Multidisciplinary Project

Report: Risk Analysis Van Sickle Island

Group MDP 350

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Risk Analysis
Van Sickle Island

by

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Abstract

Van Sickle Island, located in the Suisun Marsh in the San Francisco Bay Area, has experienced multiple levee breaches in the past. Due to the large social and economic implications of flooding, questions have arisen about whether the current management of the island is still feasible. This project, therefore, aimed to find the most financially favorable management plan for the upcoming 50 years.

First, an understanding of the system was obtained through a site investigation, a multivariate analysis, and a hydrodynamic model. It was concluded that discharge, tide, air pressure, and wind setup all contribute to the water level. These variables were then used as input for a 1-dimensional hydrodynamical model, which quantifies their effect on the water level.
Subsequently, four management plans were considered: status quo, raising the levees, conversion into an estuarine wetland, and abandoning the island. To allow for comparison between these management plans, a cost-benefit analysis and a net present value calculation were performed for every plan.

One of the major contributors to the costs is risk. Risk is defined as the product of the probability of failure and the consequence of that failure. The failure mechanism assessed is overflow, as this is the most relevant one. To quantify this failure probability, a statistical model was developed. This model includes an extreme value analysis and an event tree.

Due to large uncertainties in the behavior of both the levee and the water level over time, the original 50-year lifetime of the management plan was deemed too ambitious. Therefore the lifetime was reduced to 10 years. The conclusion of the cost-benefit analysis and the net present value was to convert Van Sickle Island into an estuarine wetland. This is because it was the only management plan that was profitable after 10 years. The net present value was found to be equal to $14,924,048.

Preface

As part of the master Civil Engineering at Delft University of Technology, we have the opportunity to participate in a Multidisciplinary Project in The Netherlands or abroad for ten weeks (one quarter). We, a group of four Hydraulic and Hydraulic Offshore Structures students, took this opportunity. Over the last 3 months, we constructed a potential long-term plan for Van Sickle Island, an island in the Sacramento/San Joaquin River delta in California. The island has experienced regular floods (as a matter of fact it was flooded during the duration of our project), which raised questions about the future of the island. Via economic optimization, we researched various potential future situations and constructed advice for the future of Van Sickle Island from an economic perspective. We hope that our work will be of use for Reclamation District No. 1607, responsible for the maintenance of the levees, and potentially other districts experiencing similar situations.

With this project, we were able to use our theoretical knowledge from the masters into practice, learning a lot about the difference between theory and practice. Apart from the amazing learning experience, the chance to live in the United States and discover the beautiful state of California is something we will never forget. We would first like to thank Robert Lanzafame for being our primary supervisor and giving us this opportunity to combine this university project with a wonderful abroad experience. Secondly, we would like to thank Stuart Pearson for being our second supervisor and providing us with a model of the entire Bay Area, as well as helping us revise our hydrodynamic model.

We would also like to specifically thank Chris and Nancy Lanzafame, firstly for their on-site supervision and guidance, but even more for their hospitality and enthusiasm, making us feel at home during our stay in the United States.

We want to give a special thanks to FAST fund Delft, for helping us realize this project and the experience that came with it.

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Group MDP 350
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FAST

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Introduction

Van Sickle island, located at the eastern edge of the Suisun marsh in the Sacramento/San Joaquin River delta (see Figure 1.1), comprises 2415 acres of land and is protected against flooding by a 9.7-mile-long levee. On the eastern side, the island is bounded by the Montezuma Slough, on the southern side, by the Sacramento and San Joaquin Rivers, and on the western side, by the Spoonbill Creek and Honker Bay. The Montezuma Slough Salinity Control Gate is located just northeast of the island. Van Sickle Island was originally dammed in for agricultural purposes, which were eventually discontinued due to the effect of salt intrusion on crops. Nowadays, the island is managed by Reclamation District 1607 and is home to a few gas-pumping installations, recreational residences, and numerous duck clubs. For the latter, the island has been managed as a waterfowl area for over 50 years [1]. Waterfowl hunting requires a certain water level during the season. This is achieved by intentionally flooding the island in mid-October by opening water control structures like pipes. After the season ends in late January, the excess water on the island is drained by pumps [2].

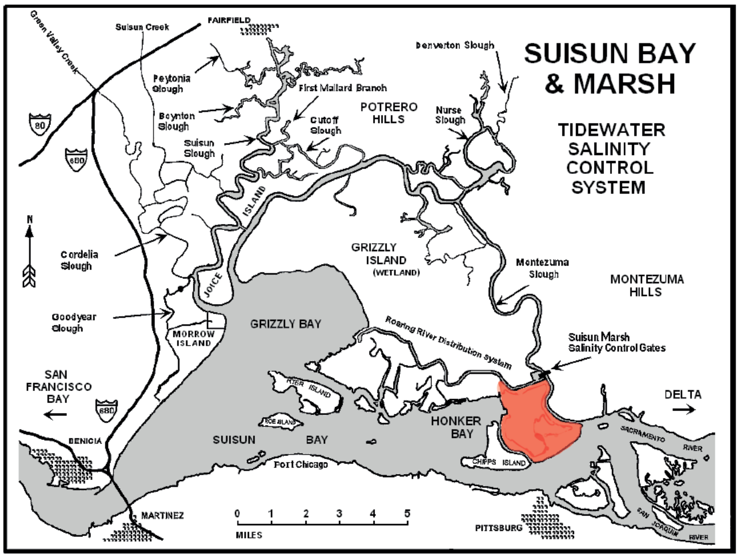


Figure 1.1: The location of Van Sickle Island in the Suisun Marsh.

However, multiple levee breaches have occurred since 1986: in 1986, 1998, 2006, 2017, and 2023. This led to the uncontrolled flooding of Van Sickle Island, which interfered with the intended use of Van Sickle Island and had a large economic and environmental impact. Due to the frequency of flooding, questions have arisen about whether the current way of managing the island is still feasible. This project therefore aims to consider multiple management plans and assess their effects and financial implications for the upcoming 50 years. To allow for comparison between management plans, an economic optimization will be performed. This is a framework that considers the relation between risk reduction (failure probability and consequences) and investment costs. The following four management plans are considered: status quo, raising the levees, repurposing into a mitigation and/or conservation bank, and abandoning the island. However, there are numerous other potential management plans and hybrid solutions possible, that are not considered in this report.

The report has the following setup, starting with an introduction to the Suisun Marsh and the levees of Van Sickle Island in Chapter 2. In Chapter 3, a multivariate analysis is performed to gain a better understanding of the physical processes behind Van Sickle Island's floods. The relevant processes form the basis of a hydrodynamic model to compute water levels at the island in Chapter 4. Chapter 5 will link chapters 3 and 4 back to the economic optimization with a statistical model. Chapters 6, 7, 8, and 9 then explain and analyze each management plan, using the statistical model of Chapter 5. The report concludes in chapter 10, by stating and discussing a conclusion and giving a recommendation for a future management plan.

2

Site Information

Van Sickle Island is located in the Suisun Marsh, a tidal wetland in the Sacramento-San Joaquin Delta in California. Tidal wetlands form in relatively low-lying sheltered areas like the San Francisco Bay Area where the freshwater of rivers mixes with the salt water from the ocean. The freshwater supply from the Sacramento and San Joaquin Rivers combined with the tidal influences of the Pacific Ocean make the Suisun Marsh a perfect spot for this interaction. In addition to freshwater, large amounts of weathered rock from the Sierra Nevada are transported by the rivers towards the Pacific Ocean. These sediments are deposited depending on the gradient of the river, the size, and the weight of the rock particles. When the gradient approaches zero, such as in the Suisun Marsh, fine particles like clays, silts, and organic material are deposited. The material accumulates and under the influence of the tide, a dynamic system of channels, pools, and islands is formed. Vegetation then starts to grow in the area, which stabilizes the sediments and supports the formation of islands. The vegetation has a limited lifetime and the decay of plants and animals causes an accumulation of organic material under the water line. Since full decomposition is impossible under water due to the lack of oxygen, organic waste will accumulate and transform under pressure into peat over time. Nowadays, the Suisun Marsh is funded on a layer of peat as thick as 60 to 80 feet [1].

Naturally, wetlands are ever-changing, but due to the construction of levees in the late nineteenth century, islands like Van Sickle Island were formed and maintained. This took away the natural dynamic nature of the area. However, the Suisun Marsh remains the largest tidal wetland in western North America and continues to support a wide range of vegetation, migratory birds, and wildlife.

2.1. Hydrodynamic Processes

2.1.1. Discharge Influence

As mentioned above, the Suisun Marsh is fed by freshwater from the Sacramento and San Joaquin Rivers. The Sacramento River originates in the Klammath mountains in northern California and southern Oregon. The San Joaquin River finds its source in Thousand Island Lake near Mammoth Lakes, California. Despite the precipitation and snow melt feeding both rivers, the discharge pattern at Van Sickle Island is unlikely to follow the precipitation, melting, and evaporation pattern of the fluvial area. This is due to the high number of reservoir dams on the western side of the Sierra Nevada. The water release of those dams varies and peaks when a high precipitation event, which threatens to exceed the reservoir limit is predicted. These releases have a significant impact on the downstream discharge of both rivers. Furthermore, water from the Sacramento and San Joaquin Rivers is redirected for irrigation and the California State Water Project, a system of channels, pipes, and pumps that supplies Southern California with water.

The amount of discharge reaching the Suisun Marsh influences the water level in the area. The closest and therefore most representative measuring station for discharge to Van Sickle Island is located at Chipps Island, directly west of Van Sickle Island. Once the water reaches the Suisun Marsh, the flow is always subcritical, due to the downstream location in the basin.

2.1.2. Tidal Influence

As mentioned at the beginning of this chapter, tidal components influence the water level in the Suisun Marsh. The Bay Area is known to have a mixed diurnal and semi-diurnal tide. Due to the interaction of the diurnal and semi-diurnal components, two high tides and two low tides occur per lunar day, with each a lower and higher variant. These four tides are known as the High-High, High-Low, Low-High, and Low-Low. As the diurnal and semi-diurnal tides have different timescales, the tidal water level changes during the lunar cycle: the highest tide is called king tide, and the lowest is neap tide. During king tide, the earth, sun, and moon are all aligned causing a maximal gravitational force of the sun and moon on the water body, thus creating a king tide. During neap tide, which occurs seven days after king tide, the sun and moon make a 90-degree angle with the earth, decreasing the gravitational energy of the sun and moon, thus decreasing the tidal water level. The tidal water level in the Suisun Marsh is measured at Mallard Island.

Apart from the water level, the tide also influences the salinity in the Suisun Marsh by bringing brackish water into the area during incoming tides. This affects the livability of the marsh for vegetation and animals. For example, delta smelt, an endangered fish species, thrives well in fresher water, but escapes the area when the water becomes too salty. To accommodate these and many other species, the Department of Water Resources manages the salinity in the Suisun Marsh through a salinity control gate in the Montezuma Slough just north of Van Sickle Island. The gate closes if the salt content in the Montezuma Slough becomes too high, thereby preventing salt water from flowing in.

2.1.3. Barometric Pressure and Wind

Furthermore, the barometric pressure, wind speed, and wind direction may influence the water level in the Suisun Marsh. The effect of the air pressure on the water level can be explained by a force balance. When air pressure exerts a downward force on the water, it compresses the water, thus lowering the water level. In large water bodies it is assumed, as a rule of thumb, that a decrease of 1 millibar corresponds to a water level increase of 1 centimeter (0.4 inches) [3]. The air pressure in the Suisun Marsh can be estimated using data from the measurement stations at Mallard Island and in San Francisco.

Another effect of air pressure is the creation of airflow (wind) due to spatial variation in pressure. In summer, a high-pressure zone is located on the Pacific Ocean, and a low-pressure zone is formed above the Central Valley due to the rise of warm air. This causes a sea-to-land breeze (from the west). In summer and fall, it is also possible that a high-pressure area forms on the Eastern Side of the Sierra Nevada and causes a short-term breeze from the east, called Diablo-winds. These winds are usually warm and extremely dry. Both the sea-to-land breeze and Diablo-winds superpose on top of the global atmospheric circulation. This circulation assumes a relatively constant high-pressure zone around 30 degrees latitude and a low-pressure zone around 60 degrees latitude, causing a south-to-north airflow. In the Northern Hemisphere, this flow is deflected towards the east due to the Coriolis effect, creating westerlies. During summer westerlies in combination with the sea-to-land breeze cause a predominant airflow to the west. Therefore, it can be concluded that the predominant wind direction in the Suisun Marsh is west during summer.

In winter, a completely different wind direction is observed: a predominant southern wind. Storms and thus low-pressure zones are formed off the coast of Northern California, creating a counter-clockwise spiral airflow pattern nearby, which translates into southern winds in the Suisun Marsh. All described airflow patterns are schematized in Figure 2.1

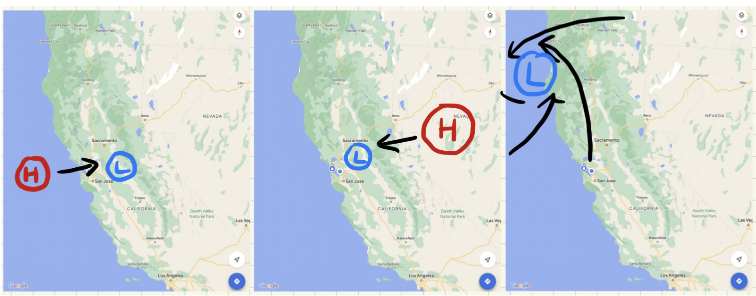


Figure 2.1: Schematisation of airflow patterns. From left to right: sea-to-land breeze, Diablo-winds, spiral airflow in winter.

The winds created by the airflow patterns can also influence the local water level by creating waves and wind setups. Wind setup is a local rise or drop in water level due to the wind pushing the water in a certain direction. Since the Suisun Marsh has relatively small fetches, wind setup in the marsh can be assumed insignificant. However, in San Pablo Bay, a bit further downstream, fetches do become significant, creating a wind setup, which could influence water levels in the Suisun Marsh. To calculate wind setup, Equation 2.1 is used.

$$\delta h = \frac{\kappa u_{10}^2}{g * h} F \cos(\phi) \tag{2.1}$$

In which κ is a constant, u_{10} is the wind speed at Mallard Island, g the gravitational constant, h the average depth of the bay along the fetch, $\cos\phi$ is the angle with the fetch perpendicular to the coastline, and F the fetch. The measuring station at Mallard Island measures both wind speed and its associated wind direction.

To simplify the calculation of the wind setup, the bathtub principle was defined (Figure 2.2). This principle assumes the bay as a square basin with closed boundaries. Within this basin, setup at the one end is equal to set down at the other end, following a linear trend along the basin length. To satisfy the square basin assumption in the bathtub principle for San Pablo Bay, the presence of San Fransisco Bay and Carquinez Strait is not considered in the basin length.

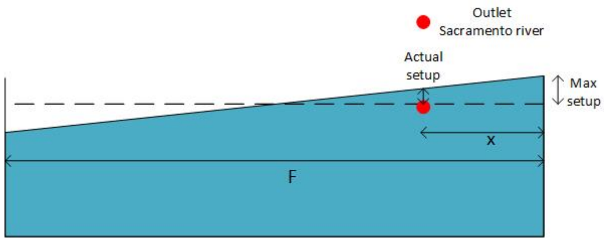


Figure 2.2: Bathtub principle explained.

With the bathtub principle defined, the variables in Equation 2.1 are quantified. ϕ represents the angle the wind direction makes with the outlet of Carquinez Strait. $cos(\phi)$ reaches a maximum when the wind is perpendicular to the outlet.

The depth is calculated using the mean tidal water level per wind direction and the bathymetry of the bay [4]. This is done to simplify the analysis and keep the wind setup independent of the water level.

The last variable is the fetch. For every wind direction, the maximum possible fetch over San Pablo Bay is measured using Google Earth, to maximize the potential wind setup. Apart from the fetch, the distance from the coast to the outlet of Carquinez Strait along the fetch line is also measured (noted as distance x). This distance is used to convert the maximum wind setup to the wind setup at the outlet, using Equation 2.2.

$$\delta h_x = \frac{\delta h_{max}}{0.5F} * x \tag{2.2}$$

All of these measurements per wind direction are shown in Appendix B

2.1.4. Overview

In summary, the discharge, tides, wind speed, wind direction, and barometric pressure can all contribute to the water level at Van Sickle Island. An overview of where, in which units, and at what interval, these variables are measured is shown in Table 2.1. The measurement stations are also pinpointed on the map in Figure 2.3. A more detailed overview of the data sources is given in Appendix A

Table 2.1: Overview of measured variables per station.

Location	Variable	Units	Frequency
Mallard Island	Water level	Feet	Hourly
	Barometric Pressure	Millibar	Hourly
	Temperature	$^\circ F$	Hourly
	Wind speed	Knots	Hourly
	Wind direction	Degrees	Hourly
	Tidal water level	Feet	Hourly
Martinez	Tidal water level	Feet	Hourly
San Francisco	Barometric Pressure	Millibar	Hourly
Chipps Island	Discharge	$m^3 s^{-1}$	Daily

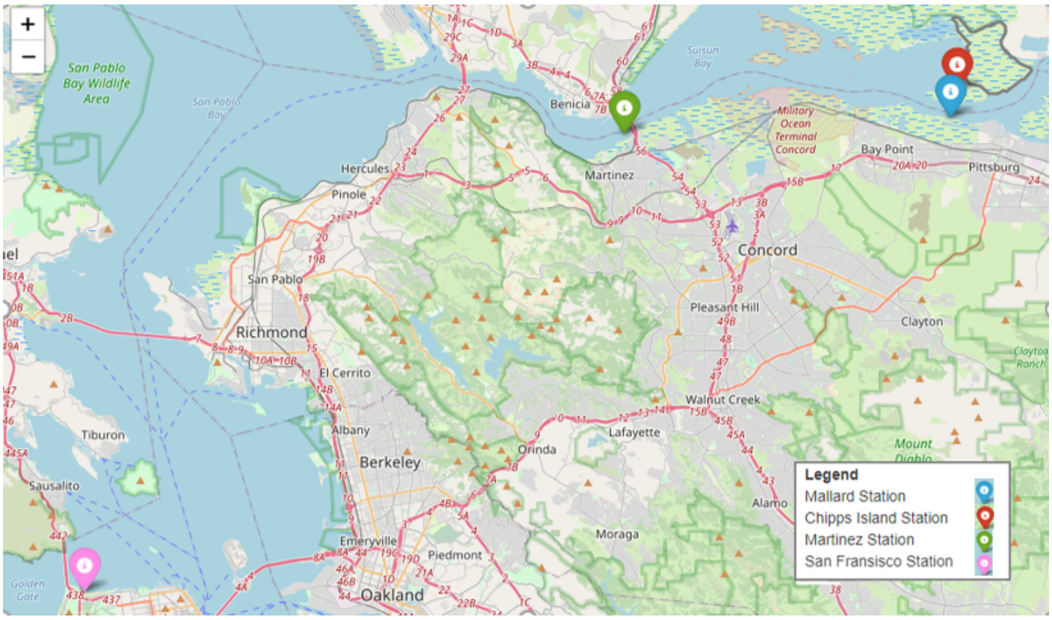


Figure 2.3: Map of the locations of measurement stations.

2.2. Levees

Since Van Sickle Island is located in a low-lying area, it is necessary to protect it against high water levels by a levee. A so-called KMZ-marker describes the location of every point along the levees of Van Sickle Island. The markers originate from Google Earth and every marker represents a single foot of levee length.

Based on the location and exposure to the physical processes described above, the levee is divided into five sections: Montezuma Slough, Sacramento River, Spoonbill Creek, Honker Bay, and Van Sickle North. The locations of the sections are shown in 2.4 and expressed in KMZ-markers in Table 2.2.

