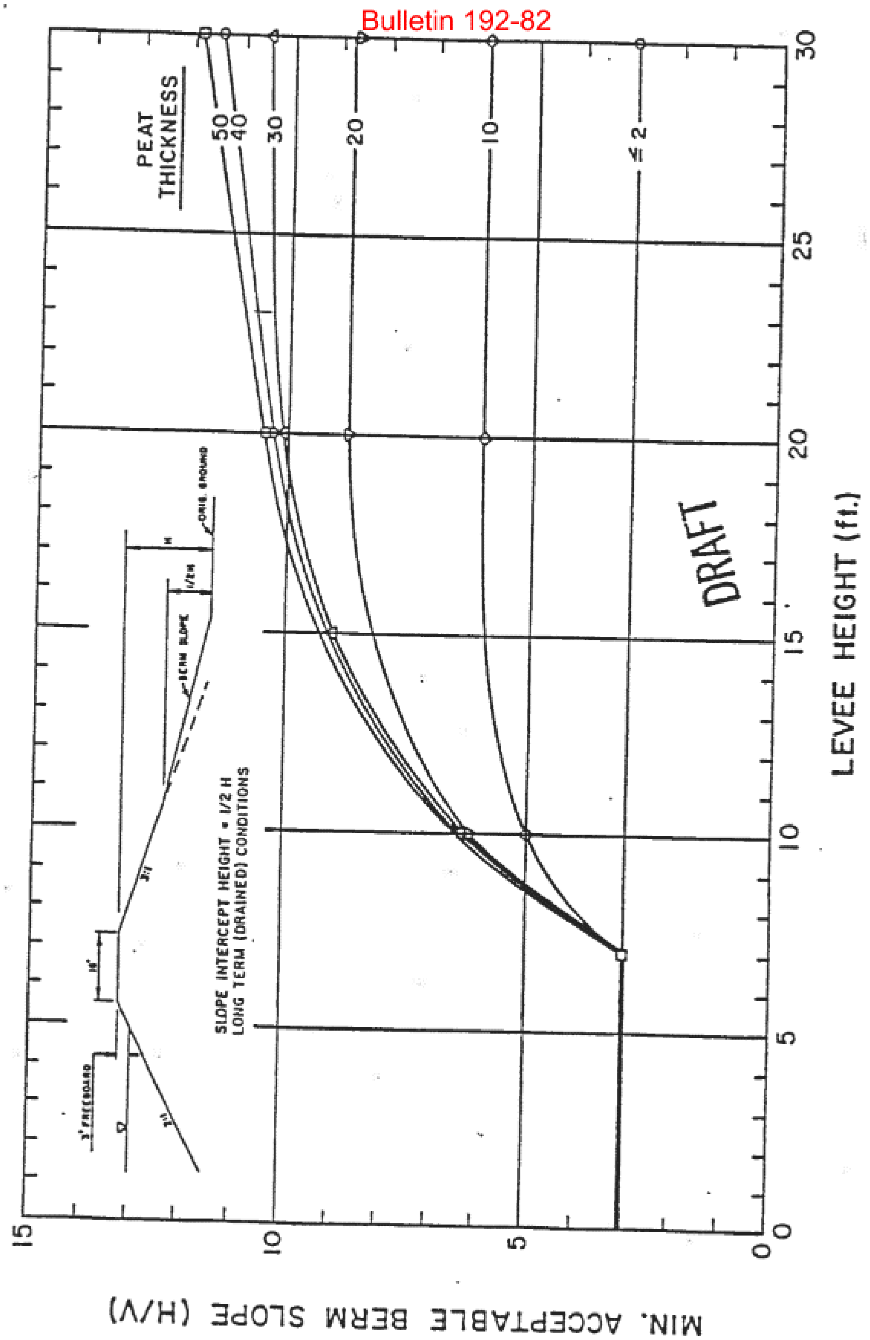


FIGURE 3: MINIMUM ACCEPTABLE BERM SLOPES FOR LEVEES PROTECTING URBAN TRACTS

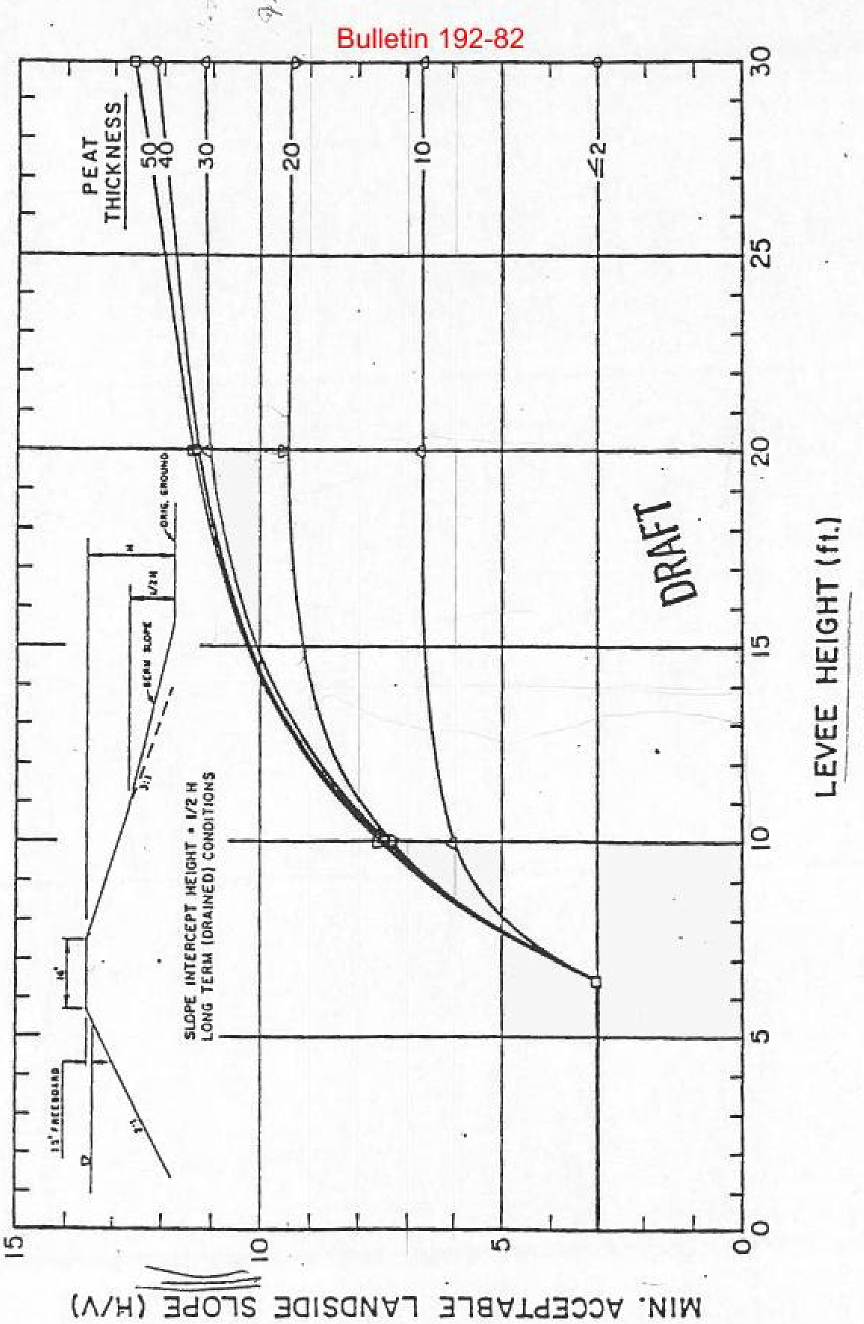
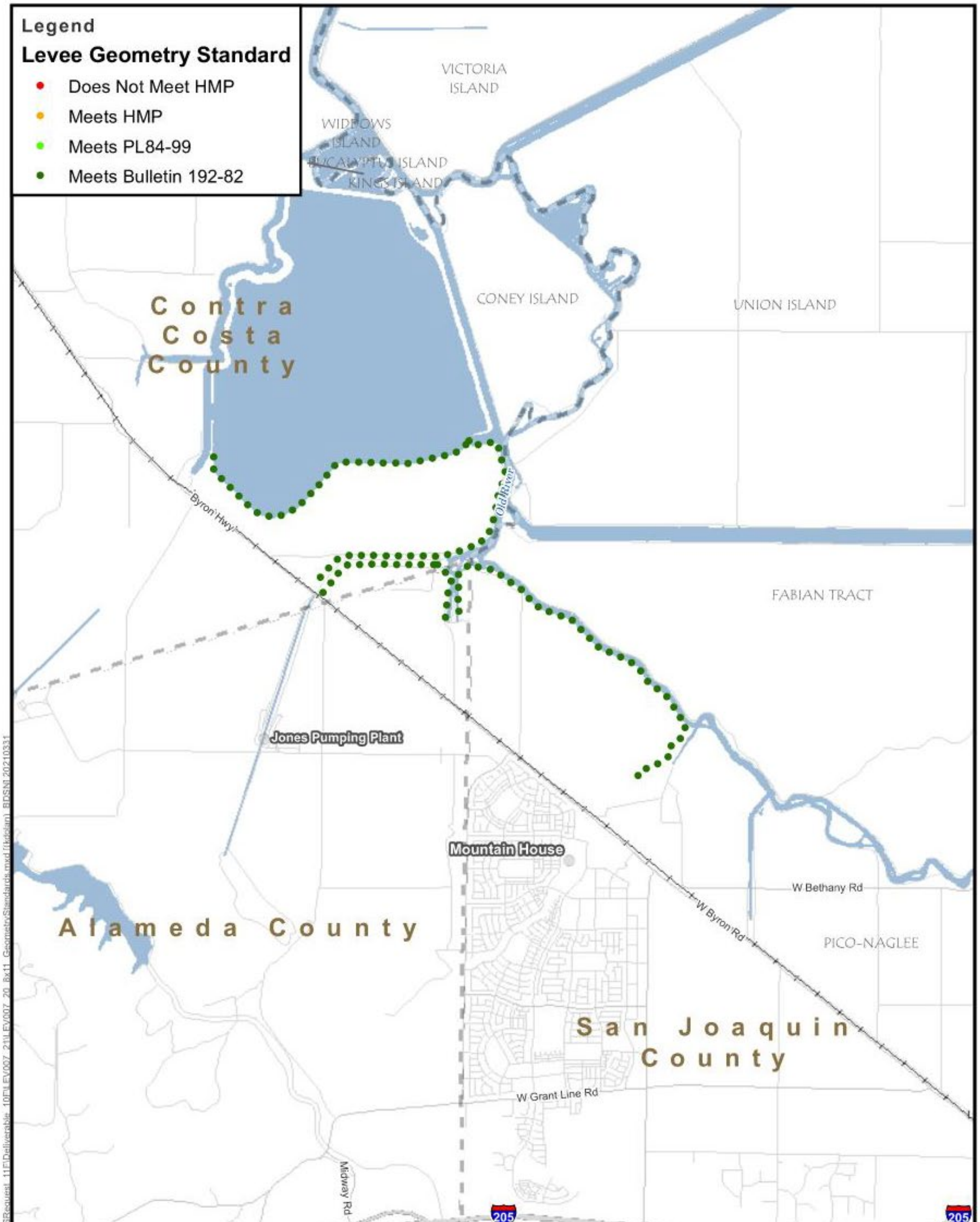