Table 2.2: Sections and their KMZ points

Name	KMZ Marker
Montezuma Slough	453+00 - 60+00
Sacramento River	60+00 - 155+00
Spoonbill Creek	155+00 - 248+00
Honker Bay	248+00 - 317+00
Van Sickle North	317+00 - 453+00

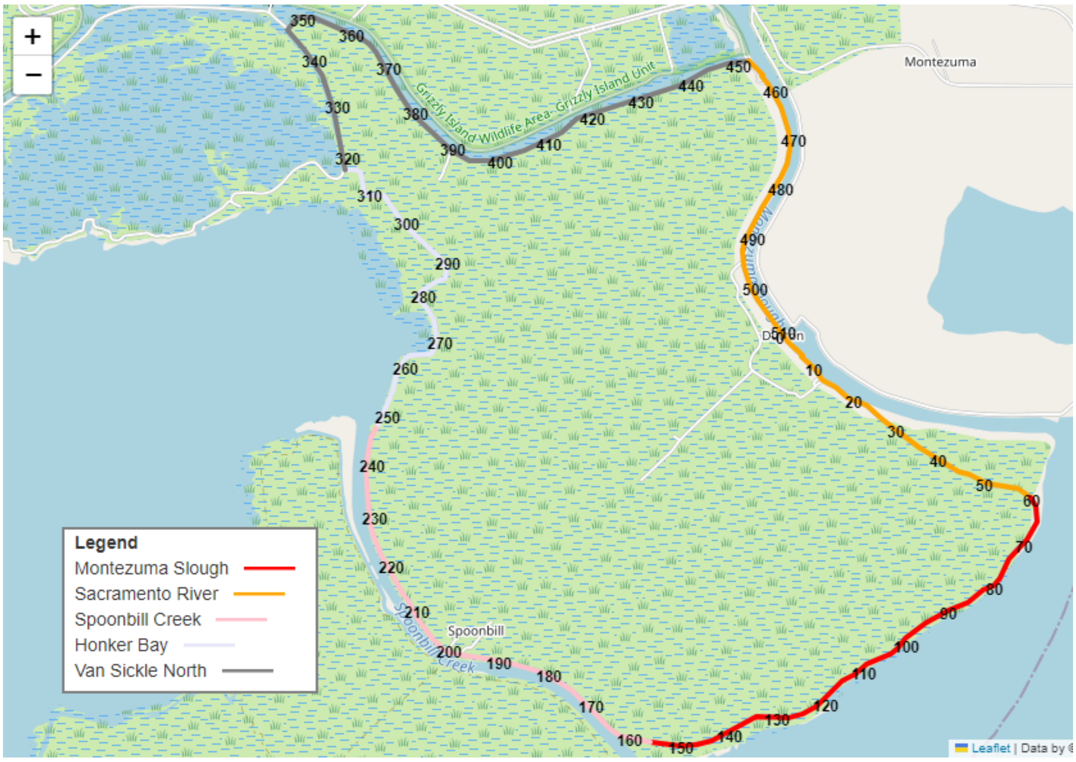


Figure 2.4: Map of levee sections on Van Sickle Island.

As seen on Figure 2.4, the Sacramento River and Honker Bay sections are more exposed than the other sections. Phenomenons, like wind setup and waves, are therefore more significant in these sections. In summer, the Honker Bay section is impacted most due to waves and wind setup, since the wind direction is predominantly west. In winter, the Sacramento River section is impacted most due to the predominantly southern wind direction. As mentioned the location of the other three sections is more sheltered. On the southern end of the Montezuma Slough section, Spinner Island is attached to Van Sickle Island (KMZ 20+00 to 60+00). The two are separated by an interior levee. The same holds for the levee between Van Sickle and Wheeler Island in the Van Sickle North section (KMZ 320+00 to 345+00). The levee section along Roaring River Slough in the same levee section (KMZ 345+00 to 350+00) is maintained by the Department of Water Resources and relatively sheltered by Grizzly Island, located just north of the section [1]. Furthermore, the Spoonbill Creek section has a sheltered location due to Chipps Island being located directly west of the section.

2.2.1. Cross-Sections and Height Profile

Apart from the location, the impact of the water level and waves on the levee is also influenced by the cross-sections and height profile of the levee. A geotechnical investigation along Van Sickle Island's perimeter was performed in 2003. It was concluded that all levees in the original state were constructed as a silt core with a gravel cover placed upon the peat soil layer [5]. During this MDP project, it was also observed that riprap was added on the outer slope of the levees as protection against waves and slamming debris. This was mostly done in the more exposed sections on the Sacramento River and the Honker Bay.

The height of the levees was most recently surveyed in 2018 and 2019. The results were published in the 5-year plan for Van Sickle Island by MBK Engineers in April 2022 [1]. The crest height along the levee is tabulated in Table 2.3 shown in Figure 2.5. These are in NGVD29 (or conversion to NAVD88 see Appendix A).

Table 2.3: Levee crest heights along different levee sections.

Sections	Length of Levee with Crest Height (NGVD29) (feet)			
	<5 ft	5-6 ft	6-7 ft	>7 ft
Sacramento River	0	4921	2789	295
Spoonbill Creek	0	495	5577	3248
Honker Bay	0	492	3444	2952
Van Sickle North	4200	4200	4200	951
Montezuma Slough	4200	1804	2395	3576

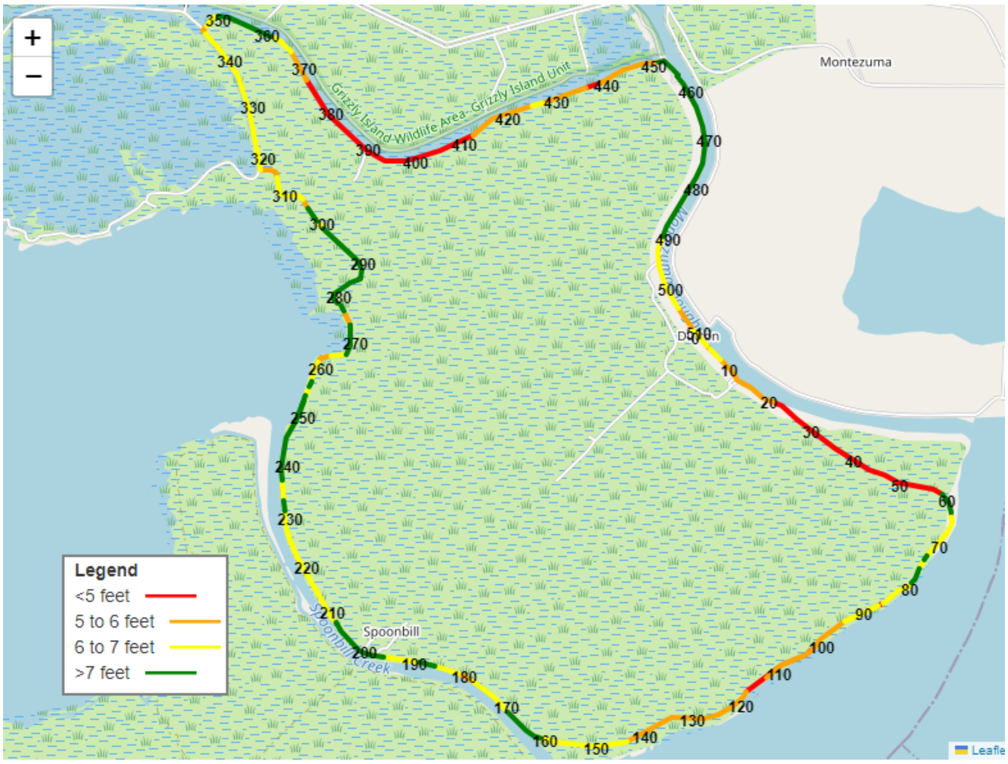


Figure 2.5: Height profile levees on Van Sickle Island.

As seen in both Table 2.3 and Figure 2.5, the height profile of the levee shows significant variations. There are three extremely low spots (<5 feet in NAVD29): the interior levee between Spinner Island and Van Sickle Island in the Montezuma Slough section, the levee in the Sacramento River section, and the section along the Roaring River Slough in the Van Sickle North section. The Spinner Island interior levee is intentionally kept low since there is a higher primary levee. However, an (un)intentional rise in water level on Spinner Island can easily overflow the interior levee and cause undesirable inflow into Van Sickle Island. This was observed during fieldwork during this project when sandbags were placed to prevent further inflow. The low section along the Sacramento River has overflowed and breached multiple times in the past.

2.2.2. Current State of Van Sickle Island

As mentioned in subsection 2.2.1, the data gathered on the cross-sections and height profile was done in 2003, 2018, and 2019. Therefore these surveys might be outdated. Unfortunately, it is unclear how and to what extent maintenance, repairs, and failure mechanisms have influenced the levees over the more recent years. To identify the failure mechanisms that are currently relevant to Van Sickle Island, a qualitative analysis was done. Overflowing was shown to be the most relevant failure mechanism and is, therefore, the main focus of the risk analysis in the report (Appendix C).

To prevent these failure mechanisms, maintenance is essential. In general, Van Sickle Island is maintained on a continuous basis as best as the budget of the Reclamation District 1607 allows. During maintenance, the Reclamation District takes care of the following:

- Raising low spots
- Filling of holes and erosion spots
- Placement of riprap on exposed spots
- Management of the water level
- Maintenance of the roads and installation

In addition to the regular tasks, emergency measures, like placing sandbags, aquadams, and geotextiles, are taken if it is expected that the levee cannot withstand the predicted water level. Even with emergency measures, overflowing and overtopping events can still occur. These can cause the levee to erode up to the point where the levee becomes too weak to withstand the water and thus breaches.
After a levee breaches, reparations are done. However, due to the large costs associated with levee repairs, an emergency levee may be constructed first instead of a full operational levee. The Reclamation District 1607 aims to complete the creation of the full operational levee over time, but this impacts the financial ability to maintain other sections of the levee. In practice, reconstruction of an operational levee therefore becomes a multiyear project. After the repair, the island is drained by pipes and pumps.

Van Sickle Island has experienced five levee breaches since 2006: in 2006, three times in 2017, and in 2023.

- On the first of January 2006, the levee in the Sacramento River section breached.
- The same happened on the eleventh of January 2017. The section was repaired at the end of January of the same year, but due to the extreme water release of the Oroville Dam, it breached again on the fifteenth of February 2017.
- The last breach in 2017 happened on the twenty-fifth of May near the Concord Club in the Montezuma Slough section.
- The most recent breach occurred on the sixth of January 2023 in the Sacramento River section. A second breach in the same section was formed at the beginning of March 2023 due to inner slope erosion.

A more extensive analysis of important dates is given in Appendix E.

As of November 2023, Van Sickle Island is still flooded due to the January 2023 flood. The breach was plugged in September 2023 and the draining of the Island started in October of the same year. Apart from the breach, Van Sickle Island shows various other damaged spots caused by different failure mechanisms. An overview of the weak spots covered in the report is shown in Figure 2.6.

Figure 2.6: Map of weak spots on Van Sickle Island.

As shown, all damaged spots of interest, except for the erosion spot in the Honker Bay section, are located in the Sacramento River section. Therefore, the Sacramento River section is the area of interest in the main report, and the spot in Honker Bay will be covered in Appendix D as a case study. However, this focus does not imply that there are no damaged spots on other sections of the levee.

Besides damages due to failure mechanisms, vegetation and animals also impact the levees on the island. Vegetation can have a strengthening effect on the levee when the plants have small roots that hold the soil together and thereby prevent erosion (up to a certain point). Plants with larger roots, like trees, can harm levees when they become less healthy and therefore weaker. On Van Sickle Island all levee slopes are covered with tall plants and the levee crest on the Sacramento River section is also overgrown. Furthermore, there are multiple locations with trees on the levee, like the southern end of the Spoonbill Creek section.

Between all those plants, there are a lot of animal species taking shelter. A few of them are worth pointing out due to their effect on the levees. Burrow-digging animals like otters and coyotes use the levee as a place to create their dens. This results in multiple holes in the sides and on top of the levee. Therefore, they negatively impact the stability and permeability of the levee. Field observations have shown that this is also a problem in the levees of Van Sickle Island. Burrows in the levee are hard to prevent, but the influence on the levee can be reduced by filling the holes up.

Another animal species impacting the levees is beavers. Beavers built their dams in flowing water, such as just behind a leaky levee section. Beaver dams are therefore a good indicator of the weaker spots in the levee. However, beavers also use materials like branches, dirt, and rocks, possibly originating from the levee, to build their dams. This again threatens the stability of the levee.

3

Multivariate Analysis

3.1. Introduction

A visualization is shown of known variables during the floods of 2017 (See Figure 3.1). From these graphs, it becomes clear that certain variables correlate to one another. For example when looking at the 1st flood of 2017 a drop in air pressure occurs simultaneously with a sudden rise in wind setup indicating that there might be a correlation between the two.

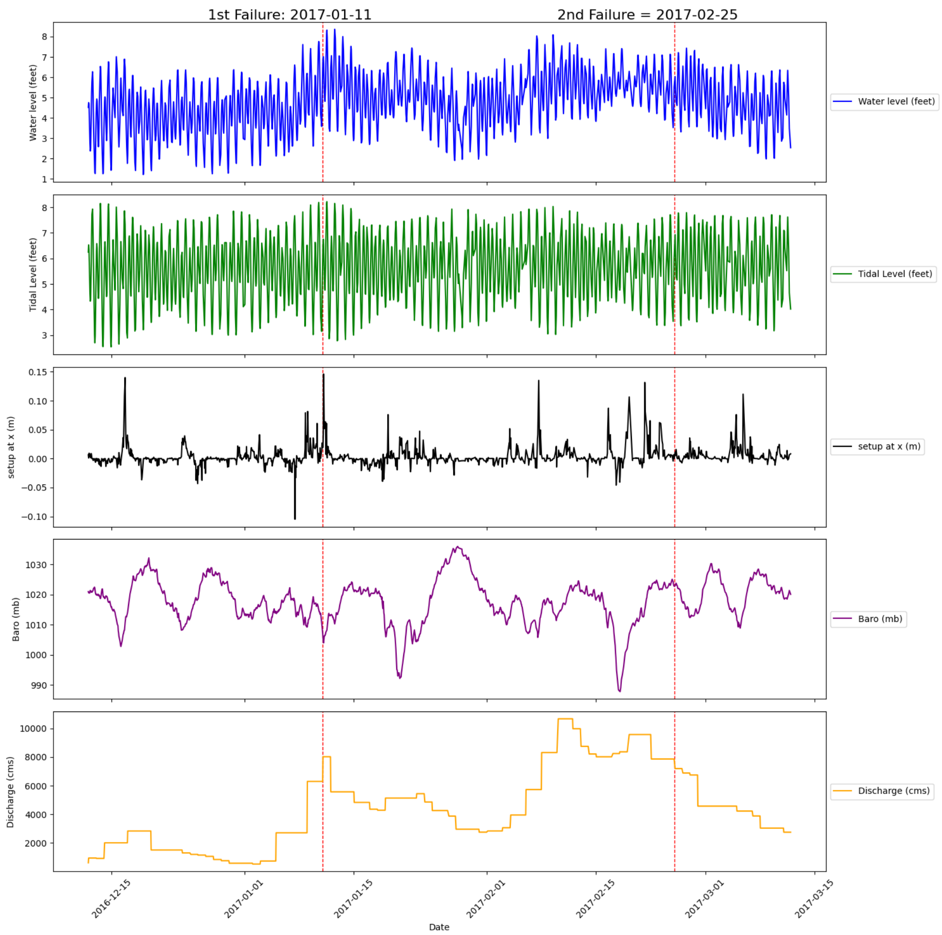


Figure 3.1: Visualisation known variables during flood of 2017

This same visualization is done for the floods of 2023 (See Figure 3.1). By comparing the floods of 2017 to the ones in 2023 trends might become clear.

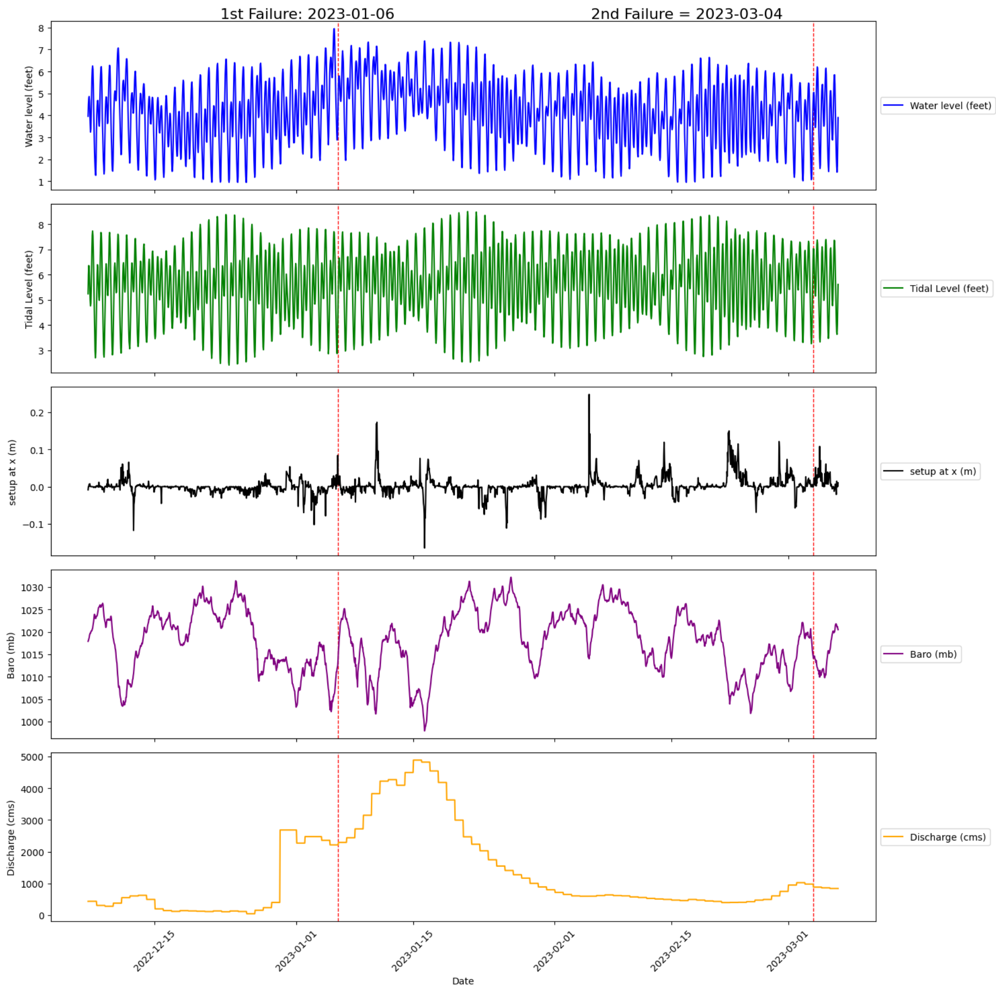


Figure 3.2: Visualisation known variables during flood of 2023

To achieve a general understanding of such correlations and potentially identify trends a visual preliminary analysis of the following variables, measured at different locations was done (Table 3.1). It is important to note that hourly data is not investigated since discharge data is only available daily. As discharge has a known relationship with the water level and can therefore not be neglected. As discharge has a known relationship with the water level and can therefore not be neglected, the choice is made to convert the frequency of the other variables to daily.

Table 3.1: Variables used in multivariate analysis and their measurement locations.

Location	Variable	Units	Frequency	Time Record	Daily Sampling
Mallard Island	Water level	Feet	Hourly	01-Jan-2001:29-Aug-2023	Max, mean, min
	Pressure	Millibar	Hourly	08-Jun-2011:30-Sep-2023	Max, mean, min
	Temperature	$^\circ F$	Hourly	08-Jun-2011:30-Sep-2023	Mean
	Wind speed	Knots	Hourly	08-Jun-2011:30-Sep-2023	Mean
	Wind direction	Degrees	Hourly	08-Jun-2011:30-Sep-2023	Mean
	Tidal water level	Feet	Hourly	01-Jan-2000:31-Dec-2025	Max, mean. min
San Francisco	Pressure	Millibar	Hourly	01-Jan-2011:09-Oct-2023	Mean
Chipps Island	Discharge	$m^3 s^{-1}$	Daily	31-Dec-1993:21-Sep-2023	Mean

This preliminary analysis was done by creating tables, in which parameters magnitudes are shown. Besides the actual value of the parameters, extra columns are added to show whether a value was considered statistically high, low, or a mean. This is done by using the p-rank, a value that shows the percentile of a value. For instance, when a value has a p-rank or percentile of 95 this means that 95% of all measured values is either equal or lower than this value. In other words, this value is in the highest 5% of all values ever measured for this parameter.
An example of this for the same flood in 2017 is shown in Figure 3.3.

	Discharge (cms)	P-Rank Discharge	Water level (feet)	P-Rank Water Level	HH-Tidal	P-Rank HH-Tidal	LH-Tidal	P-Rank LH-Tidal
Date								
2016-12-15 00:00:00	943.600000	85.801000	7.130000	98.158000	8.020000	88.200000	2.650000	0.218000
2016-12-16 00:00:00	1247.800000	89.278000	6.960000	96.826000	7.860000	77.054000	6.810000	89.266000
2016-12-17 00:00:00	2006.200000	94.330000	6.090000	63.036000	7.610000	50.630000	6.880000	90.974000
2016-12-18 00:00:00	2584.300000	95.941000	5.520000	24.861000	7.280000	19.276000	6.960000	93.034000
2016-12-19 00:00:00	3016.800000	96.862000	5.360000	17.022000	7.100000	9.171000	3.300000	3.501000
2016-12-20 00:00:00	2832.600000	96.462000	5.470000	22.110000	7.240000	16.477000	3.840000	8.905000
2016-12-21 00:00:00	2222.800000	95.190000	5.840000	45.505000	7.360000	25.442000	4.450000	17.252000
2016-12-22 00:00:00	1859.900000	93.470000	6.050000	60.116000	7.450000	33.620000	5.100000	29.961000
2016-12-23 00:00:00	1650.200000	92.416000	6.360000	79.780000	7.480000	36.394000	5.740000	45.687000
2016-12-24 00:00:00	1505.500000	91.519000	6.360000	79.780000	7.550000	43.930000	6.120000	57.197000
2016-12-25 00:00:00	1300.200000	89.702000	6.060000	60.855000	7.530000	41.665000	5.570000	41.786000
2016-12-26 00:00:00	1197.200000	88.733000	5.850000	46.341000	7.580000	47.468000	4.920000	26.000000
2016-12-27 00:00:00	1149.800000	88.151000	5.950000	53.005000	7.610000	50.630000	4.320000	15.120000
2016-12-28 00:00:00	1059.400000	87.243000	5.980000	55.113000	7.710000	62.491000	3.810000	8.578000
2016-12-29 00:00:00	841.600000	84.129000	6.020000	58.032000	7.770000	68.742000	3.420000	4.483000
2016-12-30 00:00:00	755.900000	82.396000	6.370000	80.349000	7.850000	76.169000	3.170000	2.447000
2016-12-31 00:00:00	659.800000	80.470000	6.250000	73.916000	7.820000	73.201000	6.670000	84.662000
2017-01-01 00:00:00	632.500000	79.767000	6.080000	62.297000	7.730000	64.708000	6.690000	85.474000
2017-01-02 00:00:00	580.000000	78.120000	5.820000	43.870000	7.520000	40.732000	6.810000	89.266000
2017-01-03 00:00:00	522.800000	76.242000	5.770000	40.562000	7.210000	14.732000	6.910000	91.834000
2017-01-04 00:00:00	510.800000	75.830000	6.120000	64.817000	7.090000	8.772000	6.840000	90.041000
2017-01-05 00:00:00	734.000000	81.984000	5.930000	51.745000	7.290000	20.039000	3.930000	9.874000
2017-01-06 00:00:00	1097.600000	87.570000	5.930000	51.745000	7.480000	36.394000	4.630000	20.342000
2017-01-07 00:00:00	1602.100000	92.101000	6.600000	89.593000	7.670000	57.972000	5.460000	38.951000
2017-01-08 00:00:00	1914.000000	93.736000	7.610000	99.370000	7.910000	81.221000	6.180000	60.092000
2017-01-09 00:00:00	2705.400000	96.172000	7.410000	99.128000	8.090000	91.895000	5.720000	45.142000
2017-01-10 00:00:00	4262.400000	98.510000	7.760000	99.455000	8.190000	95.493000	4.920000	26.000000
2017-01-11 00:00:00	6296.100000	99.588000	8.320000	99.576000	8.220000	96.256000	4.090000	12.079000
2017-01-12 00:00:00	8019.000000	99.794000	8.420000	99.588000	8.170000	94.863000	3.420000	4.483000
2017-01-13 00:00:00	7559.900000	99.733000	8.000000	99.528000	8.090000	91.895000	3.040000	1.599000
2017-01-14 00:00:00	6411.800000	99.637000	7.520000	99.297000	7.950000	83.887000	7.070000	95.154000
2017-01-15 00:00:00	5569.000000	99.455000	6.890000	95.929000	7.710000	62.491000	7.200000	97.044000
2017-01-16 00:00:00	5098.900000	99.237000	6.410000	82.299000	7.400000	29.004000	7.280000	98.086000
2017-01-17 00:00:00	4828.800000	99.031000	6.330000	78.386000	7.310000	21.359000	7.050000	94.815000

Figure 3.3: Tabulated data, flood 2017 number 1

All the other results of this analysis in tabulated form are presented in Appendix F. During flood events, there seems to be a general reoccurring trend: in the days prior to the flood, the discharge increases to a statistically high level, and during the event, a statically high water level occurs. In most cases, it is combined with a statistically large higher high tidal water level.

Moreover, in Appendix F, the 50 most extreme discharges and water levels are tabulated. These include the data from flood events. A clear correlation is seen between the occurrence of a high discharge and water level. Additionally, a clear correlation is shown between extreme water levels and a statistically large higher high tidal water level. This correlation is less clear between the discharges and higher-high tidal water levels that are statically high.

3.2. Bivariate Analysis

To simplify the multivariate analysis and its visualization, the concept of bivariate analysis was used repeatedly. A bivariate analysis assesses the relationship between two random variables. The strength of this relationship is expressed by the Pearson linear correlation coefficient. If the coefficient tends to $+1$ or -1 there is a perfect linear relationship between the two variables. For circular variables such as wind direction, a circular linear correlation coefficient was used instead. In this case, if the coefficient tends to $+1$ or -1, the linear variable follows a sinusoidal pattern.

3.2.1. Daily Data

The initial analysis was performed on all daily resampled variables in Table 3.1. Variables were considered if the absolute value of the correlation coefficient was larger than 0.5. This was done to reduce the amount of data that had to be visualized.

Firstly, the mean barometric pressure in San Francisco and Mallard Island approximately shows a one-to-one relationship and thereby obtained the highest correlation (Figure 3.4). It can thus be assumed that an air pressure drop at Mallard Island corresponds to an air pressure drop in the whole system downstream of Mallard Island.

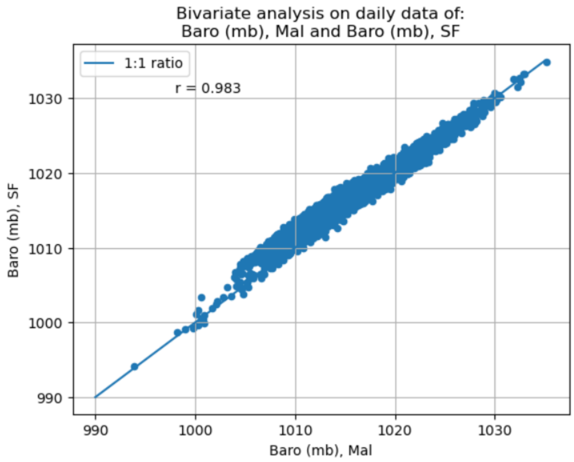


Figure 3.4: Bivariate analysis performed on daily means of pressure data measured at San Francisco and Mallard Island.

Secondly, the second-highest correlation was obtained between the maximum water level and maximum tidal water level. Interestingly, no other tidal data is correlated to the daily water level. This demonstrates that the water level is only dominated by tidal influence during the high tide.

Thirdly, the mean daily water level is related to the minimum and mean daily atmospheric pressure. This can be explained through a force balance. A decrease in air pressure decreases the force acting on the water level and therefore increases the water level. In larger bodies of water, the inverted barometer effect (Equation 4.6) can be used to quantify this effect.

Subsequently, the mean daily temperature and mean daily wind speed are related to the minimum, mean, and maximum daily atmospheric pressure. For temperature, all three increase with a decreasing temperature. Colder air is denser than hot air and therefore causes an increase in atmospheric pressure. For wind speed, air pressure increases with a decreasing wind speed. There are multiple reasons for this. This will be explored in section 3.2.2.

Finally, this analysis did not identify any significant relationships between discharge and other variables. The strongest correlation occurs between mean daily water level and mean daily discharge (Figure 3.5). This correlation seems to be stronger at the higher discharge events. There is a clear increase in water level when the discharge increases. However, with lower discharges, the correlation weakens and the water level varies due to other physical processes.

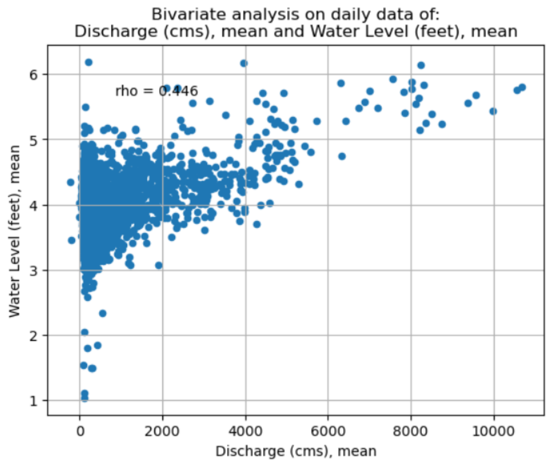


Figure 3.5: Bivariate analysis performed on daily means of discharge and water level measured at Chipps Island and Mallard Island respectively.

3.2.2. Yearly Averaged Weekly Data

For a second analysis, the weekly means averaged over the year were calculated per mean variables contained in Table 3.1. These values are used to identify a relationship with a third dimension: time. This will give insight into the seasonal effects of variables. To reduce the amount of data that had to be visualized, only water level and air pressure were compared to other variables. Air pressure is taken into account to check if the inverted barometer effect (described in chapter 2) applies for estuarine systems as well, or if other variables influence the air pressure.

In this second analysis, the highest correlation occurs between temperature and pressure (Figure 3.6). The explanation was done in subsection 3.2.1. Moreover, a clear time effect is observed as temperature, and thus it is concluded that air pressure depends on the time of the year (high in summer, low in winter).

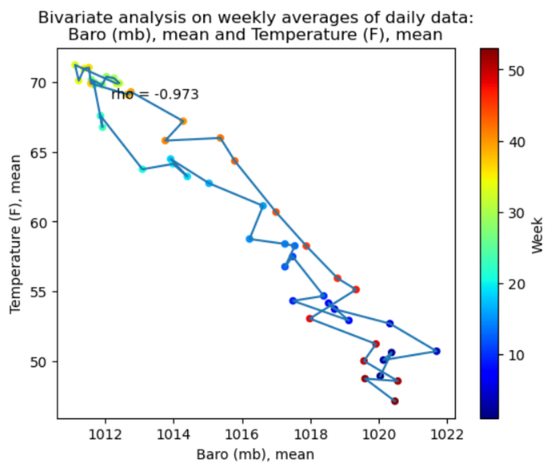


Figure 3.6: Bivariate analysis performed on yearly averaged weekly means of temperature and pressure measured at Mallard Island.

The water level is also correlated to pressure and temperature, but the time dependence is less clear due to the high water levels in January/February that occur during the storm season. Apart from these months, the water level increases throughout the start of the year and reaches a maximum in summer, similar to temperature, which coincides with a smaller atmospheric pressure, before decreasing until the end of the year.

Secondly, atmospheric pressure and water level are correlated to wind speed. Larger wind speeds occur in the summer which coincide with smaller air pressures and higher water levels.
For the former, larger wind speeds occur, which is in line with the expected sea-to-land breeze superposed by the westerlies in the summer (explained in chapter 2.
For the latter, wind set-up is proportional to wind speeds squared. Since the San Pablo Bay and the Suisun Marsh have a relatively shallow depth, it is prone to wind setup.

Thirdly, water level and atmospheric pressure also show a correlation with wind direction. The correlation is highest with air pressure because of the pressure gradient mentioned above. Interestingly, based on the 12 years of data, the average wind direction in the summer is northwesterly, which shows that diablo winds (also explained in chapter 2) aren't relevant on average. For water level, wind direction from the northwest/west maximizes wind setup in both the San Pablo Bay and the Suisun Marsh.

Lastly, a poor correlation is observed between the discharge and water level (Figure 3.7). As mentioned in subsection 3.2.1, the water level is only correlated to discharge at large discharges. This correlation can be seen in December, January, February, and March when delta outflow reaches its maximum. These large discharges occur because winter storms are frequent and dams need to release their reservoirs to accommodate the incoming precipitation.
However, in the summer and fall, during small delta outflow (approximately 200 cubic meters per second), the water level is clearly not correlated to discharge. As mentioned earlier, this correlation disappears because the water level is correlated to air pressure and wind setup during this part of the year. These effects have a larger seasonal influence on water level than discharge.

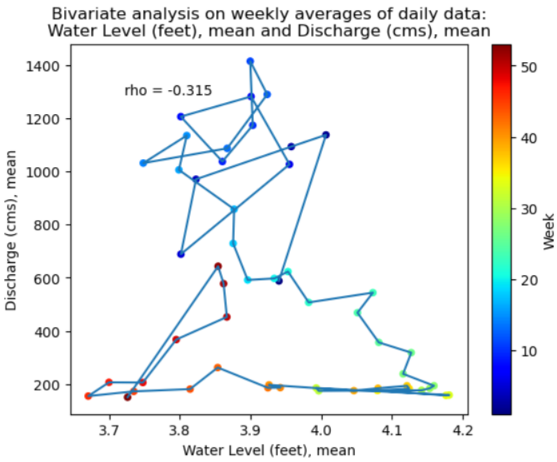


Figure 3.7: Bivariate analysis performed on yearly averaged weekly means of discharge and water level measured at Chipps Island and Mallard Island respectively.

3.2.3. Flood Events of 2017 and 2023

A final analysis was done for all the levee failures within the time record available (between 2011 and 2023, see Table 3.1). To simplify the analysis, only daily means of water level were compared to other daily mean variables in Table 3.1 as well as the maximum daily tide. Time was taken into consideration by looking at the month of the failure. This allowed the temporal effects of other processes on the water level to be identified.

As will be seen in this subsection, the correlation coefficients between certain variables will change. For example, in the case of discharge, the correlation coefficients will increase, as the correlation to water level is stronger during large discharges (as mentioned in subsection 3.2.1). Another example would be the relationship between temperature and pressure, which is affected by the low pressure caused by storms.

January Eleventh 2017

In the days preceding the flood, air pressure started to decrease, which demonstrates that a storm was moving toward the coast of California. This correlates with an increase in discharge (see Figure 3.9) and an increase in wind speed with a wind direction from the west.
The former indicates that dams started releasing water from the reservoirs to accommodate the incoming precipitation caused by the atmospheric river of the storm.
The latter is caused by the Coriolis effect caused by the low-pressure zone of the storm.

Moreover, it appears the flood occurred during the largest higher high tide of the month, maximizing the maximum daily water level (Figure 3.8).

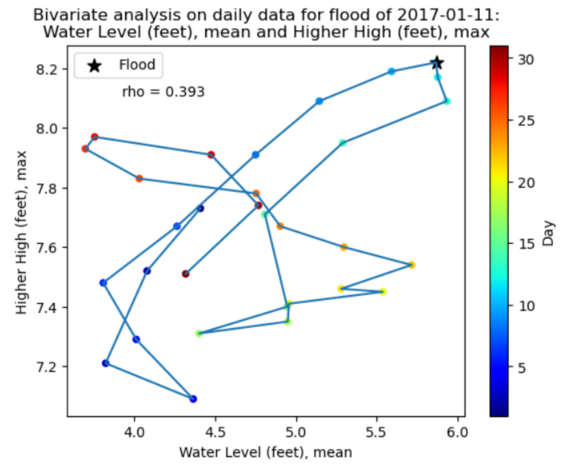


Figure 3.8: Bivariate analysis performed on daily means of water level and higher highs of tide during January of 2017

If the hysteresis curve of discharge and water level is drawn, it is observed that it is flipped (Figure 3.9). The water level reaches its maximum before discharge which is the inverse of a traditional diffusive flood wave. During the initial increase in water level, the increase in discharge was the dominant factor. However, after the flood, the water level stagnates for two days, even though the discharge increases. This coincides with an increase in air pressure and a drop in wind speed (decreases wind setup). This potentially shows that both of these effects have a significant effect on the water level during storm season.

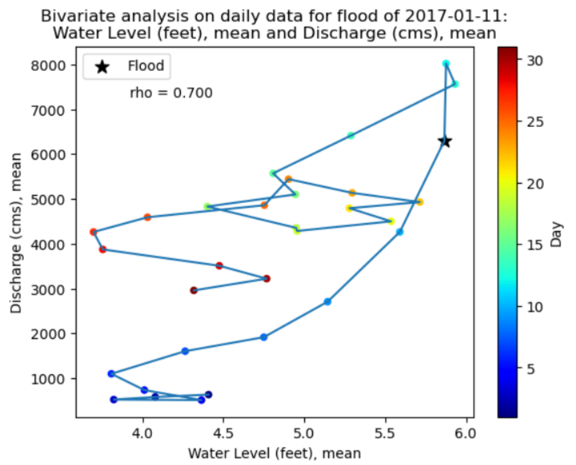


Figure 3.9: Bivariate analysis performed on daily means of discharge and water level during January of 2017

February Fifteenth 2017

As can be seen in Figure 3.10, a high discharge event caused by the Oroville spillway failure meant that discharge remained high for approximately 3 weeks.

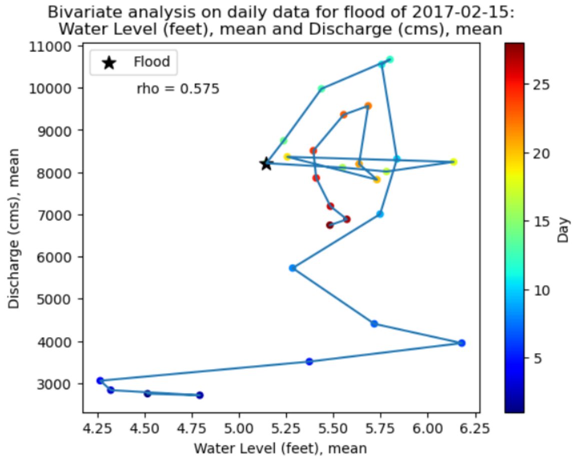


Figure 3.10: Bivariate analysis performed on daily means of discharge and water level during February of 2017

King tide occurred a couple of days before the maximum discharge, reducing the value of the maximum water level. The failure occurred during the neap tide when the higher high water was minimized. This also caused the maximum daily water level to reach its second-lowest value for the month.

Despite the neap tide, the daily mean water level remained high, most probably due to the high discharge event. It is therefore believed that the failure was caused by a high mean water level for an extensive period of time.

May Twenty-Fifth 2017

The third flood of 2017 was the only recent flood not located on the Sacramento River but on the Montezuma Slough. It occurred during the highest daily water level of the month. Compared to the two previous floods, the mean daily water level and the delta outflow were considerably smaller. Despite the lower water level and delta outflow, the island still flooded. This could be due to a crest height decrease that was caused by the previous high water level events in January and February of 2017.

The flood occurred during king tide and optimal wind direction and speed (Figure 3.11), which maximized the maximum daily water level and wind set-up. Finally, atmospheric pressure was at its smallest value for the month. When looking at the wind direction graph, it is clear that this breach was not caused by a storm, but by a different physical process, as the wind direction is constant throughout the month.

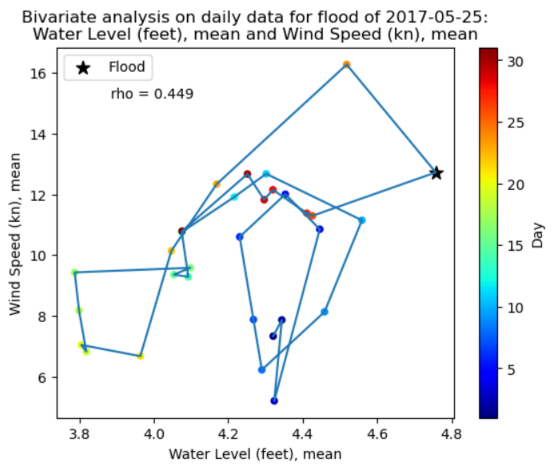


Figure 3.11: Bivariate analysis performed on daily means of wind speed and water level during May of 2017

January Fifth 2023

The first flood of 2023 occurred during the largest mean and maximum daily water level of the month. It did not occur during king tide. Delta outflow plateaued during the days leading up to the flood, meaning that discharge did not cause an increase in water level (Figure 3.12). Wind direction was also not favorable for wind setup. Therefore, the increase in water level seems to be caused by a decrease in pressure and an increase in tidal water level. However, these processes are not convincing on their own, as the linear summation of their effects does not equal the change in water level. This demonstrates that an additional physical process could have caused the increase in water level.

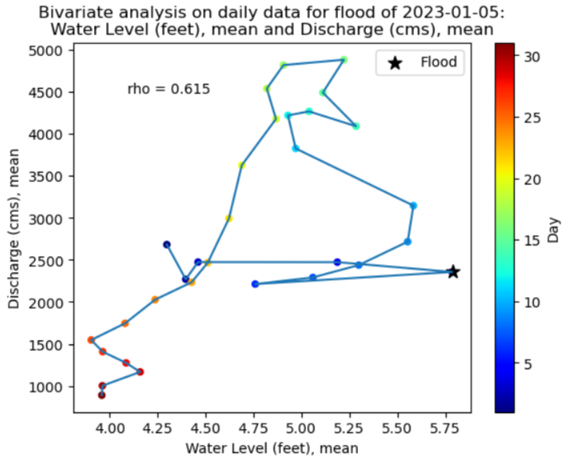


Figure 3.12: Bivariate analysis performed on daily means of discharge and water level during January of 2023

March Fourth 2023

Unlike, the aforementioned floods, the March 2023 breach was not caused by a high water level, but by erosion. As the island was still flooded due to the previous flood, the westerly winds generated winds causing erosion on the inner slope. More erosion was caused by tidal currents within the already-flooded island.

3.3. Conclusion

Van Sickle Island is located in a complex system. The strength of the correlation between certain variables depends on the dominating physical system (for example storms). This makes it unclear which physical processes cause the increase in water level at certain points in time as it is unknown which physical system is dominant.