FIGURE 4: MINIMUM ACCEPTABLE BERM SLOPES FOR LEVEES PROTECTING AGRICULTURAL TRACTS

Attachment 2
Levee Vulnerability Figures

Attachment 2. Levee Vulnerability Figures

This attachment is a graphical presentation of the relative levee vulnerability criteria and relative levee vulnerability results by cross-section location.



DCA DELTA CONVEYANCE DESIGN & CONSTRUCTION AUTHORITY For Illustration Purposes Only

N 0 1 Miles

Figure A2-1
Levee Geometry Standard Evaluation

Data Source: DCA, DWR

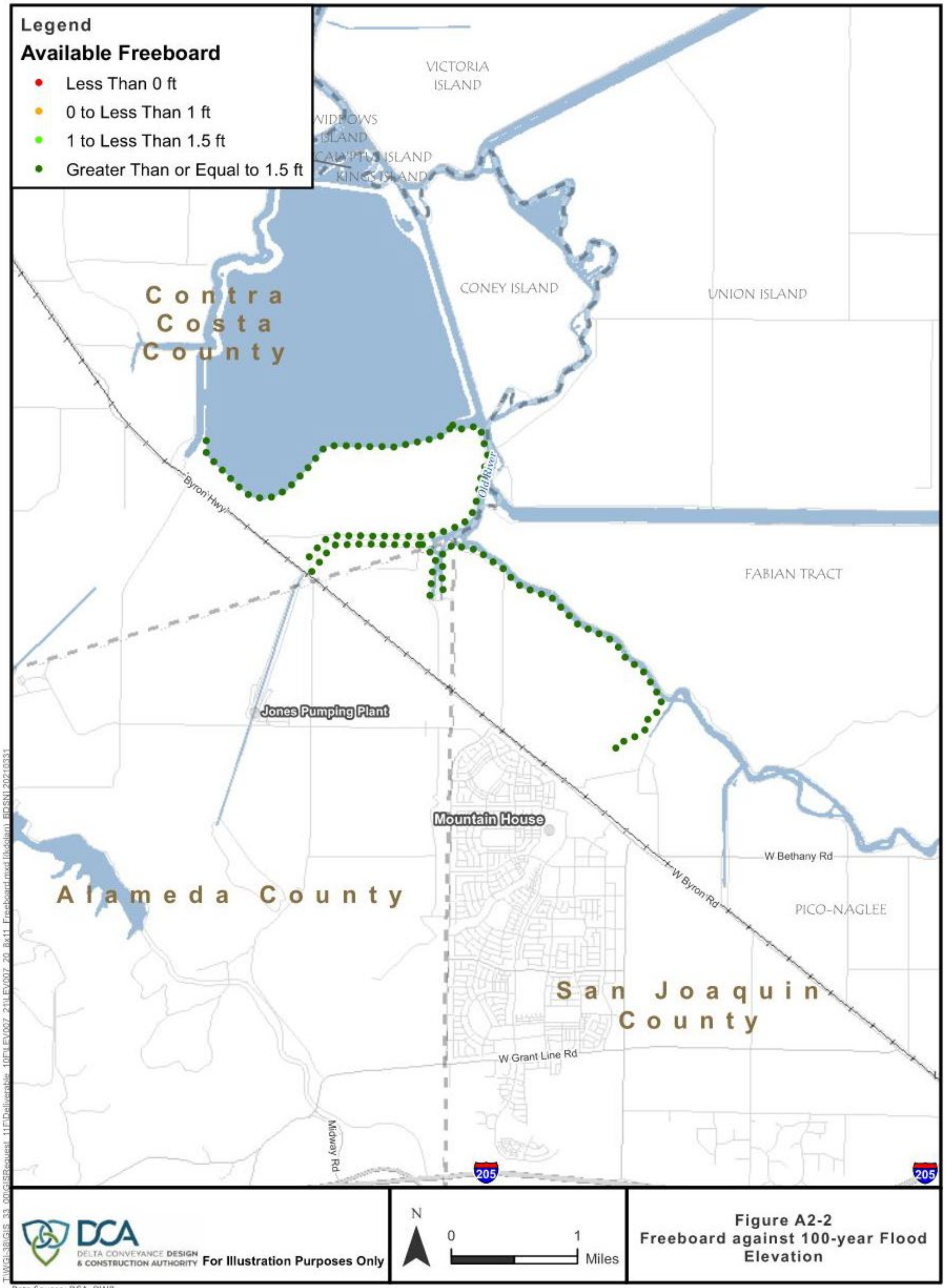


Figure A2-2
Freeboard against 100-year Flood Elevation

Data Source: DCA, DWR

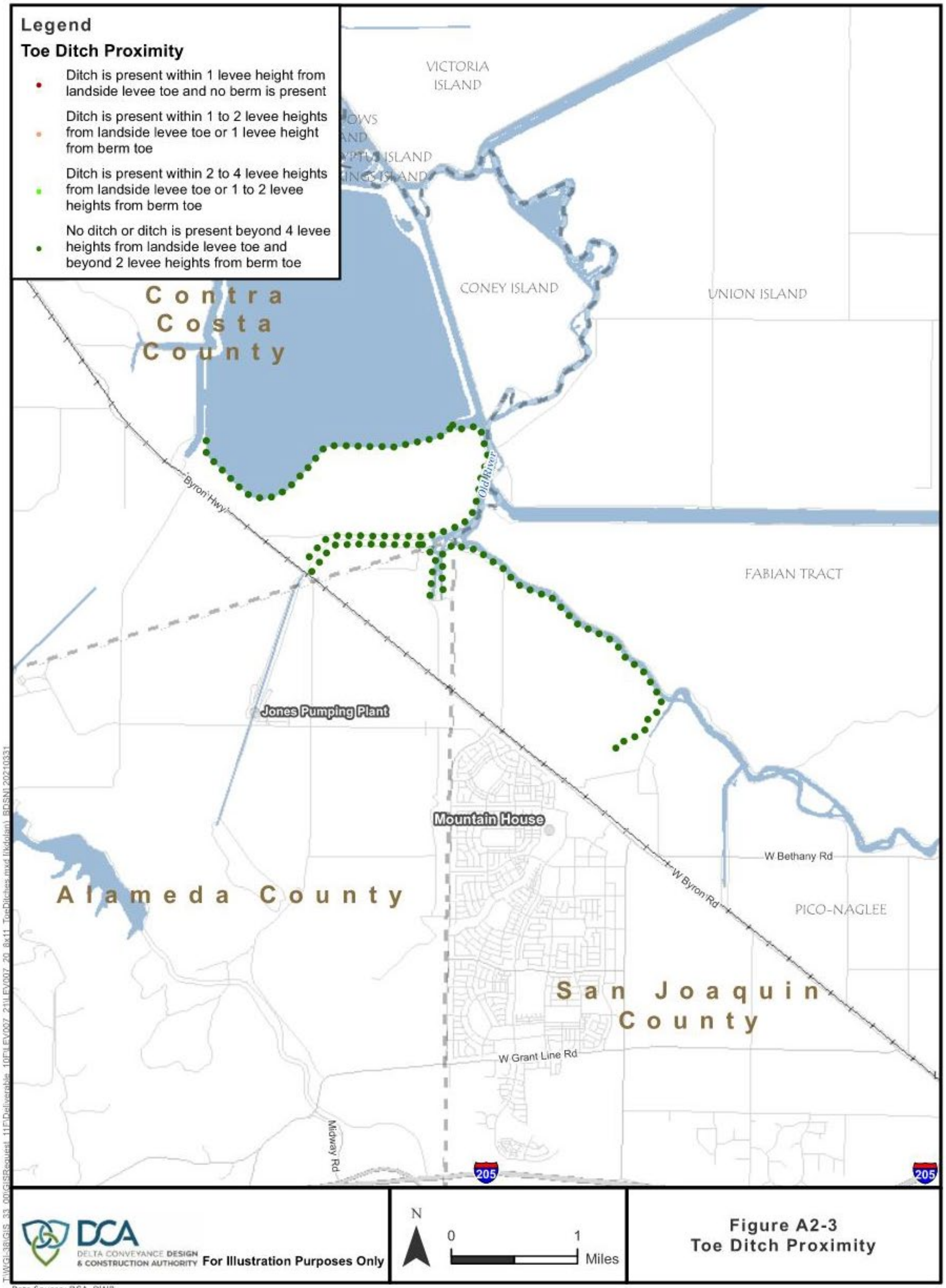


Figure A2-3
Toe Ditch Proximity

Data Source: DCA, DWR

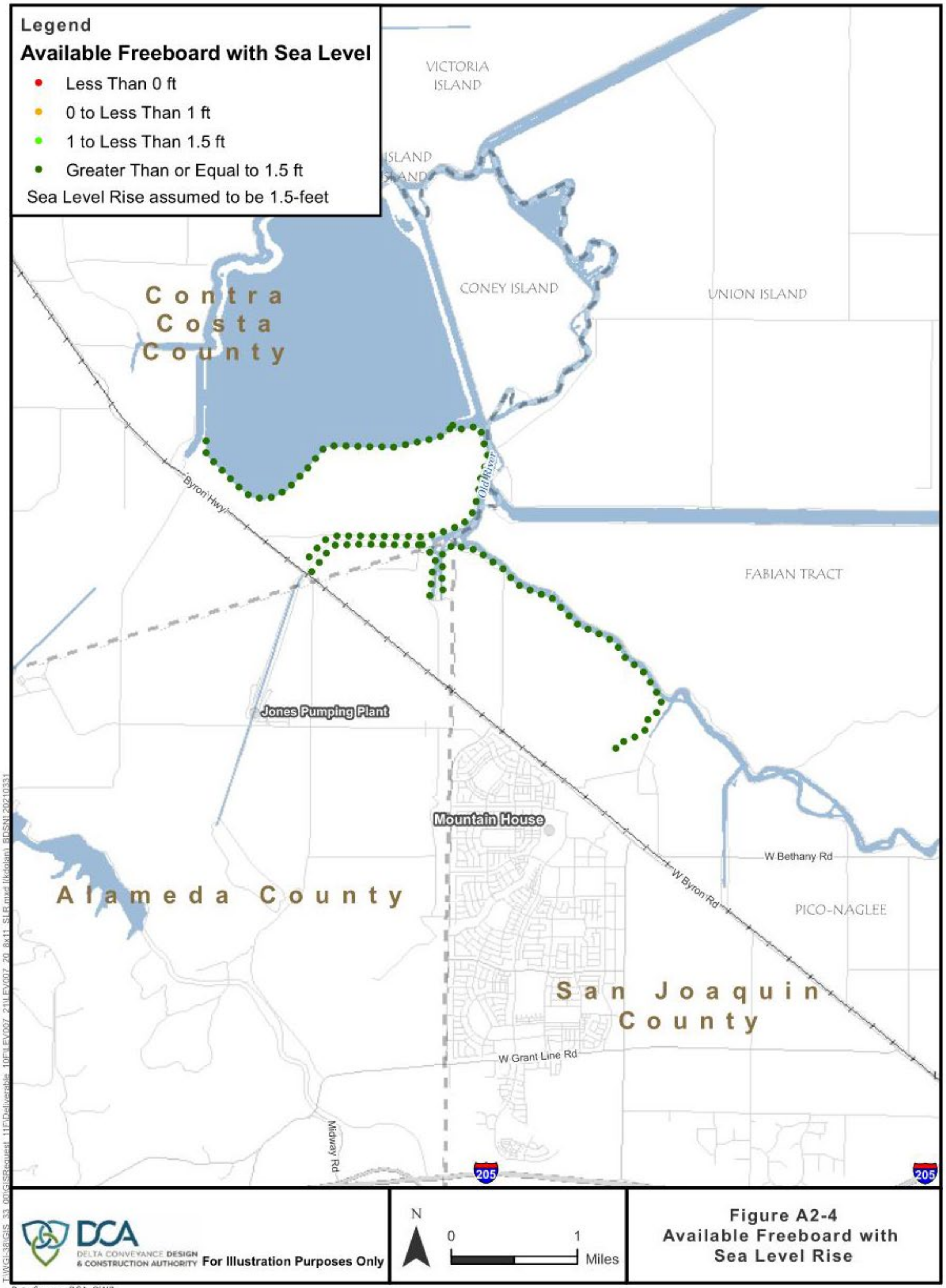


Figure A2-4
Available Freeboard with Sea Level Rise

Data Source: DCA, DWR

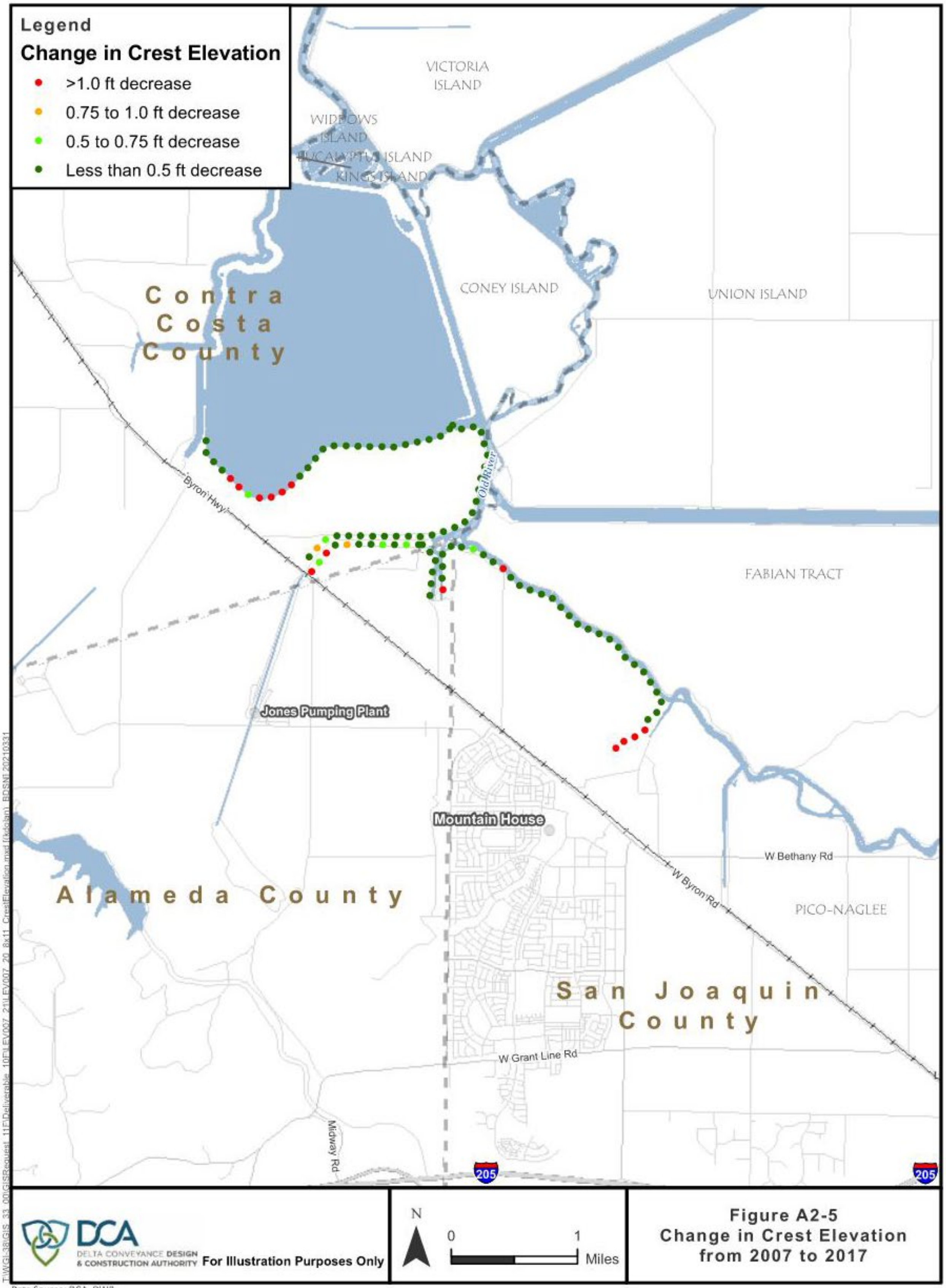


Figure A2-5
Change in Crest Elevation
from 2007 to 2017

Data Source: DCA, DWR

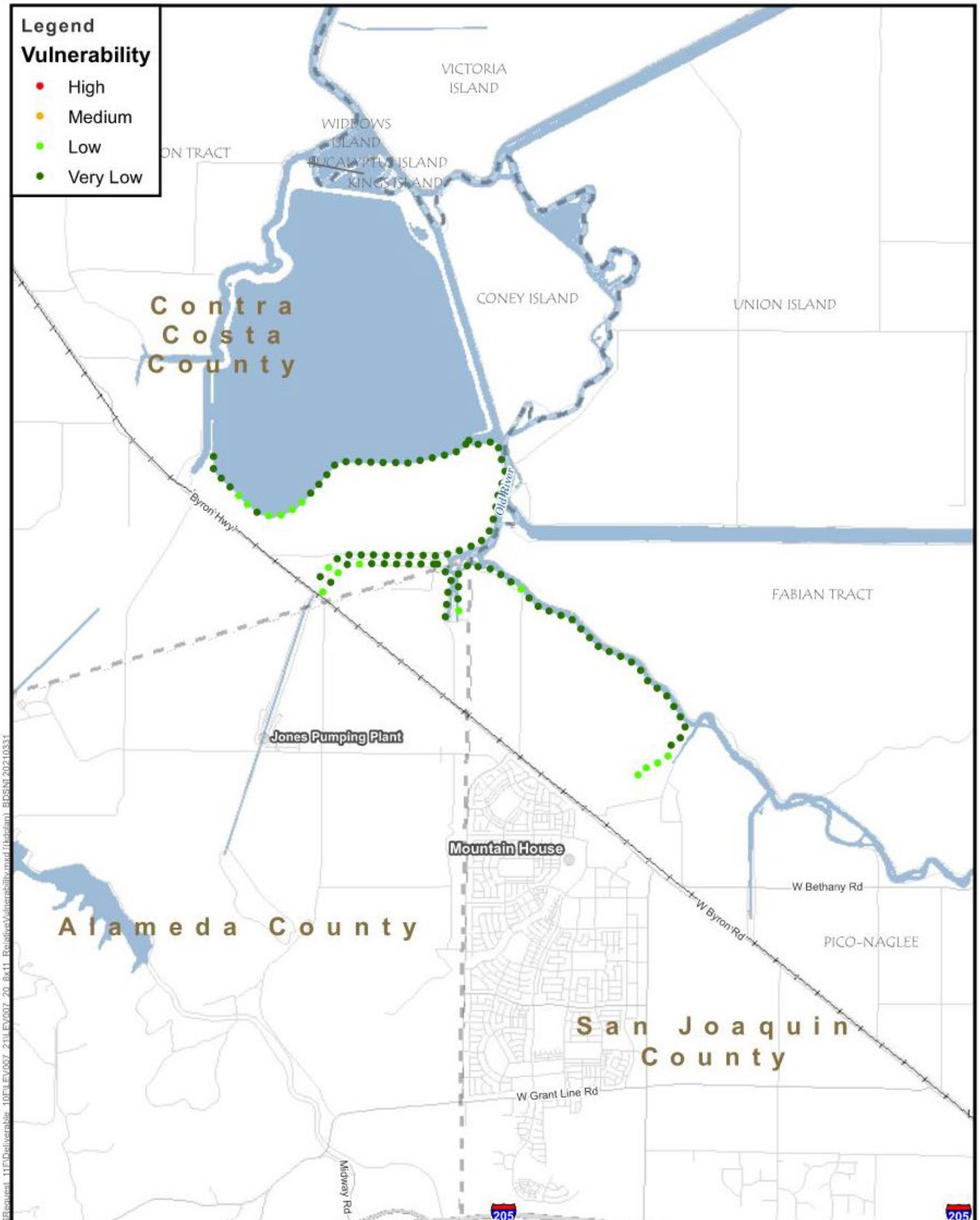


Figure A2-6
Relative Levee
Vulnerability Rating

Data Source: DCA, DWR

Attachment 3
Flood Inundation Analysis

Attachment 3. Flood Inundation Analysis

A preliminary estimate at each shaft pad site for the time to flood was conducted. To determine the inundation characteristics, a conservative and simplified approach was used to get a range of impact times and elevations at each shaft site. This analysis utilized work performed previously by the Delta Risk Management Strategy study team (URS 2008c) to determine the average levee breach geometry (depth and width)[2]. 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 (WSE) (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,

$$Q=C*L*H^{(3/2)}, \text{ where}$$

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 one 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 Delta Conveyance corridors. 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

[2] 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.

head differential (between the 100-year flood elevation 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[3] (C_d) of 0.4 and 0.2 were used, respectively (Lee, 2019). If floodwaters impacted shaft locations in under an hour, the floodwater depth was also given to highlight the severity of the flooding. Table A3-1, below, shows the minimum (worst-case) and maximum (best-case) estimated time to impact 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.