Regardless of the dominating physical system, it has been shown that tide, wind, air pressure, and discharge all have their own effects on the water level. It has also been shown that the magnitude of their individual effect is conditional to the effect of the other physical processes. Finally, the analysis has also demonstrated that there are other physical processes at play as these four mechanisms can't always quantitatively explain the behavior of the water level.

Finally, the multivariate analysis shows that the inverted barometer effect also holds up for the estuarine system and can therefore be used in the Suisun Marsh system.

3.4. Recommendations

A bivariate analysis is limited as it is only possible to analyze two variables at once. This makes it difficult to comprehend the interaction between multiple variables. It would therefore be recommended to use artificial intelligence or a machine learning algorithm to simplify this process. If this is possible, additional variables should be included to ensure that all potential correlations are identified.

4 Hydrodynamic Model

4.1. Introduction

A hydrodynamic model was created to quantify the effect of different processes on the water level. These processes are tides, air pressure, wind setup, and discharge, and were identified in chapter 3.
To solve any model, the problem must be well-posed. This means a unique solution must exist, which is always dependent on the input data. This is reliant on the spatial domain, boundary conditions, and governing equations which will all be explained in this chapter.

4.2. Complexity of the Model

Numerical models are simplified mathematical representations of real-world phenomena, subject to assumptions and limitations. Models are therefore unable to capture the full complexity, uncertainties, and nuances of reality. Increasing the complexity of a model increases the amount of data required and the uncertainty associated with that data. A more complex model may be more accurate if and only if the inputs to the model are accurate.
Due to the complex behavior of the system, it was decided to use a simple 1-dimensional hydrodynamic model based on the backwater relation.

4.3. Spatial Domain

To further simplify the model, changes in width, depth, and bed friction were not taken into account. This simplification limits the number of inputs, making it easier to schematize a model of reality. Since the physical processes are simpler, it is easier to understand which variables cause a change in water level when using the model.

The simplification causes the model to have a constant channel width (B), constant bed slope (i_b), and constant non-dimensional friction coefficient (c_f). For channel width, a representative value for the width at the downstream and upstream boundaries was taken. Estimates for the other two values were obtained from the hydrodynamic San Francisco Bay-Delta model published in 2021 by USGS [4]. The bed slope was obtained by taking the quotient of the difference between the width-averaged bed elevation (z_b) at the upstream and downstream boundaries and the length of the channel between both boundaries. Finally, the non-dimensional frictional coefficient was used to calibrate the model. As an initial value before calibration, the value given by the USGS model was used [4]. The values mentioned in this section can be found in Table 4.1.

Table 4.1: Constants used in hydrodynamic model

Variable	**Value**
Bed slope, i_b	$1.58 * 10^{-4}$
Non dimensional friction coefficient, c_f	0.04
Downstream bed level, $z_{b,up}$	-16.1 m above NAVD88
Upstream bed level, $z_{b,down}$	-13.1 m above NAVD88
Length of model, L	19,000 m
Width of model, B	900 m
Spacing between grid points, Δx	10 m

4.4. Governing Equations

The governing equation is defined by the Belanger equation (equation 4.3). This equation is based on the Navier Stokes Equations. To obtain the Belanger equation from the Navier Stokes Equations, the characteristic period is averaged, hydrostatic pressure is assumed, the vertical and lateral directions are averaged, and steady flow is assumed. The model will therefore solve a 1-dimensional equation (in terms of flow).

The Belanger equation uses critical (Equation 4.1) and normal flow depths (Equation 4.2) to compute the gradient in water depth. Critical flow depth is the depth of flow for an arbitrary discharge which minimizes the specific energy. Normal flow depth is the depth achieved when the water depth gradient is equal to zero (uniform flow). Since the flow is subcritical (chapter 2), by definition it means that the normal flow depth is larger than the critical flow depth at all points in space. This is because the non-dimensional friction coefficient will be larger than the bed slope. The flow depth will tend to the normal flow depth upstream as the flow depth gradient decreases with the streamwise direction.

$$d_g = \sqrt[3]{\frac{q^2}{g}} \tag{4.1}$$

$$d_e = d_g \sqrt[3]{\frac{c_f}{i_b}} \tag{4.2}$$

$$\frac{dd}{dx} = i_b \frac{d^3 - d_e^3}{d^3 - d_g^3} \tag{4.3}$$

Within the equations, q is the discharge per meter width, g is the gravitational constant, d_g is the critical flow depth, c_f is the non-dimensional friction coefficient, i_b is the bed slope, d_e is the normal flow depth, and $\frac{dd}{dx}$ is the gradient in-depth.

Equation 4.3 is a first-order differential equation, which means that it can be solved numerically. In this project, it was decided to use the improved forward Euler method (Equation 4.4 and Equation 4.5) as it has a small accumulative error.

$$d_{i+1} = d_i + f(x_i, d_i)\Delta x \tag{4.4}$$

$$d_{i+1} = d_i + \frac{f(x_i, d_i) + f(x_{i+1}, d^*_{i+1})}{2}\Delta x \tag{4.5}$$

Within these equations, Δx is the spacing of the grid, $f(x_i, d_i)$ is the gradient in depth at location and depth i. Finally, $f(x_{i+1}, d^*_{i+1})$ is the gradient in depth at location i+1 after solving equation 4.4. The framework can be seen in Figure 4.1.

To reduce the accumulative error of the model, a smaller Δx is preferred. However, this increases the computational time. After performing a sensitivity analysis (Section 4.9), it was decided to use the value in Table 4.1.

For air pressure, the inverted barometer effect (Equation 4.6) was used to calculate its effect on the water level.

$$\Delta H = -\frac{\Delta P}{\rho g} \tag{4.6}$$

To simplify the analysis, it was assumed that an increase/decrease of 1 millibar from the mean value of the entire time record would cause a decrease/increase in water level of 1-centimeter along the entire length of the model [3]. At Mallard Island, this average value was determined to be 1015.5 millibars. It was decided to superpose the effect of the air pressure on the final solution as a conservative assumption.

Based on these governing equations, a visualization of the framework used for the hydrodynamic model is given in Figure 4.1.

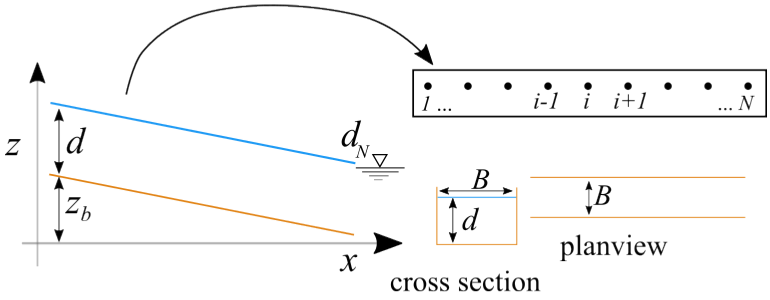


Figure 4.1: Framework used for hydrodynamic model [6].

4.5. Boundary conditions

Since the flow is sub-critical, upstream and downstream boundary conditions are needed. The location of the boundaries, as well as the path of the model, is shown in Figure 4.2.

Figure 4.2: Boundaries and path used for the hydrodynamic model [7].

4.5.1. Downstream

At the downstream boundary, the water level was defined and set at Martinez (Figure 2.3). It was composed of the tidal water level measured at Martinez and the wind setup effect in San Pablo Bay.

The tidal station at Martinez measured the water level in NAVD88. This was a necessity, as it made it easier to compare the water level at the upstream boundary with Mallard Island and the crest heights of the levees. Additionally, as can be seen in Equation 4.3, a water depth was required. The water level can thus be converted to a water depth by using the width-averaged bed elevation in NAVD88 [4] at the downstream boundary. At this location the river was clearly channelized, which was not the case at other tidal stations.

Secondly, to be able to assume mass conservation, the tributaries located in the control volume were assumed to be small and negligible.

Finally, the boundary is located upstream of San Pablo Bay, which is where the effect of wind setup is calculated. Therefore, superposing the setup effect with the tidal level at the boundary is a conservative estimate, as its magnitude should have decreased between San Pablo Bay and the boundary. For simplicity, it was assumed that wind setup could only occur in San Pablo Bay. This assumption is valid, as fetches are small within the model's physical boundaries, but are significant in San Pablo Bay. To calculate wind setup, Equation 2.1 is used. Further calculation is explained in chapter 2. Even though the wind setup is calculated in San Pablo Bay, the measurements of wind speed and wind direction of Martinez are used, as this is the closest representative station.

4.5.2. Upstream

The upstream boundary was set at Chipps Island. At this boundary, discharge was defined. It was composed of the discharge data from the delta outflow measuring station (Figure 2.3). There are three reasons for this.

Firstly, the station measures the $Dayflow$, which is an estimate from the California Department of Water Resources (DWR) of the daily discharge outflow of the delta. It does not account for tidal flows or barometric pressure changes. Therefore, it is concluded that the discharge is independent of both the tide and air pressure. [8]
Secondly, the station is located in close proximity to the measuring station of Mallard Island. Therefore, the water levels can be easily compared for calibration and verification.
Finally, the station is located three to six kilometers downstream of Van Sickle Island. Therefore, the water level calculated at the upstream boundary condition is a good representation of the water level at Van Sickle for the Sacramento Levees.

After solving Equation 4.3, the water depth was converted to the water level by using the width-averaged bed elevation, at the upstream boundary. The width-averaged bed elevation is the average bed elevation over the cross-section of the river perpendicular to the flow.

4.5.3. Remarks

The upstream boundary could have been placed further upstream. However, there are multiple reasons why this is problematic.

Firstly, there are two bifurcation/confluences between the boundary and Van Sickle Island, caused by the New York and Middle Sloughs. This will reduce the discharge in the control volume, providing an estimate of water level that will be overly conservative.
Secondly, the behavior of the system changes upstream of the upstream boundary. Downstream, the system is tidally influenced, whereas upstream the system is riverine influenced. This has an effect on the bed slope, sediment size (friction changes), and channel dimensions. Therefore, certain assumptions become invalid.
Thirdly, Van Sickle is located on an outer bend of the Sacramento River, which would make it susceptible to secondary flow effects. A one-dimensional model would be unable to take this into account.
Finally, there is no way to verify the value of the water level at Van Sickle as there are no measuring stations placed here.

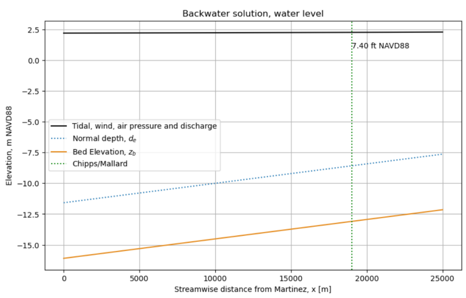
4.6. Calibration

As mentioned earlier in the chapter, c_f was used to calibrate the model. The flood event that occurred on the twenty-fifth of May was used as a reference. For this flood, the values of the tide, wind, air pressure, and discharge, as well as their effect on the water level are shown in Table 4.2. One of the assumptions made when deriving Equation 4.3, was a steady flow. By definition, this means that flow cannot change in time. This therefore means that the different processes such as tide are constant in time, and have an instantaneous effect on the water level.

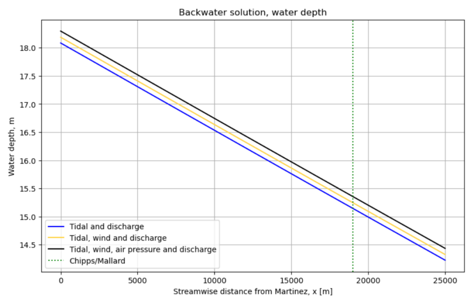
Table 4.2: Values caused by the four different physical processes on the flood event of May twenty fifth 2017.

Process	Sub Process	Value	Effect on Water Level	Application
Tidal (higher high)	————————	1.98 m NAVD88	1.98 m	Downstream Boundary
Wind Setup	Wind Direction	WNW	$0.1m$	Downstream boundary
	Wind Speed	19.44 knots		
Air Pressure	————————	1005 millibar	$0.1m$	Entire model
Discharge	————————	$1,700m^3s^{-1}$	$d_e = 4.52m$	Upstream boundary

The maximum water level measured at Mallard Island on the twenty-fifth of May was 7.37 feet NAVD88. A water level of 7.40 feet can be achieved by using a value for c_f equal to 0.04. While slightly larger than the reference value, the small difference in magnitude is negligible. All other constants used in the model are noted in Table 4.1.



(a) Water level change caused by Backwater relation



(b) Water depths caused by the summation of different physical processes.

Figure 4.3: Backwater relation for physical processes mentioned in Table 4.2

As can be seen in Figure 4.3a, the effect of discharge is weak. Its only purpose is to dictate the characteristic length scale (the length required to halve the difference in water depth between the downstream boundary and normal flow depth).

Moreover, in Figure 4.3b, it is shown that with this discharge, the wind setup effect is more or less constant throughout the length of the model. This is because it only accounts for a small portion of the water depth difference between the downstream boundary condition and normal flow depth. Therefore, the assumption made about superposing the effect of air pressure on the model is concluded to be valid and not overly conservative.

Figure 4.3b shows that the flood was clearly tidally dominated. However, wind, air pressure, and discharge all contribute to the final water level being significantly high.

4.7. Verification

Verification of the model was needed to ensure that the calibrated value for c_f was valid for an event that occurred at a different point in time. To perform this verification, the near miss on the twelfth of December 2012 is used. This near miss caused a water level at Mallard Island of 7.24 feet. The values for the tide, air pressure, and discharge and their effect on the water level are shown in Table 4.3.

Table 4.3: Values caused by the four different physical processes on the near miss event on the twelfth of December 2012.

Process	Sub Process	Value	Effect on Water Level	Application
Tidal (higher high)	————————	2.08 m NAVD88	2.08 m	Downstream Boundary
Wind Setup	Wind Direction	WSW	$0.04m$	Downstream boundary
	Wind Speed	8.36 knots		
Air Pressure	————————	1,010 millibar	$0.05m$	Entire model
Discharge	————————	$820m^3s^{-1}$	$d_e = 2.78m$	Upstream boundary

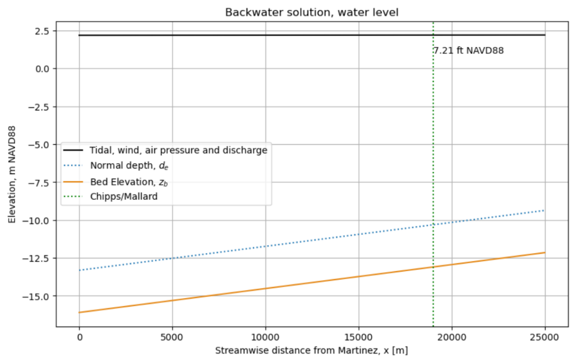


Figure 4.4: Backwater relation for physical processes mentioned in Table 4.3

The model shows that for a different set of events, the water level at Mallard Island can still be quantified accurately. Interestingly, the verification underestimated the water level, an inverse result from the calibration. The probable cause is the smaller discharge event, which caused a smaller normal flow depth. This means that a larger water depth reduction can take place over the length of the model.

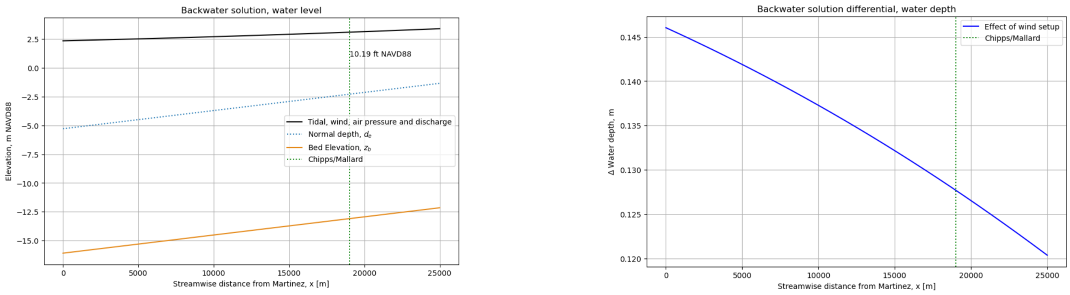
The verification shows that in an event with a relatively small discharge, no parameters had to be altered to arrive at an accurate estimate of the water level.

4.8. Validation

In the calibration and verification, events with relatively small discharges were used. Therefore, for validation, it was decided to investigate the water levels that occurred during large and extreme discharge events. For the latter, the extreme discharge event that occurred on January the twelfth was investigated. This event caused a maximum daily water level of 8.42 feet NAVD88. The values for tide, air pressure, and discharge and their effect on the water level can be seen in Table 4.3.

Table 4.4: Values caused by the four different physical processes on the flood event of January the twelfth 2017.

Process	Sub Process	Value	Effect on Water Level	Application
Tidal (higher high)	————————	2.09 m NAVD88	2.09	Downstream Boundary
Wind Setup	Wind Direction	WSW	$0.146m$	Downstream boundary
	Wind Speed	15.36 knots		
Air Pressure	————————	1,004 millibar	$0.11m$	Entire model
Discharge	————————	$6,300m^3s^{-1}$	$d_e = 10.81m$	Upstream boundary



(a) Water level change caused by Backwater relation

(b) Differential in between downstream boundary condition options.

Figure 4.5: Backwater relation for physical processes mentioned in Table 4.2

As can be seen in Figure 4.5a, the water level estimation at the upstream boundary clearly exceeds the measured value at Mallard Island by almost 2 feet. This demonstrates that the model has issues with predicting the water level at Mallard during extreme discharge events. There are multiple reasons for this, either related to fundamental assumptions made to create a 1-dimensional model, assumptions made to quantify the effect of physical processes, or assumptions related to how the spatial domain was schematized. Several potential reasons are summed up in the following paragraphs.

Firstly, the model assumes steady flow and assumes processes have an instantaneous effect on water depth. This means that the flow needs to be constant in time. However, processes such as the tide and discharge are time-dependent. For example, during extreme discharges, a flood wave will form. Due to the location of Van Sickle being in the lower reaches of the basin, the wave will diffuse, creating a discharge-water level relationship that is dependent on time. In a diffusive flood wave, the peak in discharge precedes the peak in water level as it is by definition unsteady. Extreme discharges therefore violate one of the assumptions made to simplify the model.
Secondly, as can be seen in Figure 4.5b the backwater effect has a larger influence on the water level increment caused by wind setup over the length of the model. This larger influence is caused by a larger normal flow depth. Therefore the wind setup effect accounts for a larger portion of the change in water depth than needed to reach normal flow depth. This means that the assumptions related to the wind setup (increment in San Pablo Bay is the same as the increment at the downstream boundary) and air pressure (superposed on the final solution) overestimate the water level at the upstream boundary during extreme discharge events.
Thirdly, the inaccuracy in prediction is also caused by the model schematization. As can be seen in Figure 4.2, Suisun Bay is not channelized. It consists of multiple ebb/flood channels within an intertidal area. During high water level events, this area can store large quantities of water, which decreases the water level. The model is therefore too simplistic, as it does not take changes in width, bed slope, depth, and c_f into account. This has a direct influence on the normal flow depth, as it becomes non-uniform in space. This would thus create different backwater curves throughout the model.
Additionally, the downstream boundary is not located in an optimal position. It is placed in a position where the tide is chocked, which increases the value of the tidal prediction, and discharge still affects the boundary. A final explanation for the inaccuracy could be related to the calibration/verification of the model that only took place at small discharges. The value of c_f is therefore only representative for a specific part of the data set.

Apart from the previously mentioned explanations, there could be another distinct reason why the water level might be over-predicted. This explanation is related to the accuracy of certain input variables, for example, discharge. Instead of being measured, it is calculated by performing a force balance. Moreover, it is only calculated on a daily basis. During large dam releases, the value of the discharge could thus be overestimated.

Via this validation, it is concluded that the model has a validity range that depends on the value of discharge.

4.9. Sensitivity Analysis

A sensitivity analysis is performed to demonstrate which variables assumed to be constant have a large effect on the model. It was decided to investigate the effect of bed slope and the spacing of grid points, as these have the biggest uncertainty. The physical processes used for this sensitivity analysis, their values, and their effect on the water level are stated in Table 4.5. The constants shown in Table 4.1 are not changed unless a sensitivity analysis is performed on them.

Table 4.5: Values used for the physical process in the sensitivity analysis.

Process	Sub Process	Value	Effect on Water Level	Application
Tidal (higher high)	————	1.98 m NAVD88	1.98 m	Downstream Boundary
Wind Setup	Wind Direction	WNW	$0.1m$	Downstream boundary
	Wind Speed	19.44 knots		
Air Pressure	————	1015 millibar	$0m$	Entire model
Discharge	————	$1,000m^3s^{-1}$	$d_e = 3.17m$	Upstream boundary

The bed slope was calculated as the average elevation difference over the length of the model based on the width averaged bed elevations. The value used is therefore sensitive to the accuracy of the bathymetry used in the USGS model. A small change in the bed slope (values and their effect on normal flow depth can be seen in Table 4.6) has a large effect on the water level.