Of the shaft sites analyzed in this supplement, only Union Island West was possibly inundated in less than an hour, and with potential flood depths of 0.2 feet. Table A3-1 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 shaft locations. Figures A3-1 and A3-2 of this attachment show the time to inundate the pad and time to reach maximum flooding depth for both best-case and worst-case scenarios for both maintenance shafts.

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

Table A3-1. Detailed Calculations for Inundation Analysis at Launch, Reception and Maintenance Shafts

Site Description	Alignment	Ground Elevation at Pad Site (ft)	Elevation for 66% of Total Head	Minimum LS Toe Elevation (ft)	100-year WSE at Min LS Toe (ft)	Lowest Crest Elevation (ft)	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 Impact Pad, min (hours)	Time to Impact Pad, max (hours)	Maximum Depth at Pad @ 1 hr (ft)	Minimum Depth at Pad @ 1 hr (ft)	Maximum Depth of Flooding (ft)
Surge Basin Shaft	Reception	38.0	NA	NA	10.3	NA	0.4	0.2	1.9	0.8	NA	NA	NA		NA	NA	NA	NA	-	NA
Upper Jones Tract	Maintenance	-3.0	2.2	-12.3	9.7	9.8	0.4	0.2	1.9	0.8	22.0	14.5	500.0	96610.4	41404.4	3.5	8.2	-	-	**12.8**
Union Island West	Maintenance	-4.9	4.1	-7.3	10.0	14.4	0.4	0.2	1.9	0.8	17.3	11.4	500.0	67368.8	28872.3	0.6	1.4	**0.2**	-	**19.3**

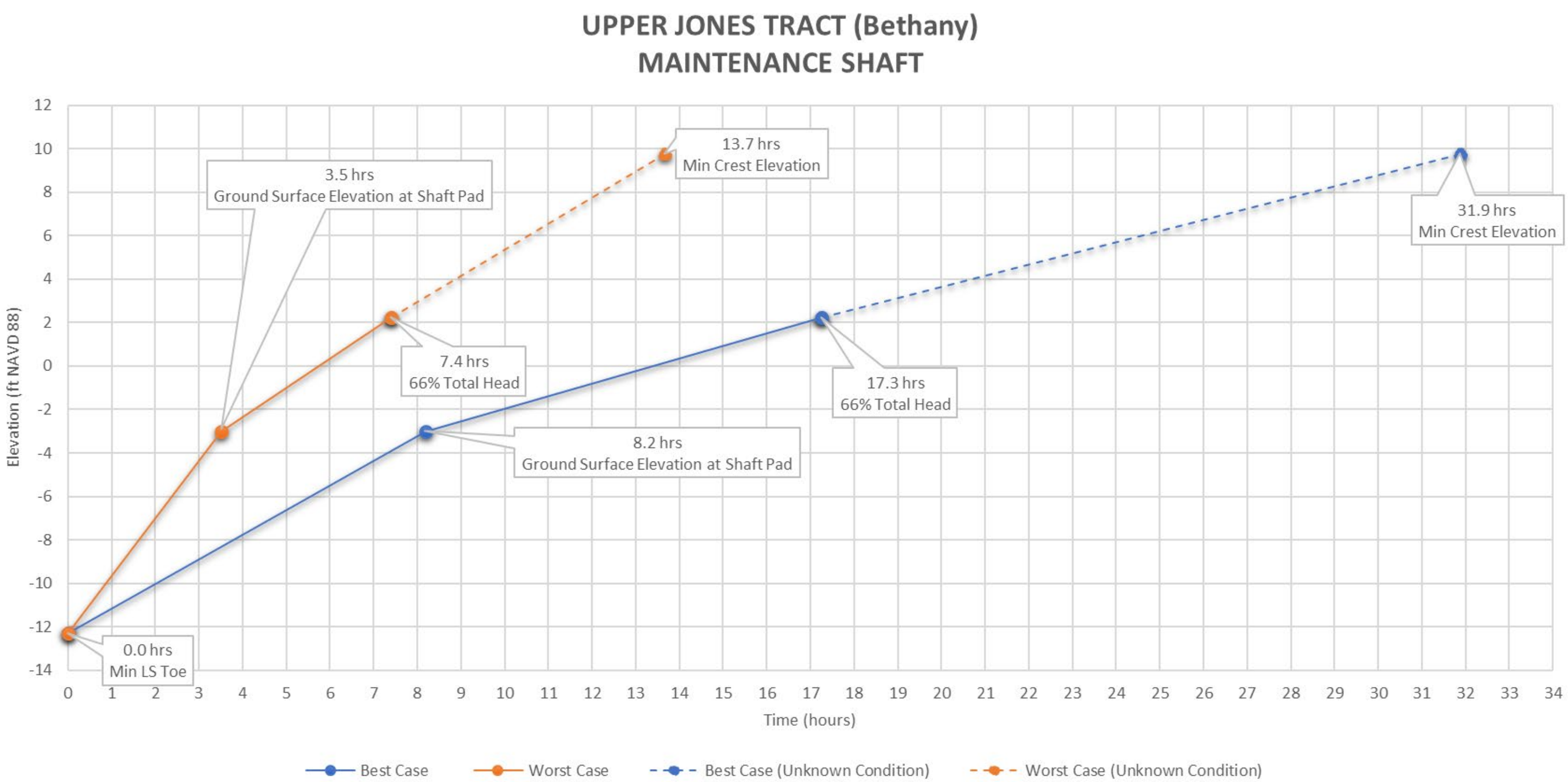


Figure A3-1. Time to inundate pad and time to reach maximum flooding depth for best case and worst-case scenario at Upper Jones Tract (Bethany) Maintenance Shaft

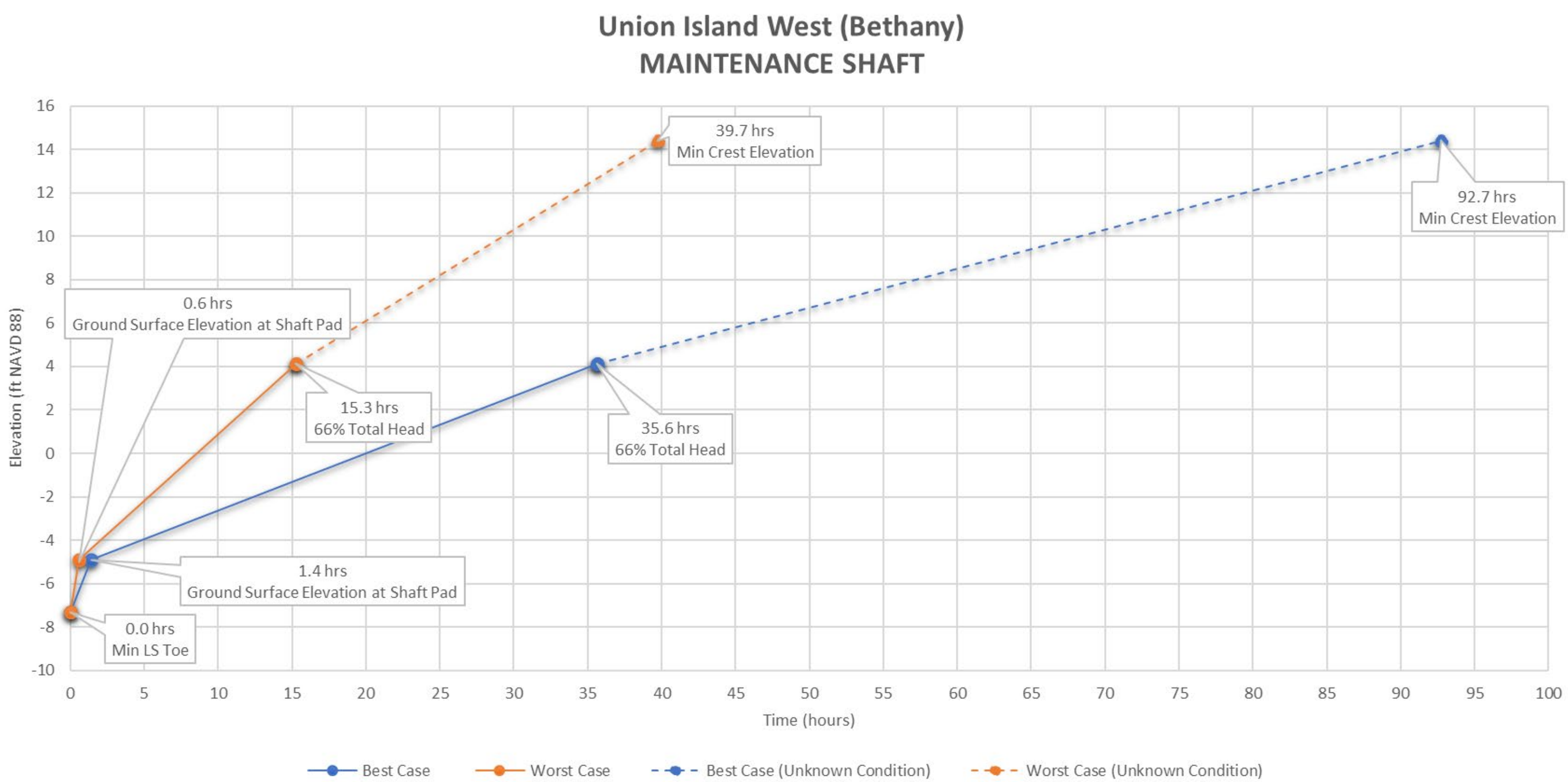


Figure A3-2. Time to inundate pad and time to reach maximum flooding depth for best case and worst-case scenario at Union Island (Bethany) Maintenance Shaft

Attachment 4
Twin Cities Complex Flood Analysis

Attachment 4. Twin Cities Complex Flood Analysis

The purpose of Attachment 4 is to present the 100-year storm frequency flood results due to the construction of a ring levee to protect the Twin Cities Complex for the Bethany alignment. The methodology followed for this analysis is identical to that done for the Central and Eastern alignments. Information regarding methodology can be found in Attachment 3 of the *Flood Risk Management Technical Memorandum (DCA, 2021b)*

4.1 Hydraulic Model Scenarios

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

Scenario 1) Hydraulic model Scenario 1 shown on Figure A4-1 presents the Bethany Reservoir Alternative (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 A4-2 shows the permanent stockpile for the Bethany Reservoir Alternative (6,000 cfs project design capacity) after the ring levee would have been removed. The top of the stockpile would be elevated above the existing 100-year floodplain.

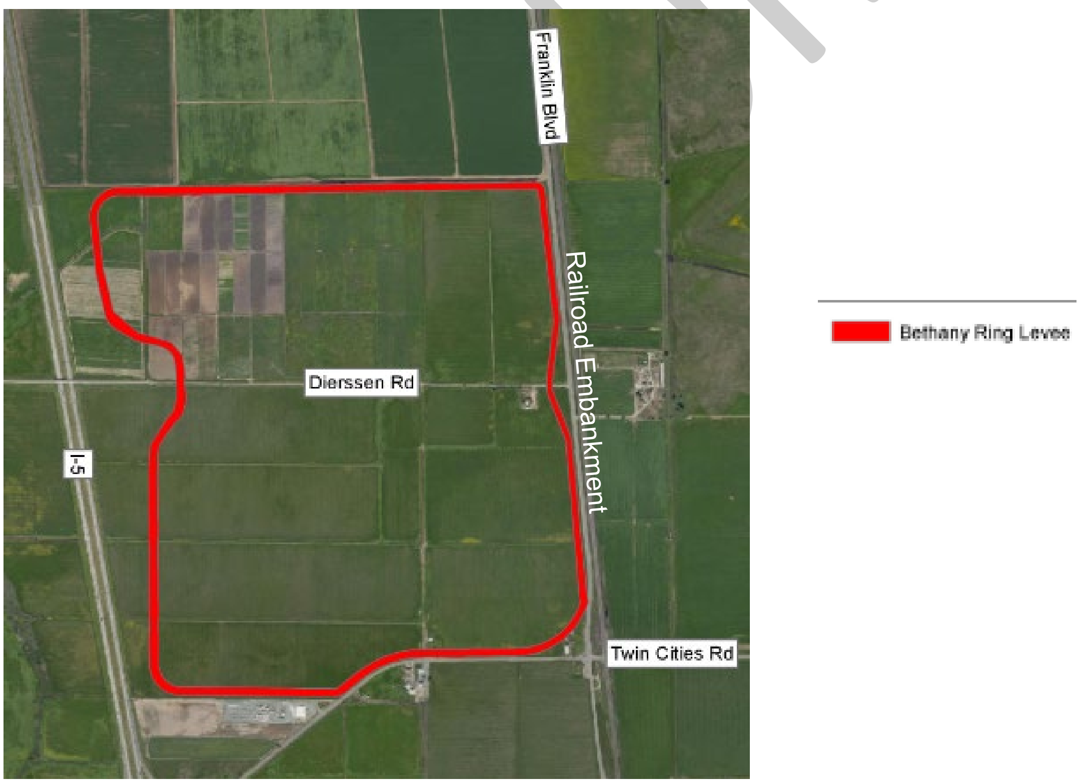


Figure A4-1. Bethany Reservoir Alternative Ring Levee at the Twin Cities Complex Site

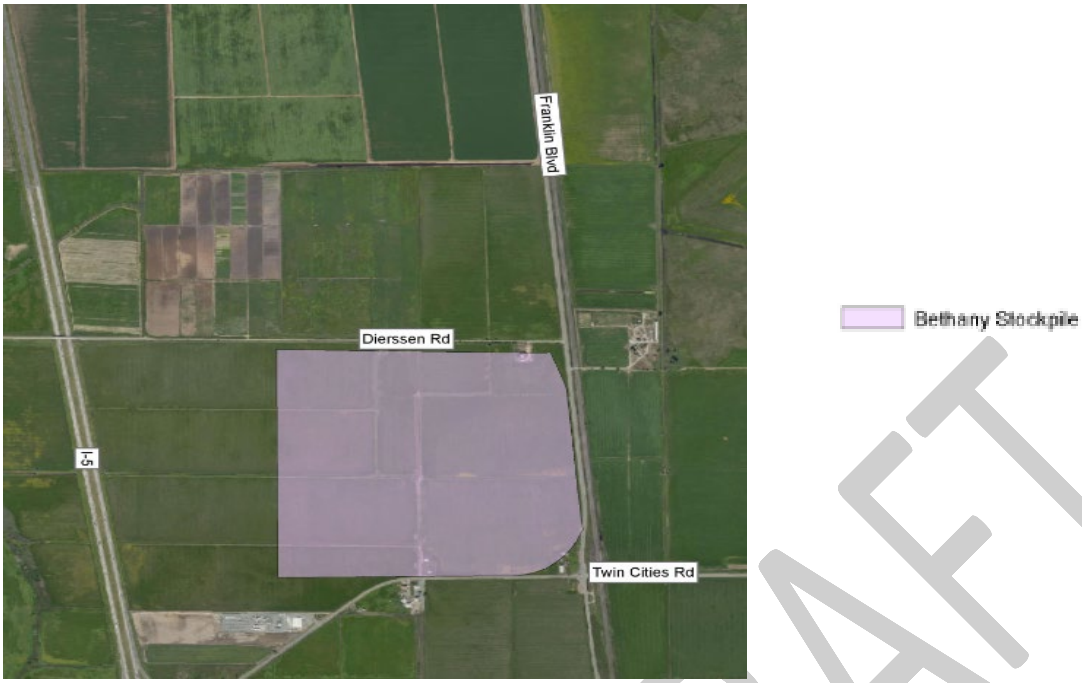


Figure A4-2. Bethany Reservoir Alternative Permanent Stockpile at the Twin Cities Complex Site

4.2 Hydraulic Model Findings

The following results comparing conditions with Scenarios 1 and 2 are based on the 20-day storm duration. The existing condition 100-year model results are presented in the *Flood Risk Management Technical Memorandum (DCA, 2021b).*