Table 4.6: Bed slope values used

original i_b	$1.58e-4$
$i_b + 1e-5$	$1.68e-4$
$i_b - 1e-5$	$1.48e-4$

This effect can be seen in Figure 4.6. On one hand, a shallower bed slope causes the water depth to increase by 19 cm. On the other hand, a steeper bed slope causes the water depth to decrease by 19 cm. This is one order of magnitude larger than the model error calculated during calibration and verification. Therefore, the input values for the bathymetry have a significant effect on the calculated water level at the upstream boundary.

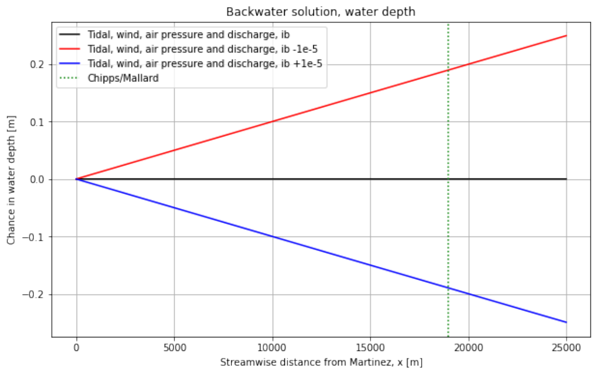


Figure 4.6: Sensitivity of water depth caused by different values of bed slope.

The second parameter reviewed was the spacing between grid points. This spacing has a direct influence on the accuracy and the speed of computation. A larger grid size means that fewer computational points are required, which decreases the computational time. For example, a grid that requires 10 times as many points, increases the computation time by the same amount. It can be assumed that as the grid spacing tends to zero, the numerical solver reaches a truncation error of zero as the forward Euler is consistent with Equation 4.3. Therefore, the smaller the grid size, the more accurate the model is.

As can be seen in Figure 4.7, the difference in water depth increases with length, and a smaller grid size causes a larger value for water depth. This means that a smaller grid size is more conservative. Moreover, both curves in the figure show a logarithmic relationship, this is caused by the logarithmic scale.

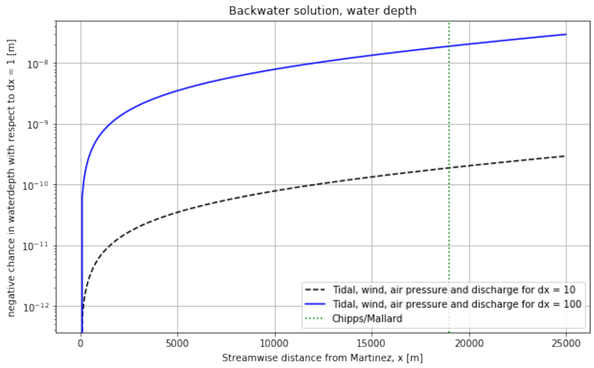


Figure 4.7: dx set to 100 in comparison to original dx (dx = 10), with dx = 1 as baseline

Figure 4.7 shows that the increase between a grid spacing of 1 and 10 meters is not significant, and is not worth the increase in computational time. On the other hand, there is a larger difference between a grid size of 10 and 100 meters. Since smaller grids are more conservative and accurate, it was decided that this difference in water depth was worth the increase in computational time.

Nevertheless, Figure 4.7 demonstrates that the model output is not sensitive to the grid spacing, as the difference in water level is negligible when compared to the model error observed during the calibration and verification model runs.

4.10. Conclusion

While the hydrodynamic model is a simplistic version of reality, it can quantify the effects of different physical processes on the water level upstream. The model is sensitive to bathymetry inputs but provides estimates that are reasonable to observations made in the past. During extreme discharges, the model is unable to give a realistic value for the water level at the upstream boundary condition as numerous assumptions are violated. The model therefore has a validity range, which is conditional on the value of discharge.

4.11. Recommendations

For further research, the hydrodynamic model could be improved in several ways. Currently, the water level cannot be accurately measured at the different levee sections of the island. It would therefore be recommended to take into account the different components of discharge (such as precipitation, dam releases, and runoff), and place the upstream boundary further upstream. This would mean that the entire delta and Bay Area would have to be modeled. This would increase the number of inputs required, increasing the uncertainty associated with the model. Furthermore, the issues at high discharge events, noted in the validation could be resolved by using a governing equation that does not assume steady flow.

5 Statistical Model

5.1. Cost Benefit Analysis

To assess and compare the different types of management plans a cost-benefit analysis will be performed. The result of this analysis will be a tabulated overview of all costs/investments and benefits associated with a certain plan, which allows for comparison between them. Benefits are the positive outcomes of a new management plan, for example, a decrease in annual risk or economic benefits due to selling property. Costs are for example the construction costs needed to arrive at a higher levee height. Apart from economic costs, there are also social, economic, and political costs, which are costs that can not easily be expressed in monetary values. If a monetary value can be used, it will be taken into account for the conclusion of the cost-benefit analyses. If no monetary value can be used, they will still noted in the report, as they could influence a reader's opinion about the proposed management plan.

To be able to take both yearly and one-time costs and incomes into account, the concept of the Net Present Value (NPV) is introduced. The NPV calculates the difference between the present value of cash inflows, such as revenue or savings, and the present value of cash outflows, such as investments or expenses, over a specific period of time. It accounts for the fact that money received or spent in the future is worth less or more than money received or spent today due to the time value of money and inflation. The result of this calculation can be positive, negative, or zero. A positive NPV indicates that the investment is expected to generate a return higher than the required rate of return [9].

The formula to calculate the Net Present Value is shown in equation 5.1, where C_t is the cash flow in year t [$], r is discount rate per year, and T is the reference period [years]. The discount rate is assumed to be 4% [10].

$$NPV = \sum_{t=1}^{T} \frac{C_t}{(1 + r^t)} \tag{5.1}$$

5.2. Risk

In this project one of the main components of costs is risk. Risk is defined as follows:

$$Risk = Probability * Consequence \tag{5.2}$$

Using this expression, risk is expressed in monetary units ($), which allows for quantitative comparison. In risk, the relevant probability is the probability of failure. Failure occurs due to a cascade of events caused by an initiating event, which results in unwanted consequences. For example, a storm leads to a high water level, which causes water to overtop the levee, causing erosion, and eventually, this leads to a flood. Thus, initiating events have a probability of occurrence and if they occur, there is a conditional probability that failure occurs. This total probability of failure can mathematically be described as the probability of failure given an initiating event multiplied by the probability of occurrence of that initiating event, as shown in Equation 5.3.

$$P(F) = P(F|I) * P(I) \tag{5.3}$$

- F = Failure
- I = Initiating event

In this project, failure is defined as a breach of a levee section, which leads to partial or complete flooding of Van Sickle Island. The primary mechanism that causes such a breach is overflow. Overflow occurs when the water level exceeds the crest height, thus the initiating event. Unfortunately, crest height information is outdated and the water levels along the different levee sections can not be correctly quantified. Therefore, a standard approach, to assess the initiating event is not possible. However via visual inspection of the water level data at Mallard during known near misses and failures, it could be concluded that overflow occurred whenever water levels exceeded seven feet. This seven feet is assumed a representative value to determine whether overflow occurs at Van Sickle Island. Therefore, a water level exceeding seven feet at Mallard Island is determined as the initiating event that can lead to potential failure. See Figure 5.1 for a visual explanation of this concept.

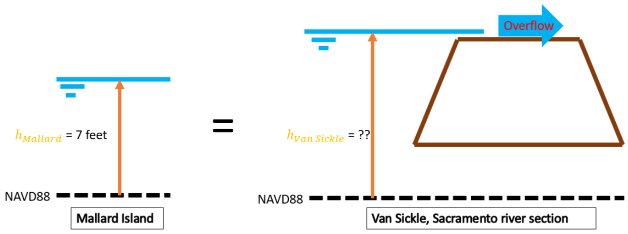


Figure 5.1: Concept determining overflow threshold

With the initiating event defined, a probabilistic method to asses $P(F|I)$ and $P(I)$ has to be chosen. Though there are many statistical methods to assess failure probability, event trees have the special ability to clearly show events included in a model after an initiating event. They are commonly used to simplify and visualize complex systems, like Van Sickle Island. An event tree is composed of many chains of events, all starting from the initiating event.

Figure 5.2 explains the process. Following event A (the initiating event), three follow-up events can occur, namely event B_1, B_2, and B_3. The probability of those events occurring is conditional to event A ($P(B_1) = P(B_1|A)$). To get the total probability of a 'branch', for example, the chain of events caused by A and B_1, the probabilities need to be multiplied. To get the probability of failure, the probabilities of each 'branch' exceeding the failure threshold are summed up, in this case, branches B_1 and B_2. If a branch does not lead to failure, it is not taken into account in the summation, like branch B_3.

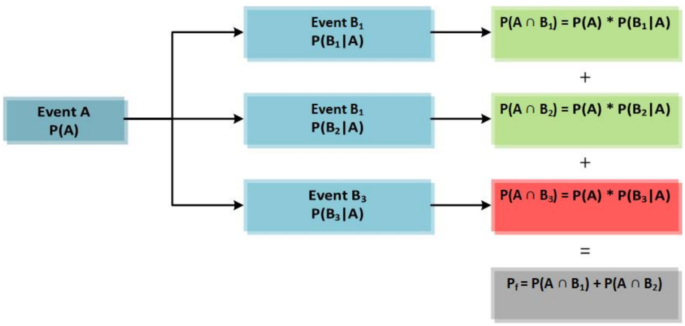


Figure 5.2: Concept of an event tree

To use an event tree correctly, two rules have to be satisfied: the outcomes need to be mutually exclusive (if one event happens the other events cannot happen) and collectively exhaustive (an event needs to happen, this leads to having a total probability per branch of 1). An event tree can have both dependent and independent events. For independent events, the probability of the B-events doesn't change based on the outcome of event A ($P(B_1|A) = P(B_1)$). If the events are dependent, this rule does not hold, and event B_1 would have a probability that is conditional on the outcome of event A.

To determine the probability of occurrence of the initiating event, a special type of event tree is used, namely a logic tree. A logic tree uses states of nature that influence the initiating event as its columns, and together make up its probability of occurrence ($P(I)$). Once this probability of occurrence is known, the event tree that leads to the probability of failure ($P(F)$) can be computed.

5.2.1. Probability of Occurrence Initiating Event

As shown in Appendix I an attempt was made to calculate the probability of occurrence of the initiating event by using the concept of a logic tree. This used the hydrodynamic model presented in chapter 4. However, after setting up this method and performing the analysis, it was noticed that with the current available data using this method was not possible. So this method is not a part of this chapter anymore. However, to obtain a deeper understanding of the physical processes that lead to a flood Appendix I can be read.

So another method had to be used to assess the probability of occurrence. This method was chosen to be an Extreme Value Analysis. For this analysis, data was only available for a short period of time (less than 25 years), so the Peak Over Threshold method was used. The threshold used to determine whether or not a value was considered extreme was set at 6.6 feet, this is equal to the 99th percentile. The declustering time, the minimal time between 2 extreme events was set to 28 days (obtained after performing a visual inspection of the water level data). This is done to ensure that extreme values are independent and identically distributed. The extreme water levels that were identified are shown in Figure 5.3.

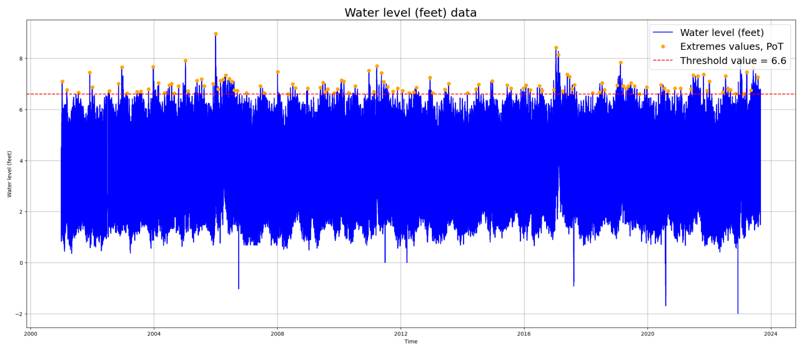


Figure 5.3: Graph of extreme water levels.

For these obtained extreme water levels a generalized Pareto distribution was fit using the Maximum Likely hood Estimator method. This fitted distribution together with the empirical data is shown in Figure 5.4. Also, a QQ plot is shown to allow for a visual assessment of the goodness of fit. As can be seen in the figure the distribution estimates the low empirical values correctly but fails to do this for high values. The goal of an extreme value analysis however is to correctly fit the high values. So the distribution is incorrect for the assessment that needs to be performed. An effect of this incorrect distribution is that the return periods of events will most likely be overestimated. A better distribution would be if the distribution would be skewed upwards.

However, since obtaining a probability of occurrence is of vital importance for the continuation of this project this distribution will still be used. It gives an estimate for the probabilities of occurrence which is better than not having any probability at all. Keep in mind that this will most likely overestimate the return values of extreme water levels.

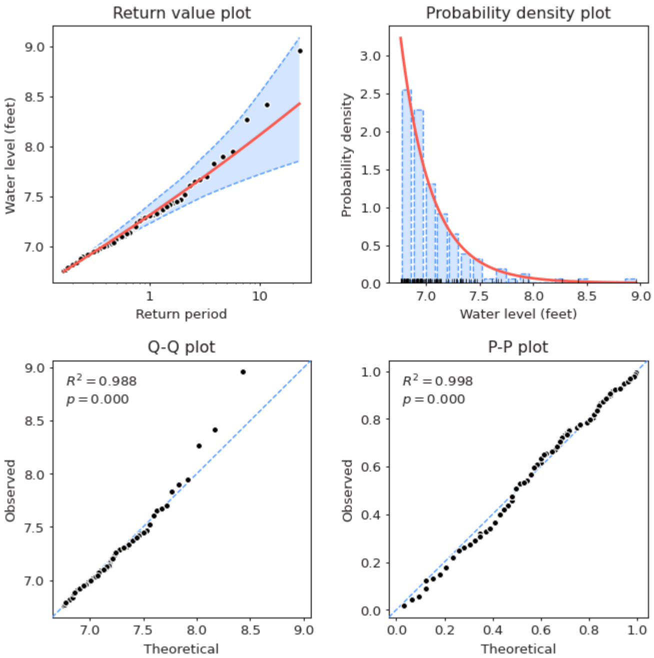


Figure 5.4: Generalized Pareto Distribution of extreme water levels.

By using this distribution the return period of a water level of 7 was calculated. This return period was 0.4 years. This means that on average per year 2.5 events occur with a water level that exceeds 7.0 feet.
Some return periods of important water levels for this distribution are shown in Figure 5.5.

Return period	Average occurences per year	Water level (feet)
0.2	5.00000	6.811410
0.4	2.50000	7.024374
0.8	1.25000	7.244963
1.0	1.00000	7.317644
2.0	0.50000	7.548731
10.0	0.10000	8.117710
25.0	0.04000	8.463016
100.2	0.00998	9.017766

Figure 5.5: Return period Generalized Pareto Distribution

5.2.2. Event Tree

To determine whether or not failure occurs given the water level exceeds 7 feet. 2 processes were identified as important. These processes were if emergency measures were taken and what the overflow volume was. So these processes were taken as the columns in the event tree. The choices of thresholds per column and their probabilities will be described in the next sections.

Emergency Measures

When the water level exceeds the crest height, it is important to know whether emergency measures were taken. It is known that the reclamation district applies emergency measures 3 times per year on average. Note that this corresponds to the average annual occurrence of water levels higher than 7 feet per year. Furthermore, for all the documented floods and near misses emergency measures were taken. Based on this information it is assumed that the probability of applying emergency measures given that the water level exceeds the crest height is 1.

It was also considered to include the process of whether or not emergency measures were applied correctly and which types of emergency measures were applied. However, due to a lack of documented data on extreme events, this could not be done. Only four failures and three near misses were correctly documented. In the data 39 near misses were identified.

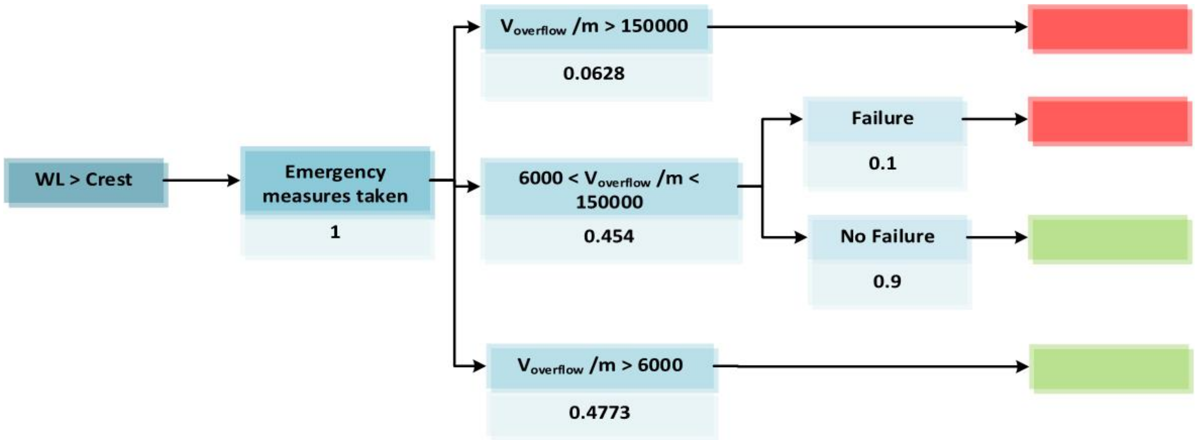


Figure 5.6: Visualisation event tree.

5.2.3. Overflowing Volume

The second important process is the amount of overflowing volume. To calculate the overflowing volume per meter width, the overtopping discharge needs to be calculated, this is dependent on the freeboard (R_c). The freeboard is the difference between water level and crest height. Equation 5.4.

$$q_{overflow} = 0.54 * \sqrt{g * |-R_c^3|} \tag{5.4}$$

In this equation, R_c represents the freeboard and 0.54, a constant value, which may vary depending on the crest geometry. Subsequently, the overflow discharge is integrated over time to obtain the overflow volume per meter width (Equation 5.5).

$$\int_{t1}^{t2} q_{overflow}\, dt \tag{5.5}$$

To perform this integration a time interval was defined. For near-miss events, this time interval is from 14 days prior to the most extreme water level to 7 days after this. For events that led to failure only the 14 days prior to failure are taken into account. Because if the 7 days after failure were also taken into account this would give an overestimation of the overflow volume that leads to failure.

After visual inspection of the tabulated data shown in Appendix H, three distinction between volumes were made.

- If the total overflow volume per meter width is greater than 150,000 m^3 m^{-1} failure occurs, the probability of this event is $3/44 = 0.0681$.
- For a total overflow volume per meter width between 6,000 m^3 m^{-1} and 150,000 m^3 m^{-1} failure can occur. The probability of occurrence of this event is $20/44 = 0.4545$. The probability of flooding given this event has occurred is $2/20 = 0.1$, it follows that the probability of not flooding is then 0.9
- For a total overflow volume per meter width below 6,000 m^3 m^{-1}, the probability of occurrence is $21/44 = 0.4773$, The probability of flooding given this event has occurred is 0, since no recorded flooding has ever occurred during such an event.

5.3. Conclusion

The annual probability of failure occurring is calculated by multiplying the average amount of events that could lead to failure per year (2.5) with the probability that failure occurs during this event (0.11) of any event. This leads to an annual probability of failure occurring of 0.28.

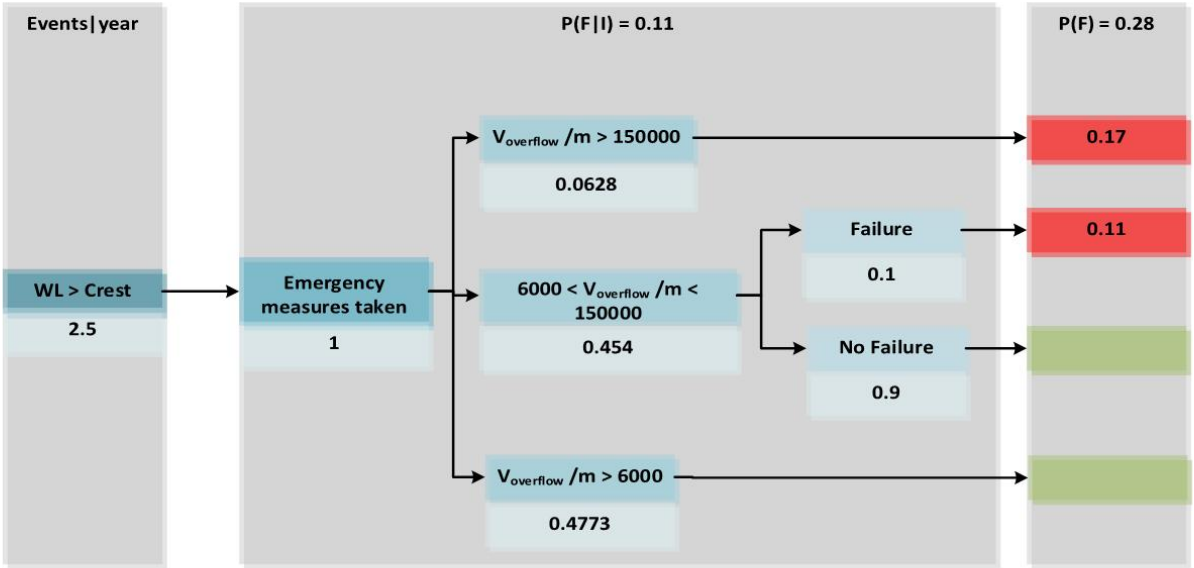


Figure 5.7: Caption

5.4. Recommendations

Currently, the probability of failure of the levee system only assesses failure due to overflow. However, the levee can also fail due to other mechanisms. So in a more complete approach, multiple failure mechanisms would be taken into account when assessing the risk, for example, earthquakes. The most important mechanism that should be investigated further is settlement (Appendix C).