4.2.1 Bethany 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 A4-3a and Figure A4-3b, which allows overland flood flows to inundate Dierssen Road just to the east of I-5 consistent with existing conditions 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 A4-3a. 100-Year Flood Depths under Hydraulic Model Scenario 1 at the Twin Cities Complex Site

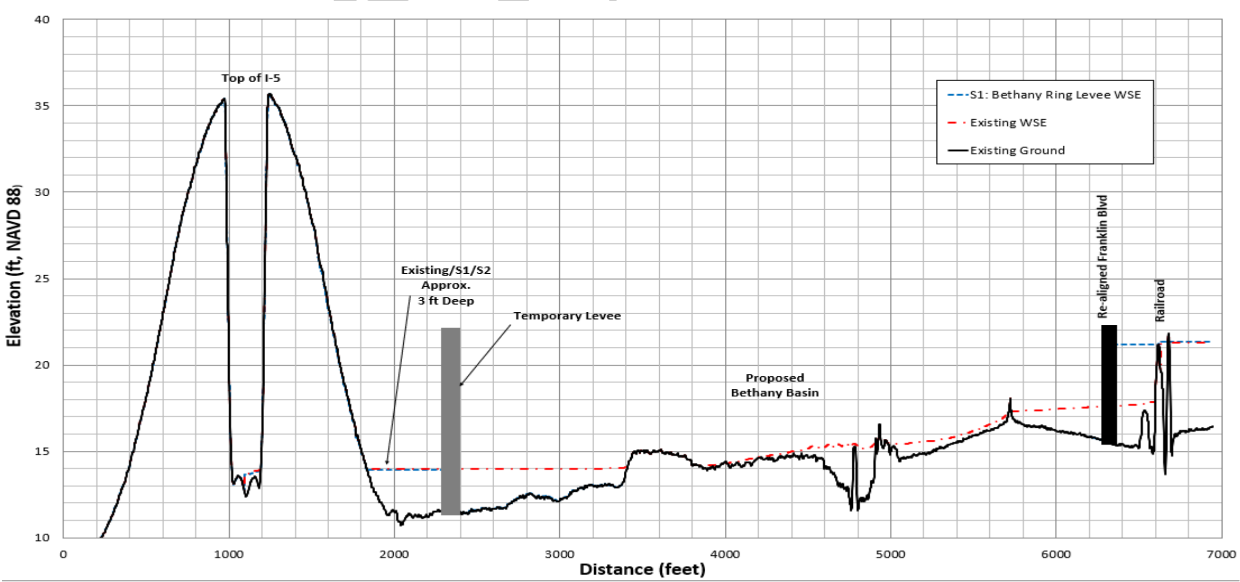


Figure A4-3b. 100-Year Flood Depths Along Dierssen Road under Hydraulic Model Scenario 1

Figure A4-4 shows the ring levee would increase water levels approximately 0.4 ft. in the 100-year storm event compared to existing condition. This effect located 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 will be impacted from approximately 2.5 days.

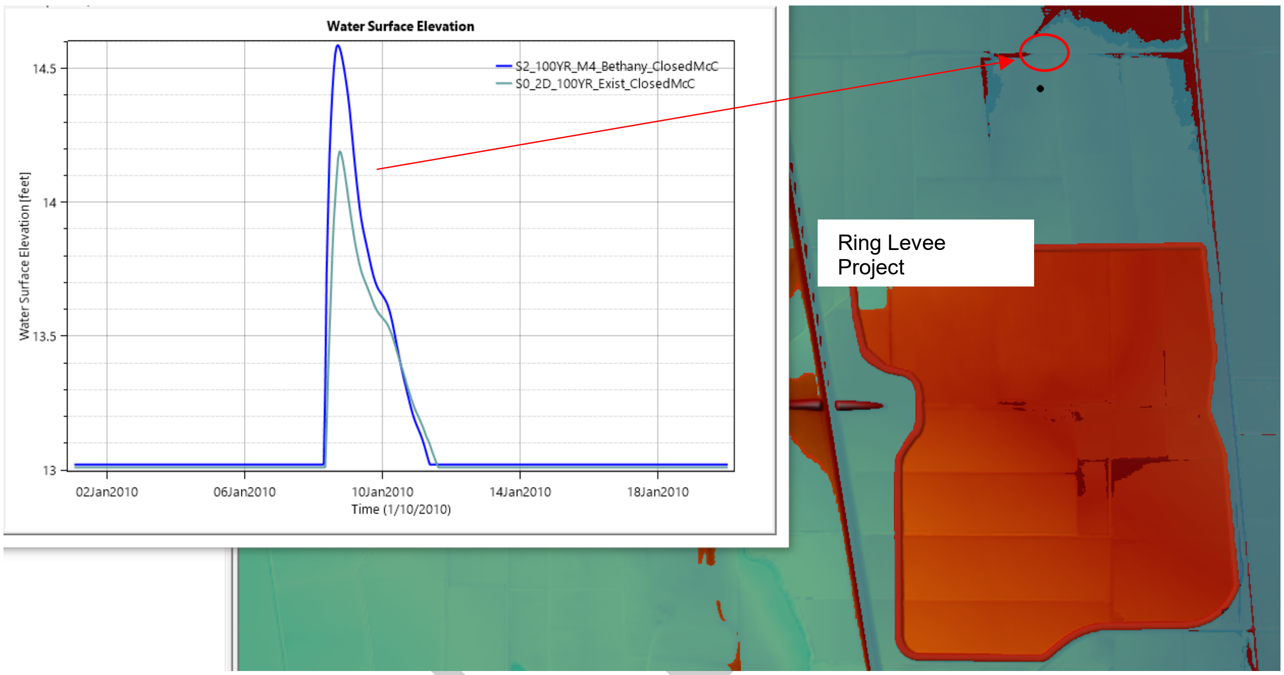


Figure A4-4. 100-Year Flood Depths under Existing Conditions and Hydraulic Model Scenario 1 to the North of Twin Cities Complex Site

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 A4-5 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.

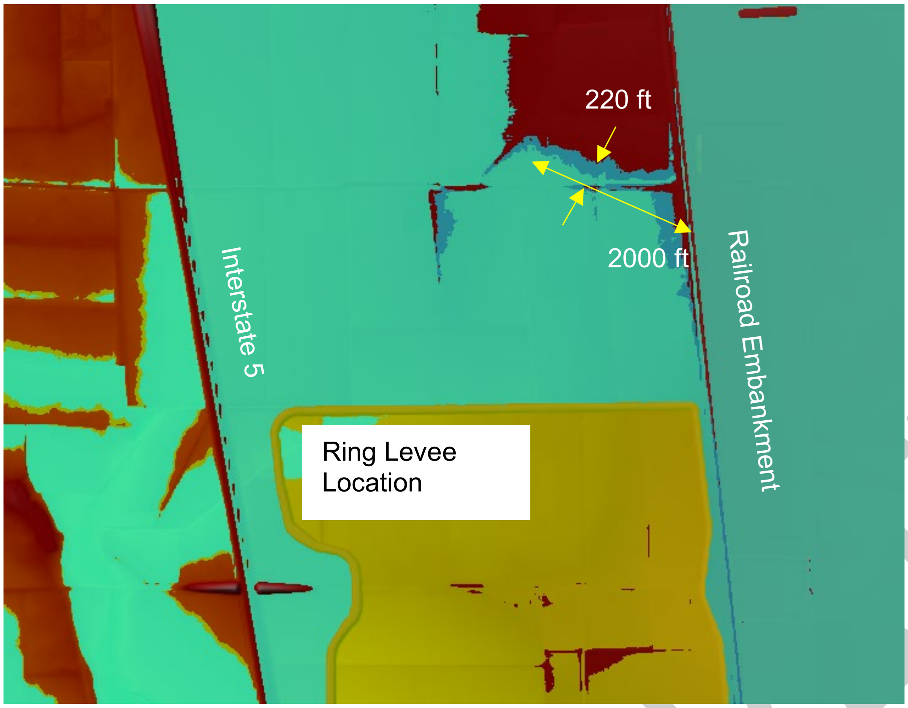


Figure A4-5. 100-year Floodplain under Existing Conditions and Hydraulic Model Scenario 1 north of Twin Cities Complex Site

Figure A4-6 presents the cross section 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.