Apart from other failure mechanisms, processes that increase the risk over time (like sea level rise), are not taken into account in the current analysis. The inclusion of this process would again contribute to a better estimate of the total risk.

In the case where another statistical model could be used, similar to the logic tree located in Appendix I, the extreme value analysis can be used to validate the model.

6

Management Plan A: Status Quo

In this chapter, all the costs and incomes of the current management of Van Sickle Island referred to as the status quo, are explained. For a clear comparison between the management plans, a similar structure is followed. The final result of this chapter shows an overview of the costs and incomes of the status quo.

6.1. Concept

Keeping Van Sickle Island at status quo means continuation of the current management of the island. The levees will thus be maintained as described in chapter 2 and the island continues to be primarily used for gas extraction, recreation, and duck hunting. Important parts of the maintenance worth noting include the focus on weaker spots of the levees, repairs after flooding, and the application of emergency measures. There are roughly 3 types of emergency measures that have been taken on Van Sickle Island in the past, these are shown in Table 6.1.

Table 6.1: Overview Emergency Measures.

Measure	Goal	Disadvantages	Costs
Sand Bags	Raises levee to prevent or delay overflow and overtopping	Limited height: effective up to 5 feet (USACE)	$1.50 per filled bag, can be less expensive or free depending on source sand and bags
Aqua Dam	Raises levee to prevent or delay overflow and overtopping	Buoyant when water level becomes too high or water flows underneath.	$16 per foot levee (18 inches high)
Plastic/Geotextile	Prevents erosion of levee slopes	If not installed properly, water can still flow underneath and erode the levee	$0.15 - $0.50 per square foot

6.2. Costs

Monetary values associated with the current management of Van Sickle Island were obtained from the yearly budgets. The python code that analyzes them can be found in Appendix J.

Investment Costs

Since the levees in the status quo maintenance plan are maintained to the current status of the levee, no investments are needed. However, investments were made in the past to reach the current state of the levees.

Maintenance Costs

Maintenance is an essential part of the management plan and will be done on a continuous basis. For estimating the average maintenance cost per year, the annual costs of all levee maintenance from 1960 to 2022 are assessed and averaged. In this calculation, costs for emergency repairs are excluded and inflation is accounted for This results in an average maintenance cost of $235,000 per year.

6.2.1. Current Risk

The main contributor to the costs of the status quo management plan is associated with the current flood risk of Van Sickle Island.

During high water events, a levee section on Van Sickle Island can fail. Failure is, in this case, defined as the loss of structural integrity of the levee due to total erosion of the levee cross-section resulting in flooding of the island. This is most likely to happen due to long-term significant overflowing of the levee. A schematic overview of how failure occurs is shown in Figure 6.1.

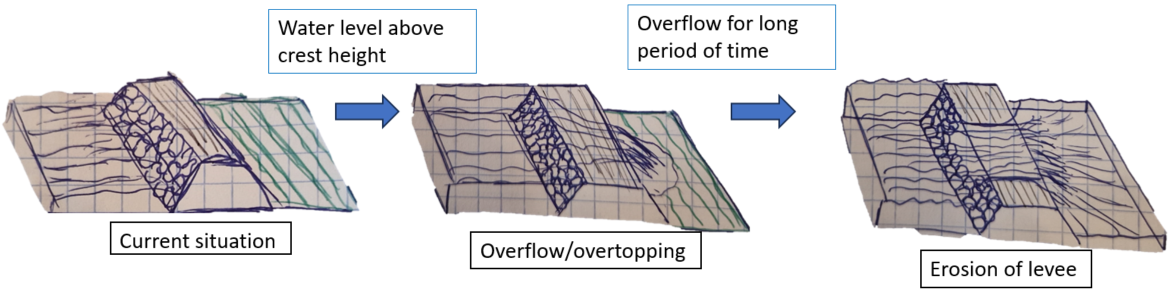


Figure 6.1: Schematic/sketched overview of failure levee.

Probability of Failure

The probability of failure of the levee due to overflowing is estimated at 0.28. The calculation is found in Section 5.3.

Consequences of Failure

Failure of the levee has financial consequences since it requires a repair and damages property on the island. An overview of all consequences due to flooding is schematized in Figure 6.2 and listed Table 6.2.

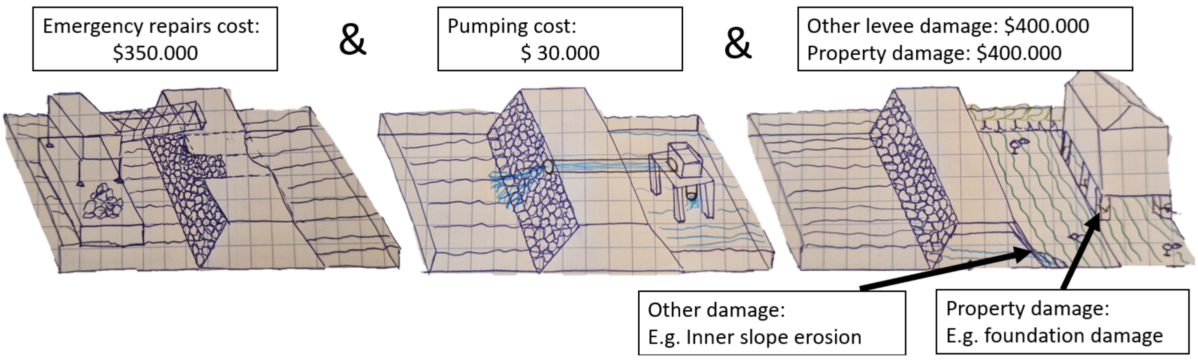


Figure 6.2: Schematic/sketched overview of consequences of failure.

Table 6.2: Overview consequences of a levee failure.

Emergency Repairs
The estimation of an emergency repair of a failed levee is **$ 350,000** See Appendix K for estimation of breach sizes in 2023 and a more detailed overview of all repair costs.
Damage to Properties/Personal Damage
The estimated costs of damage per building per flood is $5000. There are 20 buildings on the island, so the total cost of damage to buildings is **$100,000.** Apart from buildings there are duck blinds, which are dug into the ground It is assumed these are completely destroyed in the event of a flood, with a cost of $3000 per blind With a total of 20 blinds, this is a total of **$60,000** It is also assumed there is no financial damage to the land itself, only to the properties on the land. Do note this damage is not paid by the reclamation district but by the landowners themselves.
Damage to Levees
These are the costs due to inner slope erosion or damage to roads. the costs of emergency measures are excluded. Inner slope erosion occurs due to wind-generated waves inside the island while it is flooded. These damages are estimated to be **$400,000** for the flood of 2023 and is assumed to be constant for all other floods. Do note that because no money is available to pay for these repairs these will probably never be done. By not performing these repairs the probability of flooding will most likely increase.
Costs of Pumping
The costs of pumping to achieve a normal water level after flooding are estimated to be **$30,000** per flood. This is calculated by estimating the volume of water that needs to be drained, assuming a constant price for electricity and a constant outflow of a pump for a certain amount of electricity.
Gas Extraction Losses
For the gas extraction company located on the island the damages to their facilities are estimated to be **$250,000** Note this excludes the costs of not being able to pump. When gas prices are high per day revenues of $100,000 can be made. This is not taken into account, but could significantly change the total costs.
Total: $1,185,000

6.2.1.1. Political, Environmental, and Social Consequences

Apart from these economic consequences, there are political, environmental, and social consequences that cannot be expressed in monetary values. These are therefore not considered in the cost assessment but are nevertheless still worth mentioning since they influence the feasibility of the status quo management plan in practice.

Socially, flooding interferes with the intended use of the island by limiting access to property, damaging property, and limiting duck hunting in blinds. The latter can be resolved by hunting from boats instead of blinds, thereby circumventing the negative effects.

Politically, applying for state and federal emergency funds in the aftermath of a flood requires a lot of effort and time. Even with an eligible application, receiving the funds is not assured since other disasters might have priority and the total emergency fund budget of governments is limited. This causes uncertainties for the Reclamation District and landowners about when and to what extent the levees and properties can be restored.

Ecologically, the habitat of both aquatic and land animals is affected. When the island floods, the habitat of land animals decreases, and new habitat forms for aquatic animals. When the levee is repaired and the island is drained, the habitat of aquatic animals decreases and the habitat of land animals is restored. Therefore, both the flooding and repair, significantly impact the survival rate of animals.

6.2.2. Total Risk

The economic consequences and failure probability as described above can be used to calculate the risk of the status quo management plan. The total risk is the probability of failure times the consequences (in monetary values) of failure: Risk = $0.28 * \$1185,000 = \$331,800$.

6.3. Income

The income of the Reclamation District also referred to as benefits, consists of the annual contributions of the gas company and landowners, and the reimbursement for repairs by the state of California or the federal government. Based on the financial reports of the Reclamation District, the income for every year between 1960 and 2022 was determined. After accounting for inflation, the mean value was taken such that an average total income per year could be estimated. This is $400,000.

6.4. Summary

The assessment of all the current costs and incomes for the status quo is shown in Table 6.3. As is shown there is an annual loss of $366,800 for keeping the island at status quo. It is important to note that an annual risk of $331,800 is only spent when Van Sickle Island

Please note that having an annual risk of $331,800 does not imply that this amount is spent on an annual basis; it will only be spent in case of flooding. In conclusion, the continuation of the current management plan is economically non-feasible since on average more money is lost than made.

Table 6.3: Overview total costs and incomes for Status Quo.

	Costs	
Risk	Consequences for district:	
	Emergency repairs Post flood pumping Other levee damage	$350,000 $30,000 $400,000
	Consequences for land owners:	
	Personal/property damage Gas well damage	$150,000 $250,000
	Pf	0.297 per year
	Total (per year)	$1,185,000 * 0.28 = $331,800
Maintenance	Cost per year	$235,000
	Total (per year)	$235,000
General expenses	Cost per year	$200,000
	Total (per year)	$200,000
		Total costs (per year) **$766,800**
	Income	
General income	Annual land owner fee State reimbursement Emergency funding Contributions	
	Total (per year)	$400,000
		Total income (per year) **$400,000**

Income minus costs = **-$366,800** per year

Table 6.4: NPV for future years for management plan A.

	Now	5 years	10 years	50 years
NPV	-$366,800	-$1,999,728	-$3,341,877	-$8,246,465

7

Management Plan B: Raising the Levees

7.1. Concept

The probability of failure for flooding has a direct relationship with the water level. For flooding due to overflow, the freeboard (distance between levee crest and water level) must be less than zero. This is also called a freeboard deficiency. Increasing the crest height will decrease the chance of this deficiency occurring, and therefore reduce the probability of failure. This is because a smaller number of events in the statistical model causes the crest height to be exceeded. In this management plan, levees will be raised as this will decrease the probability of failure, and decrease the risk associated with the island. An economical optimization will be conducted to identify which crest height provides the largest amount of benefits.

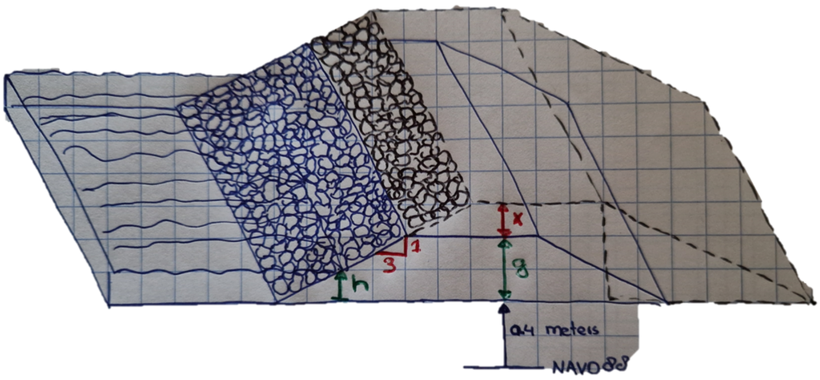


Figure 7.1: Conceptual plan of raising the levee.

7.1.1. Application

Since Van Sickle Island's levees are split into five different sections (see chapter 2), the levees can be non-uniformly raised depending on the current height and exposure to water at the section. The levee section along the Sacramento River is the most susceptible to overflow, as seen in chapter 2. Moreover, the crest heights are not constant along each section.

Some assumptions have to be made to simplify the calculation of the costs of raising the levee. A sheet pile is located along the Sacramento River. This will be taken into account by assuming an equivalent levee is placed in its location with a crest height of 9 feet NAVD88. Secondly, crest heights along the different levee sections will be taken from the latest 5-year plan published in April 2022 [1] (Table 2.3). Approximately 0.68 feet will be subtracted from the lower limit of a given contour. For the smallest contour, an additional foot was subtracted from the upper limit. There are two motivations for this. Firstly, the surveying was completed in 2017 and 2019 which was half a decade ago. Therefore, the crest height will have decreased because of wear (settlement is an example mechanism that can cause this). Secondly, in chapter 5, it was mentioned that if the water level at Mallard Island exceeds 7.00 feet, overflow occurs. Based on this information, an intermediate datum was introduced with a datum conversion factor estimated to be 0.68. This factor is used to go from an overflow water level at Mallard to the crest height at which overflow occurs at Van Sickle. This is graphically illustrated in figures 7.2 and 7.3. In this example, a crest height of 7.68 feet NAVD88 is used.

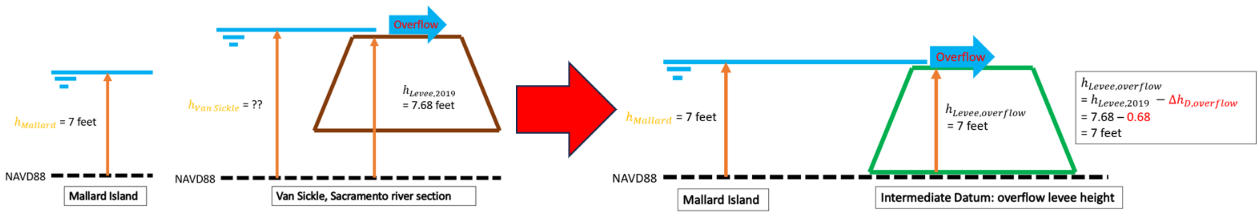


Figure 7.2: Concept of intermediate 'datum'

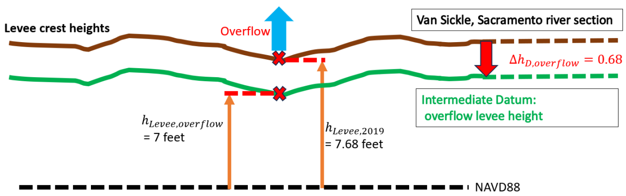


Figure 7.3: Crest height elevation intermediate 'datum'

After the conversion, some sections of the levee are below 6 feet NAVD88, which is below the crest height used to define the probability of failure in chapter 5. These are interior levees and are therefore only at risk of failing if the exterior levees fail first. Table 2.3 also demonstrates that the Sacramento River levees have the largest proportion of their external levees that is below 7 feet. This makes this section the most susceptible to failure during floods.

To simplify the analysis, it was decided to raise all levee sections to a given crest height. Therefore, for a crest height of 7 feet, an investment will have to be performed to raise the interior levees by 1 foot. This investment will not bring a risk reduction, as the interior levees are currently protected by the exterior levees. However, in the future, this might not be the case as the exterior levees are managed by a different reclamation district. Therefore, these interior levees could be faced with a high water level event, if the exterior levees are neglected, which would ruin the investment made to raise the other levees to a crest height larger than 7 feet.

Since Van Sickle Island is located in a tidal wetland chapter 2, the levees can only gain in width to the inside of the island. This is shown visually in Figure 7.1.

Finally, it was decided to only look at crest height increases of 0.2 feet. This is because the increment needs to be technically feasible.

7.2. Costs

One of the costs associated with this plan is the investment cost related to the additional material required to raise the levee. A reference value was calculated from the emergency repair made in August/September of 2023 on two breaches on the Sacramento River which accumulated to 180 meters. The cost was split between dirt and rip rap. For dirt, $155,000 was spent on 5160 meters cubed of material. Therefore, per meter cubed of material the cost was $30 per meter cubed. For rip rap, $220,000 was spent on a 180-meter-long levee with a height of 2.35 meters. This resulted in rip rap costing $520 per meter height per meter length. These calculations are located in Appendix K. For the rest of the chapter, this value was assumed to be representative of the cost of material when raising the levee (excluding permits).

Assumptions will be made to simplify the calculation. Firstly, the crest height of the levee will be given by the unknown variable x, which will depend on the current crest height (given in table 2.3) and the desired final crest height. Secondly, the width of the crest will be assumed to be 10 feet wide [11]. Thirdly, the inner and outer slopes will be assumed to be 3 to 1. Finally, the base of the levee is assumed to be located at 1 foot

NAVD88 [4]. Thus, the crest height will be subtracted from this value. Figure 7.1 demonstrates this concept visually.

Based on the assumptions made above, the cost to repair the levee is given in Equation 7.1. The derivation is given in Appendix K.

$$30(3x^2 + 3x + 6x(y - 0.4)) + 520x \tag{7.1}$$

In equation 7.1, x is the amount the levee is being raised and y is the current crest level of the levee with respect to NAVD88. The equation returns the cost per meter length of the levee in USD.

In addition, the cost of an RGP3 permit needs to be taken into account when raising the levees. The cost is $4 per acre [11], and since the island is 2415 acres, the permit will cost a total of $9,660.

7.2.1. Political, Environmental and Social Investment Costs

Raising the levees will have indirect social costs. The majority of the individuals living/using the island are located in close proximity to the levees. During construction, individuals will need to move their equipment that is located in proximity to the levee and may be unable to access their property. Moreover, raising the levees will not only increase the crest height but will also increase the width of the levee (for stability). Therefore, individuals may need to modify their property to accommodate this management plan. If an individual/group needs to move their property due to the increase in width, it is estimated to cost $30,000 per property. If a gangway to a dock needs to be altered, it is estimated to cost $1,000 per gangway.

As a conservative approach, it will be assumed that all gangways and properties will be affected. Therefore, 10 properties need to be moved, and 10 gangways need to be raised. Therefore, the total social cost will be $310,000. This assumption will have a larger impact on smaller crest heights, as the social cost will contribute to a larger proportion of the total cost, even though some properties will not be affected.

The total investment cost of raising the levee by a certain increment can be seen in Figure 7.4. As can be observed, the investment cost for a crest height increment of 0.0 feet is greater than zero. This is because the interior levees need to be raised, and the social cost is taken into account.

Crest Height Increase From 7 (feet)	Investment Cost (USD)
0.0	1017670.49
0.2	1367078.91
0.4	1720620.74
0.6	2078295.98
0.8	2440104.63
1.0	2806046.69
1.2	3512064.45
1.4	4225967.80
1.6	4947756.74
1.8	5677431.26
2.0	6414991.36

Figure 7.4: Investment costs for raising the levees by a certain increment from 7 feet NAVD88.

There will also be indirect environmental costs. Firstly, extraction of the material can be done in different ways: dredging and mining. Both of these will increase the amount of suspended sediment and wash load in the river. Secondly, when placing material on the levee, it could affect both land and aquatic species in proximity to the levee as their habitat will be occupied/changed. However, no monetary value can easily be placed on these, and will therefore have to be taken into consideration when acting on the plan.

7.3. Benefits

The benefits of this management plan are related to the reduction in risk. Raising the levees affects risk in two way, the probability of failure and the consequences. The risk reduction can be calculated with Equation 7.2.

$$\Delta(D) = (P_{f,old} - Pf_{f,new}) * (C_{old} - C_{new}) \tag{7.2}$$

Within this equation, D denotes risk in USD, p_f denotes the probability of failure, and C denotes the consequences of failure in USD.

7.3.1. Probability of Failure

The probability of failure is dependent on the probability of occurrence of a water level, and the probability of failure given that water level. On one hand, the probability of occurrence depends on the crest height and will be calculated using the extreme value analysis mentioned in chapter 5. On the other hand it is assumed that the probability of failure given the water level is independent of the crest height and will therefore remain constant. The probability of failures for certain crest height increments can be found in Figure 7.5. From this figure, it can be seen that the relationship between crest height increment and the probability of failure is non-linear. This is because of the non-linearity in extreme value distribution.