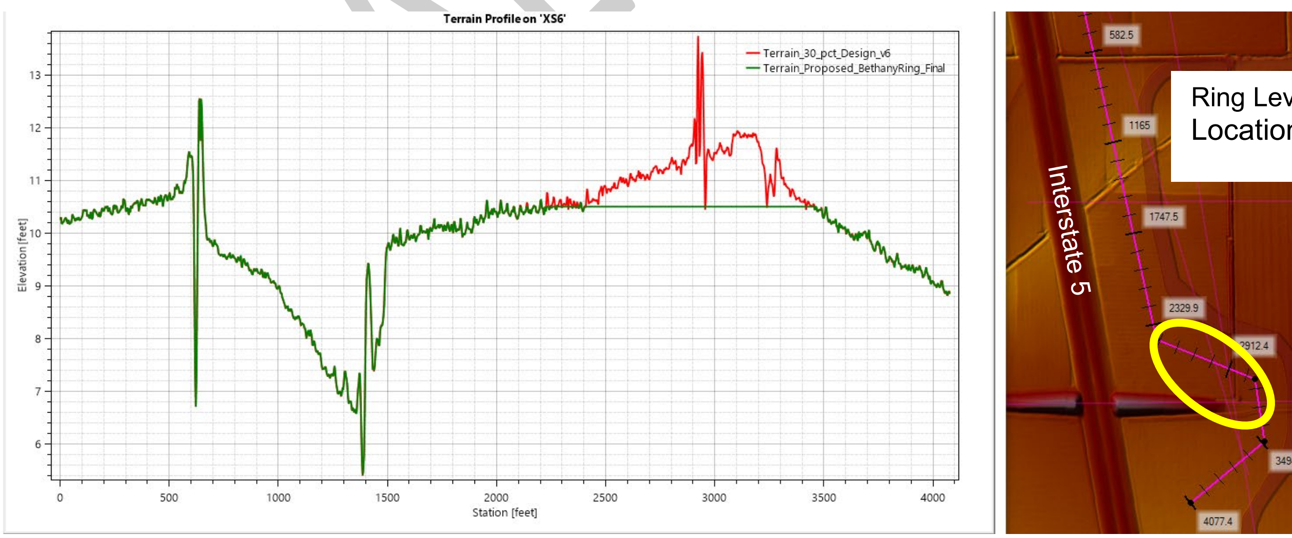


Figure A4-6. Proposed Grading North to South Profile North of Dierssen Road and East of Interstate 5 to Facilitate Flood Flows from North to South towards Dierssen Road

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

Figure A4-7 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 will 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 will 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 will 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.

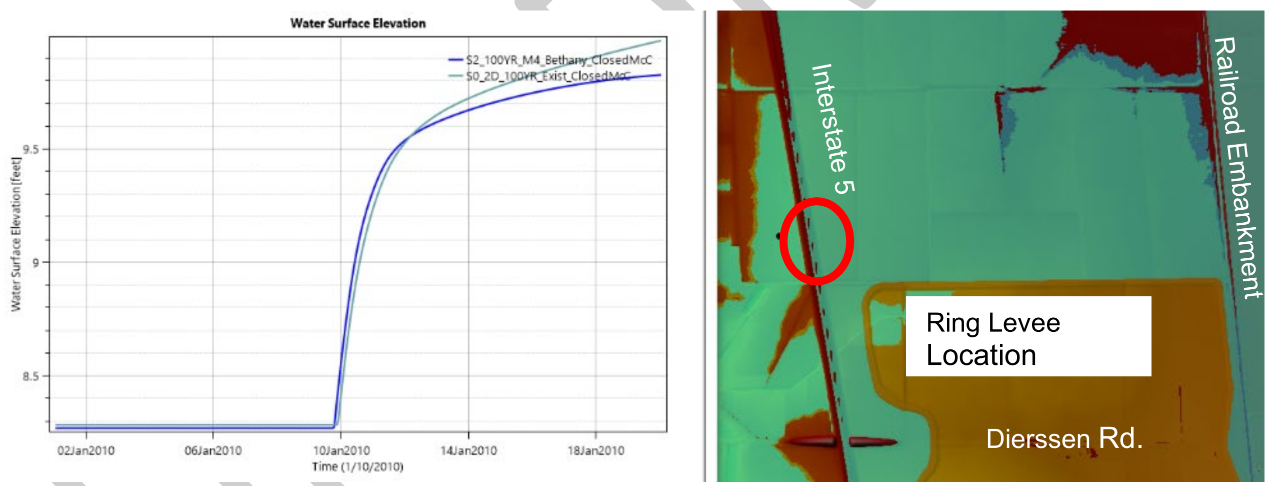


Figure A4-7. 100-Year Flood Condition under Existing Conditions and Hydraulic Model Scenario 1 West of Interstate 5 to the North of Dierssen

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

4.2.2 Bethany Reservoir Alternative Stockpile Scenario 2 Results

Figure A4-8 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.

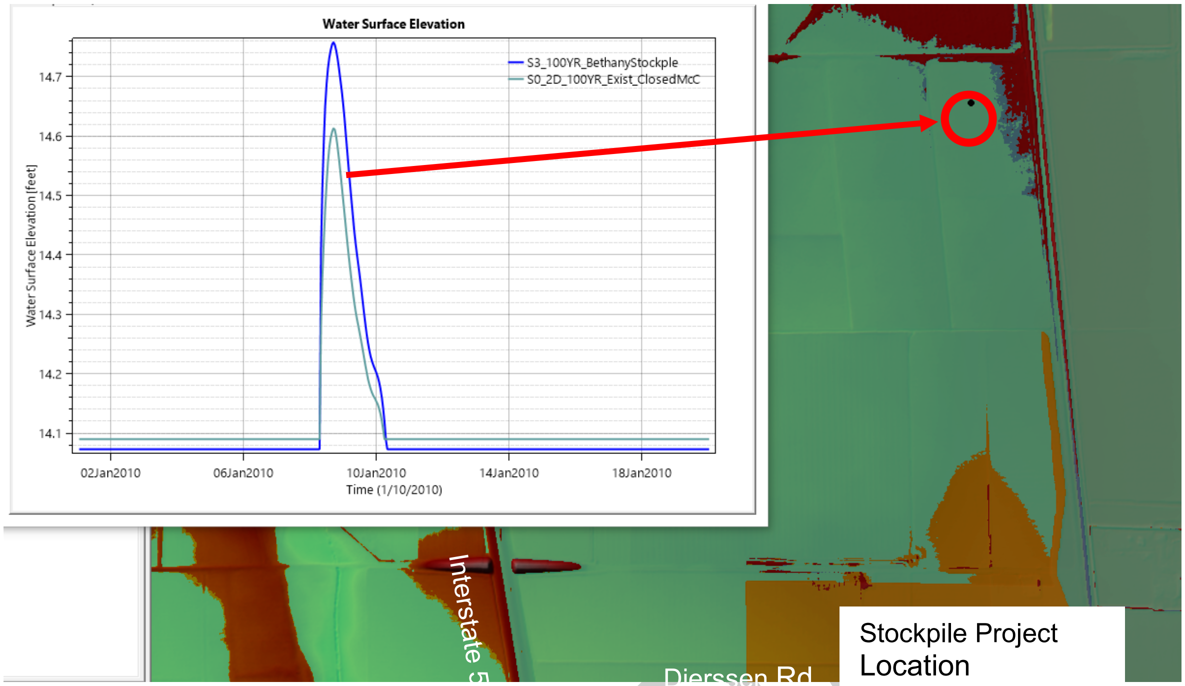


Figure A4-8. 100-Year Flood Depths under Existing Conditions and Hydraulic Model Scenario 2 North of Twin Cities Complex Site

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

Figure A4-9 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 A4-9. 100-year Floodplain under Existing Conditions and Hydraulic Model Scenario 2 north of Twin Cities Complex Site

4.3 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.