Crest Height Increase From 7 (feet)	Probability of Failure
0.0	0.284
0.2	0.189
0.4	0.095
0.6	0.052
0.8	0.032
1.0	0.018
1.2	0.010
1.4	0.006
1.6	0.004
1.8	0.002
2.0	0.001

Figure 7.5: Probabilities of failure for raising the levees by a certain increment from 7 feet NAVD88.

7.3.2. Consequences

For this management plan it is assumed that, the economic consequences from Status Quo (chapter 6) still occur. But there is an added consequence cost. That of the construction costs for raising the levee. It is assumed that after flooding the heightened part completely fails and the same amount of money used to construct it needs to spend on repairing it.

The political, environmental, and social consequences are assumed to be the same as mentioned in chapter 6.

7.3.3. Total Risk Reduction

The total risk reduction of plan B for different levee crest heights can be seen in Figure 7.6. As can be seen, the risk reduction for a crest height increase of 0 feet is negative. The reason for this is an investment had to be made to increase the interior levees by 1 foot, but there was no decrease in the probability of failure. Therefore the consequences have increased, creating a larger risk (negative risk reduction).

Crest Height Increase From 7 (feet)	Risk Reduction (USD)
0.0	-20328.112727
0.2	104674.657273
0.4	219686.067273
0.6	272312.347273
0.8	296999.017273
1.0	314156.077273
1.2	323852.077273
1.4	329095.027273
1.6	332085.847273
1.8	333868.707273
2.0	334936.597273

Figure 7.6: Risk reduction for raising the levees by a certain increment from 7 feet NAVD88.

7.4. Net Present Value

The NPV takes into account both the investment cost as well as the yearly cash flow, which includes income, maintenance, other expenses, and risk. To simplify the calculation, it is assumed that the income, maintenance, and other expenses are not affected by the increase in crest height. These values will be equal to the values in chapter 6. Moreover, it will be assumed that the levees are correctly maintained and that a change in boundary conditions for the basin (i.e. sea level rise, increase in snowmelt) will not have an effect on the probability of failure. This means that the risk reduction will remain constant throughout the years.

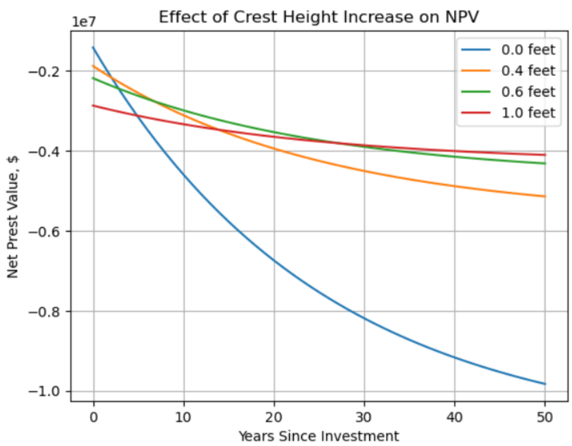


Figure 7.7: Net present value curves for crest height increases which are the most economically favorable after 0, 5, 10, 20, and 50 years.

Figure 7.7 demonstrates the effect of yearly cash flow and investment cost. A larger investment cost can be seen in year 0, it sets the starting value of the NPV calculation. The larger the investment, the larger the initial negative NPV. The effect of the yearly cash flow can be seen in the gradient of a NPV curve. If the gradient is zero, the cash flow breaks even. If the gradient is positive, the cash flow is positive. If the gradient is negative, as is the case for all curves, the cash flow is negative. The reason the gradient decreases with time is caused by the discount rate.

As can be seen in Figure 7.7, no crest height increase is able to give a return on their investment after 50 years. The main reason for this is that the yearly cash flow is negative. Even if the risk were to become zero, the yearly cash flow would still be negative because the sum of the maintenance and other expenses always outweighs the income. Therefore, risk reduction only decreases the yearly loss of money.
Moreover, the NPV associated with certain crest heights demonstrates that the longer the planned lifetime

of the structure is, the more beneficial it is to increase the crest height by a larger amount. This is because the yearly cash flow is less negative. However, there is a larger investment risk associated with a larger crest height. This is because of the assumptions mentioned above. For example, the risk reduction could decrease with time (the probability of failure increases), which would have an effect on the economic optimal crest height for a given life time. Additionally, it is unknown how the crest height will decrease in time. It would therefore be recommended to choose a crest height increase that optimizes the NPV in 5 to 10 years, as the change in probability of failure will not be significant over that time (when compared to 20-50 years). After this lifetime, the levees should be reassessed.

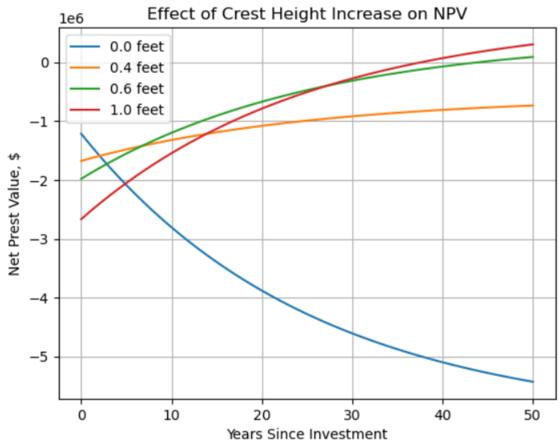


Figure 7.8: Net present value curves for crest height increases which are the most economically favorable after 0, 5, 10, 20, and 50 years, if the general expenses are 55% of their original value.

Interestingly, if there is a change in expenses or income, there will be a change in NPV. However, the crest height increase which maximizes the NPV after a certain period of time remains the same. This is because this will affect all crest increments in a linear way. This wouldn't be the case if the change in income/expense would depend on the crest height increment.
An example is shown in Figure 7.8. If the general expenses do decrease by 55% their original value, two crest height increases are able to have a return on their investment after 50 years. Moreover, one additional crest height increase starts to increase its NPV. This is because the yearly cash flow is now positive every year, which demonstrates that the risk reduction is significant. This also demonstrates that the yearly cash flow needs to be optimized to optimize the NPV.

7.5. Remarks

It is difficult to discuss this management plan without mentioning maintenance. A rigorous maintenance plan will need to be created to ensure that the levee crest heights are at a sufficient height in the years following the initial intervention. Yearly inspections will have to be conducted to ensure that there aren't any severe damages to certain levee sections caused by high water levels.
Moreover, in the long term, the levee crest height will naturally decrease, partly due to settlement. This decrease of height which will increase the probability of failure. The crest height of the levees should therefore be regularly measured to ensure that the desired risk reduction is taking place.

7.6. Summary

As shown in this chapter, the economic optimum for a certain lifetime is independent of income and expenses (if these are constant for all crest heights). However, the NPV associated with a certain crest height is dependent on these two values. Unless the income is increased, or the expenses are decreased, the NPV is negative regardless of the crest height increase. Therefore, the economic optimum only maximizes the NPV, it doesn't cause a return on investment.

If the expenses and income remain constant, it is recommended to raise the levees for a lifetime of 10 years (crest height increase of 0.6 feet). This is because it is the most economically favorable option between years 7 and 25. A crest height increase of this amount also means that the social cost was overestimated, as the majority of houses will remain unaffected by the increase in the width of the levee. Therefore the investment cost should be smaller, further increasing the duration of time the crest height increase is economically favorable. Moreover, the gradient in the NPV is quite weak, which means that a relatively small increase in income/decrease in expense can cause the yearly cash flow to become positive.

It is important to mention that the increase of the crest height by 0.6 feet, will set a target for the probability of failure. However, due to settlement or other mechanisms, the crest height will decrease in time. This means that the probability of failure will always be larger than the predicted value. This will decrease the NPV even further as the yearly cash flow will be more negative.

Finally, this conclusion is based on the extreme value analysis which was formulated in chapters 5. A high level of uncertainty is associated with this as the extreme value curve is unable to correctly quantify the probability of occurrence of the three most extreme events. This therefore propagates to the values used in Figure 7.7. The conclusion of this management plan should therefore be taken with care as an increase in the probability of failure will increase the relative magnitude of risk on the yearly cash flow.

Table 7.1 demonstrates the overview of costs and income related to this management plan.

Table 7.1: Overview of total costs and incomes for Raising Levees assuming a crest height increase of 0.6 feet.

	Costs	
Investment	Raising Levee Social	\$1,768,295.98 \$310,000
	Total	\$2,078,296
		Total investment costs **\$2,078,296**
Risk	Consequences for district:	
	Emergency repairs Material to reraise levee Post flood pumping Other levee damage	\$350,000 \$60,533 \$30,000 \$400,000
	Consequences for land owners:	
	Personal/property damage Gas well damage	\$150,000 \$250,000
	Pf	0.054 per year
	Total (per year)	\$1,245,533 * 0.052 = \$64,768
Maintenance	Cost per year	\$235,000
	Total (per year)	\$235,000
General expenses	Cost per year	\$200,000
	Total (per year)	\$200,000
		Total costs (per year) **\$464,768**
	Income	
General income	Annual land owner fee State reimbursement Emergency funding Contributions	
	Total (per year)	\$400,000
		Total income (per year) **\$400,000**

One time, benefits - costs = **-\$2,078,296**

Annual, benefits - costs = **-\$64,768**

	Now	5 years	10 years	50 years
NPV	-\$2,177,631	-\$2,619,855	-\$2,983,330	-\$4,311,572

Table 7.2: NPV for future years for management plan B

8

Management Plan C: Conversion into Estuarine Wetland

8.1. Concept

Along the entire Bay Area, the state of California has been creating mitigation and conservation banks. These banks are permanently protected lands that are conserved and managed for their natural resource value. Converting the island into such a bank also has a financial upside as per square area bank, so-called credits can be obtained. These credits can be sold to permittees or project developers to compensate for their environmental impacts. These banks can be divided into two types: conservation banks and mitigation banks. A conservation bank protects important habitats for threatened endangered and other special-status species that live, can live, or have lived in that bank. A mitigation bank protects, restores, creates, or enhances wetland habitat. Apart from that it may also serve as a conservation bank. Van Sickle Island could be converted into both a conservation and mitigation bank, by turning it into a managed wetland [12].

8.1.1. Wetland Classification

Wetlands are areas where saturation with water is the dominant factor determining the nature of substrate development. There is a distinction between wetlands and deep water habitats, with the limit being 8.5 feet, a depth that is not reached at Van Sickle Island for the current boundary conditions of the basin. To apply for credits, a further classification of the wetland is done by using the Cowardin system. This classification system uses several criteria to classify the wetland. The first criterion is salinity. For a salinity above 0.5 ppt the wetland qualifies for an estuarine or marine wetland. If the salinity is below 0.5 ppt it qualifies as palustrine, riverine, or lacustrine wetlands [13]. The water around Van Sickle Island is brackish (slightly salty), reaching up to 12 ppt during winter months [14]. Therefore Van Sickle Island could be a marine or estuarine wetland. The difference between these two is that a marine wetland should have little to no obstruction to open ocean interaction, whereas estuarine wetlands can be semi-enclosed by land, and interact with runoff as well. Following this differentiation, Van Sickle Island should be classified as an estuarine wetland [13].

8.2. Costs

8.2.1. Investment Costs

The investment costs to turn Van Sickle into an estuarine wetland, consist of permitting, construction, and reviewing costs.

Permitting and Review Costs

To qualify for credits the state of California needs to review the potential, to assess whether the criteria to be classified as a wetland are met and whether this wetland is a suitable habitat for the desirable species for credits. In Table 8.1 the total application procedure consisting of both permitting and reviewing credits is calculated by summing all contributions up, giving a total cost of $150.000.

Table 8.1: Review fees for conservation and mitigation banks.

Bank Document Name/Phase	Review Fee 1/1/2023 to 12/31/2023
Draft Prospectus (optional)	$1,972.50
Prospectus	$13,150.00 ($11,177.50 if Draft Prospectus was evaluated)
Bank AgreementPackage	$32,874.25
Unsolicited Change	$13,150.00
Bank Amendment Package (initial/simple)	$9,862.50
Bank Amendment Package (complex)	$32,874.25 (or $23,011.75 if deemed complex but initial Bank Amendment Package fee already paid)
Implementation	$78,898.00

Construction Costs

To turn Van Sickle Island into an estuarine wetland, the island needs to be in flooded conditions permanently. To create a proper habitat for native species (which is a necessity to obtain credits) has to be created. Invasive species, such as phragmites and tumbleweeds, have been observed to overgrow native species on Van Sickle Island. Therefore invasive species should be removed from the island, as is currently being done on a neighboring island. The cost of removal of invasive species is estimated at around $1.82 per square meter [15]. With the total area of the island being 2415 acres (= 9,773,158 square meters), the total construction costs would be 18 million dollars.

Once the invasive species have been exterminated, the levee needs to be breached to allow for an unhindered in and outflow of water. This breach should be a controlled breach, as this will ensure a suitable location for the outlet of the estuarine wetland. Assuming just the crew costs of the levee reparations of 2023, these costs will be $ 41,000. The breach will have to be maintained at a certain width in order to ensure bank conditions aren't violated. This is done by implementing sheet piles at both breach ends to prevent erosion (similar to a jetty).

The total cost of the sheet pile is estimated as $ 133,333. This is calculated by assuming the sheet pile wall will be of the same material and have the installation costs as the one placed in 2012. The cost (after accounting for inflation up to the current year) was $800,000 and is 300 feet long, giving an approximate cost per foot of $2,667. With a levee width of approximately 25 feet (see Appendix K) the sheet piles to prevent erosion are estimated to $ 133,333.

Maintenance Costs

Even though the levees might have been breached, maintenance still needs to be done to prevent unwanted breaches and to sustain the correct habitat. Therefore the normal maintenance costs stay the same ($ 235,000). On top of the old maintenance costs, new maintenance costs involve the prevention of inner slope erosion. These extra costs are not expected to be significant, and therefore, the maintenance costs will be assumed to remain the same.

Some new maintenance costs are the maintenance costs of the wetland. To be conservative, these are assumed to be equal to the levee maintenance costs, which gives another $ 235,000

Income Loss
Turning the island into a wetland has to be a unanimous decision over the island. In the worst-case scenario, duck hunting won't be possible anymore when turning the island into a mitigation bank, which could be a problem for multiple landowners. A solution could be the construction of interior levees, which then again would cost extra money, or buying the land owners with the money gained from the credits. The same goes for the gas companies, who would have to stop pumping unless interior levees are constructed. To get the duck-hunting community and gas companies on board, half of the credit value will go to them. Apart from these costs that have to be made, the island also loses some income sources, as the duck hunters and gas company will not pay their annual fees anymore, which will result in an income loss of $ 210.000. Even though the income will disappear, the general expenses will not, adding another $200.000 in costs to the balance.

8.2.2. Risk Reduction
With the breached levees implemented, the levee system becomes rather complicated. Therefore no risk reduction calculation is done. It will be assumed that the yearly expenses will ensure that the levee system does not fail.

8.3. Benefits

8.3.1. Economic Benefits
Via the credit system, a price is put on preservation. The credits per bank can be given for species, wetlands, streams, or combinations of those (called groups). To get an idea of what credits could be relevant for Van Sickle Island, the database of RIBITS is used, which has an overview of all credits granted and sold in the United States. It should be stated that the potential credits presented here should solely be interpreted as hypothetical. To get an actual estimation of which credits Van Sickle could apply for, the process described in Section 8.2 should be followed.

8.3.2. Wetland and Stream Credits
In the district of San Francisco (where Van Sickle is located) a list of potential wetland credits is available on RIBITS. As established in subsection 8.1.1, Van Sickle Island would be classified as an estuarine wetland. Within this classification, Van Sickle Island could apply for either wetland credits or eel grass credits. [16] The value of Eel grass credits is unknown, but wetland credits sell for 50000 to 400000 per credit [17].
To determine the area which could be turned into a wetland, the flooded area has to be determined. This flooded area is assumed to be 80 % of the island, which adds up to a total flooded area of 1932 acres. With this flooded area, Van Sickle Island would be able to sell 1932 credits, resulting in a minimum income of 96.6 million dollars. With half of these credits going to the landowners and gas companies, they would get 48.3 million dollars.

Van Sickle might qualify for tidal stream credits. [18]. These sell for an average of $ 260, adding an extra 502,320 dollars. [17]

8.3.3. Species Credits
To estimate what species credits would apply to Van Sickle, RIBITS has a tool to see what banks have species that could occur near the selected location, based on the species in surrounding banks. For each bank presented by RIBITS, the credits have been summed up, to calculate the total credits given out per species, and how often they occur (how many banks have the credits). After this analysis, the habitat per species has been investigated, and compared to the habitat that would become available, if Van Sickle would turn into a mitigation bank. The analysis is available in Appendix L [19]. If Van Sickle becomes a mitigation bank, this could result in species credits for delta smelt, longfin smelt, coho salmon, steelhead, and other salmonids. These potential credits are in line with the species credits a neighboring island is applying for. [20]. The exact values per credit are confidential, but based on 26 banks, the average NPV is $4,205.90 per acre per species. [21] Using the same area calculation as for the wetland credits, with credits for 4 species, a total NPV of 32.5 million dollars is created.

8.4. Interest

If all these savings are stored in the bank, an annual income can be generated to compensate for the loss of income on behalf of the landowners. Interest rates can go up to 4% [10], generating an annual revenue of 2.51 million dollars. It is important to note that this interest of course goes down if the money received from credits is spent. Therefore, this money should stay frozen in the bank, and only be used in case of emergency.

8.5. Remarks

Something worth mentioning is the situation at one of the neighboring islands. They have gone through the process of converting into a mitigation bank, but have run into a lot of issues with an organization, concerning a shared levee. This has caused their application to stagnate and potentially cause disapproval of their credits altogether. Van Sickle Island also shares a levee with that organization and could therefore run into similar problems. This could result in a lot of invested costs and no revenues. This should be kept in mind when considering this management plan. It should also be noted that for this plan to work, a lot of reserves are needed, as the credits are gained after approval of the wetland. Currently, these reserves are not present. A solution could be to pay for it in credits. Once the credits are gained, these can be converted into a corresponding monetary value. This does however still not solve the first problem.

8.6. Summary

In the tables below all the costs and incomes have been summed up. A distinction is made between an annual balance and a one-time balance.

Table 8.2: Costs and incomes for Management Plan C.

One Time Costs		
Investment Costs	Review costs Compensation of landowners and gas company	$150,000 $48,300,000
Construction Costs	Extermination invasive species Levee breach Construction sheet piles	$18,000,000 $41,000 $133,333
		Total one time costs: **$66,442,150**
One Time Benefits		
Credits Income	Wetland credits Stream Credits Species credits	$96,600,000 $502,000 $32,500,000
		Total one time benefits **$129,602,000**

Annual Costs		
Maintenance Costs	Levee maintenance Wetland maintenance General expense	$235,000 $235,000 $200,000
		Total annual costs: **$670,000**
Annual Benefits		
General Income	Annual landowner fees Contributions	$0 $0
Credits Income	Interest (4%) Contributions	$2,510,000 $0
		Total Annual benefits **$2,510,000**

One time, benefits - costs = **+$63,159,850**

Annual, benefits - costs = **+$1,840,000**

The numbers given in the previous tables are used for the net present value calculation. It should be noted that even though the one-time balance adds up to a positive balance of $63 million, all of this money has to be put into the bank to get the interest needed to pay for the yearly maintenance costs. Therefore, the net present value starts at zero.

Table 8.3: NPV for future years for management plan C

	Now	5 years	10 years	50 years
NPV	$ 0	$ 8,191,353	$ 14,924,048	$ 39,527,220

9

Management Plan D: Abandon the Island

9.1. Concept

Abandoning the island denotes two things:

- No maintenance
- No repairs of failed levees

This will ultimately lead to a state of permanent flooding and the potential disappearance of the island entirely.

9.2. Costs

Abandonment of the island will mean the loss of all properties on the island. The value of all properties, including land combined is estimated by an appraisal done in the year 2019 at $8.750.000. Apart from the loss of properties, the yearly income will also disappear, resulting in an additional income loss of $ 400.000.

Apart from these general losses, salt intrusion significantly influences the costs of abandoning the island. As stated in chapter 2, north of Van Sickle Island, the Montezuma Slough contains salinity gates, to prevent salt intrusion. These gates are owned by the District of Water and Resources and currently have a maintenance cost of $ 1 million per year [22] [23]. Since the gates were constructed in 1970, over the past 50 years, it was assumed that a total of $ 50 million has been spent on maintenance alone, not taking construction costs into account. Studies have shown that the operation of these gates could potentially be harmed, if Van Sickle Island is abandoned. If this operation is harmed, salt intrusion further up the bay could endanger the ecosystem.
In a disastrous scenario, the salt intrusion might reach upstream so far that the California Aquaduct becomes brackish, which would become a multi-million dollar problem. It is therefore essential to emphasize the importance of no further salt intrusion and the role of Van Sickle Island in that. As these costs would not have to be paid by the Reclamation district, they are not taken into account in the economic optimization. However, they are of enormous importance when considering this management plan.

9.3. Benefits

Even though this management plan does not technically create revenue, the annual expenses (like maintenance), will go to $0, which is considered a benefit. Compared to status quo, the island would therefore 'save' $766.800

9.4. Remarks

Abandonment of the Island is a purely hypothetical scenario, but one that needs to be considered in this analysis. Apart from the salt intrusion, when choosing this plan, multiple problems could occur, like resistance from the state, duck clubs, and the District of Water and Resources. Each of these parties has enough influence to prevent this management plan from happening

9.5. Summary

Table 9.1: Overview costs and benefits for management plan D

One time costs		
Property loss	Estimated value	$8,750,000
		Total one time costs: **$8,750,000**
One time benefits		
None		$0
		Total one time benefits **$0.00**

Annual costs		
Loss of income	Estimated value	$400,000
		Total annual costs: **$400,000**
Annual benefits		
Loss of expenses		$766,800
		Total Annual benefits **$766,800**

One time, benefits - costs = **-$8.750.000**

Annual, benefits - costs = **+$366,800**

The net present values for this management plan are shown in Table 9.2. A clear increase in NPV can be seen since annually the benefits are higher than the costs.

	Now	5 years	10 years	50 years
NPV	-$ 8,383,200	-$ 6,750,272	-$ 5,408,123	-$ 503,535

Table 9.2: NPV for future years for management plan D

10 Conclusion

The goal of this project was to find the most economically favorable management plan for the next 50 years. Four management plans have been assessed: status quo, raising the levees, conversion into an estuarine wetland, and abandoning the island. The relevant results are discussed here. Do note that these results only included the economic benefits and consequences.
Due to large uncertainties in the behavior of both the levee and the water level over time, the original 50-year lifetime of the management plan is deemed too ambitious. Therefore the lifetime is reduced to 10 years.

Firstly for the status quo, the annual income is $400,000. The annual cost including the annual risk is $766,800. So there is a negative yearly cash flow of $366,800. This results in a net present value of -$3,341,877 after 10 years.

Secondly for raising the levees by an increment of 0.6 feet, the investment cost is $ 2,078,296, and the yearly cash flow is -$ 64,768. Therefore the net present value in 10 years is -$2,983,330.

Thirdly for conversion into an estuarine wetland, the one-time costs are estimated at $66,442,150. The one-time income is estimated at $129,602,000. This gives a total one-time profit of $ 63,159,850. Since all of this profit is assumed to be put into an investment fund, instead of a one-time profit there will be an annual income of $2,510,000 due to the interest. The annual costs will be $ 670,000. So there is a positive yearly cash flow of $1,840,000. Therefore the net present value in 10 years is $14,924,048.

Finally, for abandoning the island, there is a one-time cost of $8,750,000 because it is assumed the island will flood and all property on it is lost. The annual cost is $400,000 due to a loss of income. The annual benefit is a loss of expenses of $766,800. So there is a 'hypothetical' positive yearly cash flow of $366,800. Therefore the Net Present Value in 10 years is -$5,408,124.

The annual costs of the status quo and raising the levees are highly dependent on annual risk. To determine this risk multiple assumptions and simplifications were made in estimating the consequences of failure and calculating the probability of failure. Small changes in these assumptions can lead to significant changes in the annual risk and thus costs. Changes in costs could lead to different outcomes.

For the estuarine wetland conversion, a significant investment needs to be made. This plan also relies heavily on the support of landowners, neighboring landowners, and state agencies. Moreover, to simplify the management plan, risk was omitted. Therefore this management plan comes with large uncertainties concerning feasibility.

For abandoning the island, the potential consequence costs of salt intrusion in the Bay Area are not taken into account. They are leading to underestimation of the total costs associated with this plan. Apart from this underestimation, potential issues could arise for parties that are affected by this salt intrusion.

Based on the information above, the recommended management plan for a lifetime of 10 years is conversion into an estuarine wetland.

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Appendices

A

Data Acquisition

Types of Data

River Stage/Measured Water Level

General info: River stage

Source:	California Data Exchange Center
Measuring Station:	Mallard Island
Station ID:	MAL
Datum:	(NAVD88)
Period of data:	01/01/2001 to 29/08/2023
Units:	Feet (ft)
Sampling frequency:	Hourly
URL:	`https://cdec.water.ca.gov/dynamicapp/staMeta?station_id=MAL`
Remarks:	See section A, about outliers in this data set

Tidal Water Level Predictions at Mallard

General info: Tide predictions at 9415112
Datum info: (STND): A fixed base elevation at a tide station to which all water level measurements are referred. The datum is unique to each station and is established at a lower elevation than the water is ever expected to reach. It is referenced as the primary benchmark at the station and is held constant regardless of changes to the water level gauge or tide staff. The datum of tabulation is most often at the zero of the first tide staff installed.

Source:	National Oceanic and Atmospheric Administration
Measuring Station:	Mallard Island, Suisun bay, CA
Station ID:	9415112
Datum:	(STND)/Unknown
Period of data:	2000-01-01 to 2025-01-01
Units:	Feet (ft)
Sampling frequency:	Hourly
URL:	`https://tidesandcurrents.noaa.gov/noaatidepredictions.html?id=9415112`
Remarks:	None

Tidal Water Level Predictions at Martinez

General info: Tide predictions at 9415102

Source:	National Oceanic and Atmospheric Administration
Measuring Station:	Martinez-Amorco Pier, CA
Station ID:	9415102
Datum:	(NAVD88)
Period of data:	2000-01-01 to 2025-01-01
Units:	Feet (ft)
Sampling frequency:	Hourly
URL:	`https://tidesandcurrents.noaa.gov/noaatidepredictions.html?id=9415102`
Remarks:	None

Calculated Discharge

General info: DTO is an estimate of net Delta outflow at Chipps Island and is derived by performing a water balance about the boundary of the Sacramento-San Joaquin Delta, taking Chipps Island as the western limit.
Datum info: N.A.

Source:	California Data Exchange Center
Measuring Station:	Delta Outflow Station
Station ID:	DTO
Datum:	N.A.
Period of data:	01/01/1994 to 21/09/2023
Units:	Cubic feet per second (cfs)
Sampling frequency:	Daily
URL:	`https://cdec.water.ca.gov/dynamicapp/staMeta?station_id=DTO`
Remarks:	See section A, about outliers in this data set

Meteorological Observations Pittsburg

General info: Wind speed (knots), Degrees (F), Barometric pressure (mb) measured in the Pittsburg Marina
Datum info: N.A.

Source:	National Oceanic and Atmospheric Administration
Measuring Station:	Pittsburg, Suisun Bay, CA
Station ID:	9415115
Datum:	N.A.
Period of data:	08/06/2011 to 30/09/2023
Units:	Wind speed (knots), Degrees (F), Barometric pressure (mb)
Sampling frequency:	Hourly
URL:	`https://tidesandcurrents.noaa.gov/met.html?id=9415115`
Remarks:	This data set is very limited, it is only available 2011 to 2023

Meteorological Observations San Francisco

General info: Barometric pressure measured in the central bay near San Francisco.
Datum info: N.A.

Source:	National Oceanic and Atmospheric Administration
Measuring Station:	San Francisco, CA
Station ID:	9414290
Datum:	N.A.
Period of data:	08/06/2011 to 30/09/2023
Units:	Barometric pressure (mb)
Sampling frequency:	Hourly
URL:	`https://tidesandcurrents.noaa.gov/met.html?id=9414290`
Remarks:	None

Datum Conversion

NGVD29 to NAVD88

On October 1, 2006, the California Department of Water Resources (DWR), changed the Datum of the measuring stations in the Suisun Marsh from National Geodetic Vertical Datum 1929 (NGVD29) to the North American Vertical Datum 1988 (NAVD88). From that date onward all river stage measurements will be displayed in NAVD88 datum.

The difference between NGVD29 and NAVD88 for **Mallard Island measuring station** is **+2.68 ft**

The formula for datum conversion for the Mallard Island measuring station is given below, h = water level [feet].

$$h_{NAVD88} = h_{NGVD29} + 2.68 \tag{A.1}$$

Deleting Outliers

After visual inspection of both the river stage/measured water level and calculated discharge outliers have been identified.
Outliers are not taken into consideration while performing any analysis.

River stage/ measured water level

For the river stage, all the outliers were found above a value of 9.25 feet.

Calculated Discharge

For the station 'Delta outflow' two outliers have been identified. They occurred on:

- 2012-07-01
- 2013-12-31

B

Wind Setup Data

Relevant measurements for wind setup per wind direction

dir	Angles (deg)	Depths (m)	phi	cos(phi)	Fetches (m)	half	relevant point	x
N	360	4.159567111	114	0.619520613	17457	8728.5	9204	475.5
NNE	22.5	3.382942594	136.5	-0.158607787	16854	8427	4719	3708
NE	45	4.564224485	159	-0.342494779	17262	8631	1342	7289
ENE	67.5	2.758117505	181.5	0.756812347	20128	10064	0	10064
E	90	6.26390805	204	-0.97936069	20190	10095	0	10095
ESE	112.5	4.409654752	226.5	0.953748122	20438	10219	0	10219
SE	135	3.979192554	249	-0.686464631	17044	8522	0	8522
SSE	157.5	4.131828581	271.5	0.245237374	18986	9493	7202	2291
S	180	4.159567111	294	0.258130759	17457	8728.5	8253	475.5
SSW	202.5	3.382942594	316.5	-0.696090952	16854	8427	12135	3708
SW	225	4.564224485	339	0.957668159	17262	8631	15920	7289
WSW	247.5	2.758117505	1.5	0.070737202	20128	10064	20128	10064
W	270	6.26390805	24	0.424179007	20190	10095	20190	10095
WNW	292.5	4.409654752	46.5	-0.811612192	20438	10219	20438	10219
NW	315	3.979192554	69	0.99339038	17044	8522	17044	8522
NNW	337.5	4.131828581	91.5	-0.923452664	18986	9493	11784	2291

C

Failure Mechanism Analysis

As mentioned, levees can fail due to multiple failure mechanisms. To assess the relevant failure mechanisms per section, Table C.1 has been created.

Table C.1: Summary of relevant failure mechanisms per section.

	Montezuma Slough	Sacramento River	Spoonbill Creek	Honker Bay	Van Sickle North
Overflow	Y	Y	Y	Y	N
Wave overtopping	N	N	N	N	N
Sliding inner slope & Settlement	Y	Y	Y	Y	Y
Shearing	N	N	N	N	N
Sliding outer slope	N	N	N	N	N
Micro-instability	Y	Y	Y	Y	Y
Piping	Y	Y	Y	Y	Y
Erosion outer slope & first bank	Y	N	N	Y	N

As can be seen in the table the relevant failure mechanisms for the island, are overflow and overtopping, settlements/macro-instability, micro-instability, piping, and erosion of the outer slope. These failure mechanisms are reviewed further in the next few paragraphs. The theory behind each failure mechanism can be found in Flood Defences by S.N. Jonkman, R.E. Jorissen, T. Schweckendiek, and J.P. van der Bos.

Overflow and Overtopping

Overflow of the levee occurs when the still water level exceeds the crest height of the levee. Overtopping occurs when only the waves exceed the crest height. Flood events in the past have shown that overflow has been the main contributor to the levee breaches. Overtopping is less significant, as no significant fetches are measured in the Suisun Marsh.

Macro Stability & Settlement

All levees on Van Sickle Island are constructed on top of a peat layer. Since peat is a soft soil, significant settlements may occur, possibly causing the crest height of the levees to be inadequate. There are two types of settlements: vertical settlement due to loading and rotational settlement where the inner slope slowly rotates due to a macro-stability issue (Figure C.1). Due to the lack of data, it is challenging to determine the settlement type and quantify the settlement. Therefore this failure mechanism is not reviewed further in this report.

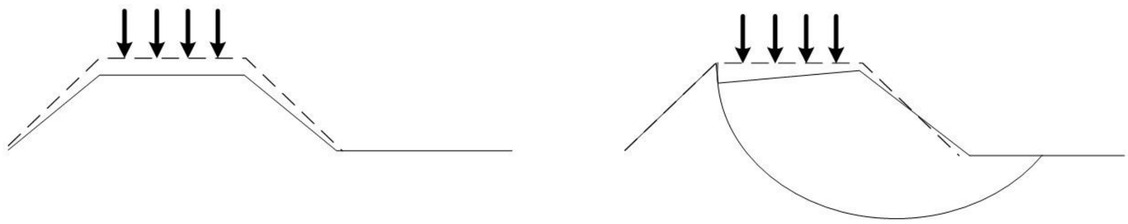


Figure C.1: Vertical settlement (left) and rotational sliding of the inner slope (right).

Micro-instability & Piping

Micro-instability occurs when seepage reaches the inner slope of the levee. This seepage could eventually lead to piping or inner slope erosion. Field observations have shown several signs of leakage in the levees, but as overflow is a more dominant failure mechanism, piping is not reviewed in this report.

Erosion of the Outer Slope & First Bank

Currents and waves can erode the outer slope of the levees due to loosening the levee material and transporting it elsewhere. In the case of Van Sickle Island, erosion intensifies due to the large amount and size of debris in the waves and currents. For example, the erosion spot in Honker Bay (see Figure 2.6) is caused by large tree trunks repetitively impacting the levee. Outer slope erosion can be reduced by placing riprap. Since riprap is not present along the entire levee stretch, outer slope erosion is still a relevant failure mechanism but is secondary to the impact of overflow and settlements.

Conclusion

The main focus of this risk analysis will be on overflow, as this is the most relevant failure mechanism. Settlements can not be taken into account, due to lack of data, and piping is deemed secondary. Outer slope erosion is easier to review, and therefore also taken into account in the report.

D

Case Study: Outer Slope Erosion

The problem of outer slope erosion is most urgent in honker bay, as this section has big wave (and debris) impacts, and no outer slope protection like riprap to disperse these impacts without damaging them. The easiest solution would be to place slope protection that can withstand these impacts, but legislation states that no material can be imported to the weak spots with the current permit. This case study reviews one of the eroded parts of the levees at Honker Bay, and is used to propose several potential solutions, using the material already in place.

The case study is done on the location pinpointed with a red circle in Figure 2.6. At the moment some mitigation measures are already in place, in the form of vertical wooden logs to prevent the impact of large pieces of debris.

The first solution presented is an extension of the vertical logs already in place. The concept is also used at beaches to decrease erosion. By placing extra logs in the soil, a similar effect could be created, using the big debris logs that are already at the site anyway. As legislation states that building into the bay is not allowed the logs will be placed along the levee instead of perpendicular (as is normal at beaches).

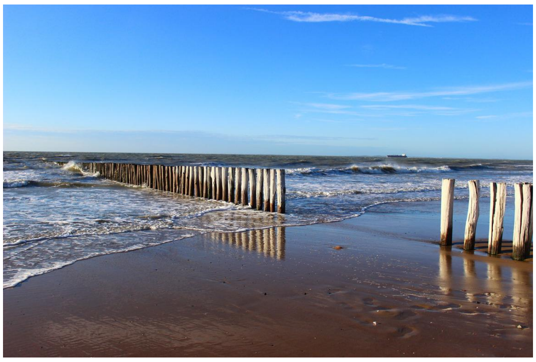
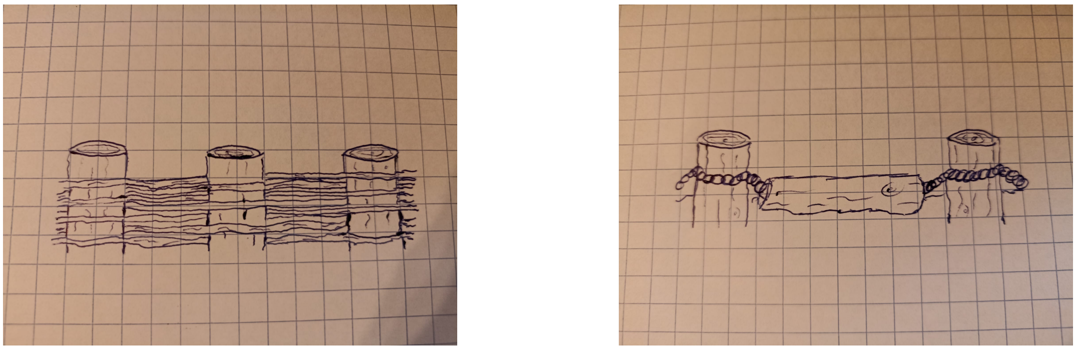


Figure D.1: Erosion prevention measures at the Dutch beach

In case the first solution does not suffice, a second solution can be implemented to extend the log construction, decreasing impact even further. By braiding bamboo stakes in between the logs, the wave impact becomes even lower, yet still permeable to the water, so that no pressure difference builds up between the inside and outside of the newly created barrier. depending on how tightly braided the bamboo is, it might even be able to keep some of the sediment eroding from flooding away into the bay, so that no new material has to be imported that often. Downside of this solution is that the bamboo might be fragile when it comes to debris impact, making it a less durable solution.

Another extension to the first solution could be the use of floating debris instead of bamboo. By chaining the floating logs to the vertical logs, wave impacts are dampened, without fragility problems or pressure buildup behind the barrier. The downside of this solution is that it does not stop the eroded material from floating into the bay. Both extensions of solution one are sketched in Figure D.2.



(a) Bamboo solution

(b) Floating debris solution

Figure D.2: Potential extensions to the vertical logs

So to prevent outer slope erosion, without importing new material, the debris that is causing the damage can be used to prevent it. If not sufficient, this solution can be extended with the solutions in Figure D.2.

E

Important Dates and Flood Waves

Important Dates Floods

Van Sickle Island was flooded 5 times since 2006. The important dates concerning these floods are listed below.

- 2006-01-01 Levee breach
- 2017-01-11 Levee breach at Sacramento River section
- 2017-01-27 Repair levee breach at Sacramento River section
- 2017-02-15 Levee breach: repaired section at Sacramento River fails again due to large dam releases in the Sierra
- 2017-05-25 Levee breach at Concord Club
- 2023-01-06 Levee breath at the Sacramento River section
- 2023-03-04 2^{nd} breach, possibly caused by inner slope erosion due to impact waves

Data analysis opens opportunities to further understand the origins of these floods. The floods are caused by water levels higher than the crest height of the levels and are caused by a combination of high discharges, low air pressures, and high wind set-ups. The discharge and water level data from 2001 onwards were analyzed to understand the flood waves of these flood events. By means of an extreme value analysis using the peak-over-threshold method and a decluttering time of 30 days, potential flood dates were determined. For the floods listed above, the highest water level in the 30-day frame was chosen as the peak of the flood wave. It can be concluded, that on average the water level peaks a few days before the levee breaches.

- 2005-12-31: flood wave of 2006-01-01 flood
- 2017-01-12: flood wave of 2017-01-11 flood
- 2017-02-09: flood wave of 2017-02-15 flood
- 2017-05-25: flood wave of 2017-05-25 flood
- 2023-01-05: flood wave of 2023-01-06 flood

Important Dates Near-Misses

Apart from the floods, there also have been high water events where the levee barely held up, these will be referred to as near-misses. The Reclamation District 1607, has documented the well-being of Van Sickle Island in his field notes since 2010. All possible flood dates, obtained by an extreme value analysis, were looked up in the field notes and it was concluded that the flood waves of 2021-06-25 and 2022-07-13 are certain near misses since respectively 1/3 of the levee eroded and the placed aquadam was at the maximum limit. Other dates since 2010 were short of field notes and there were no field notes of the events before 2010, so it was determined on similar trends in the water level height and water level if a certain date qualified for being a near-miss. Regarding the water level, it was concluded, based on the field notes, that all water levels of 7.05 feet and below do not cause any substantial damage, and are thus no near-misses. Based on the flood waves of the floods and the near-misses of 2021-06-25 and 2022-07-13, it was observed that all had a clear high water level peak, after which the water level decreased to a constant relatively high water level (between 6 to 7 feet). Furthermore, the water level was high (>6.5 feet) for a period of at least about a week and the discharge strongly increased after the water level peak. It was assumed that all near-misses among the dates missing representation in the field notes followed similar trends. The following dates were identified as near-misses:

- 2005-01-08 identified based on a similar trend as floods and reported near-misses
- 2010-12-19 identified based on a similar trend as floods and reported near-misses
- 2011-03-23 identified based on a similar trend as floods and reported near-misses
- 2019-02-14 identified based on a similar trend as floods and reported near-misses
- 2021-06-25 identified based on field notes, which state that 1/3 of the levee eroded
- 2022-07-13 identified based on field notes, which state that the placed aquadam was at the maximum limit
- 2023-06-06 identified based on a similar trend as floods and reported near-misses

Flood Waves

The development of the water level and discharge during the flood waves of floods and near-misses is shown in E.1. Concerning the water level, the flood waves follow, in general, the same trend for floods and near-misses: the water level sharply increases and peaks, after which it decreases to a relatively high constant water level of around 6 to 7 feet. However, the water level in floods peaks higher than in near-misses. Concerning the discharge, it can be concluded that discharge peaks a few days later than the corresponding water level and increases for both floods and near-misses, but the discharge during floods is significantly higher. It must be remarked that it is rather odd that the discharge increases and peaks after the water level.

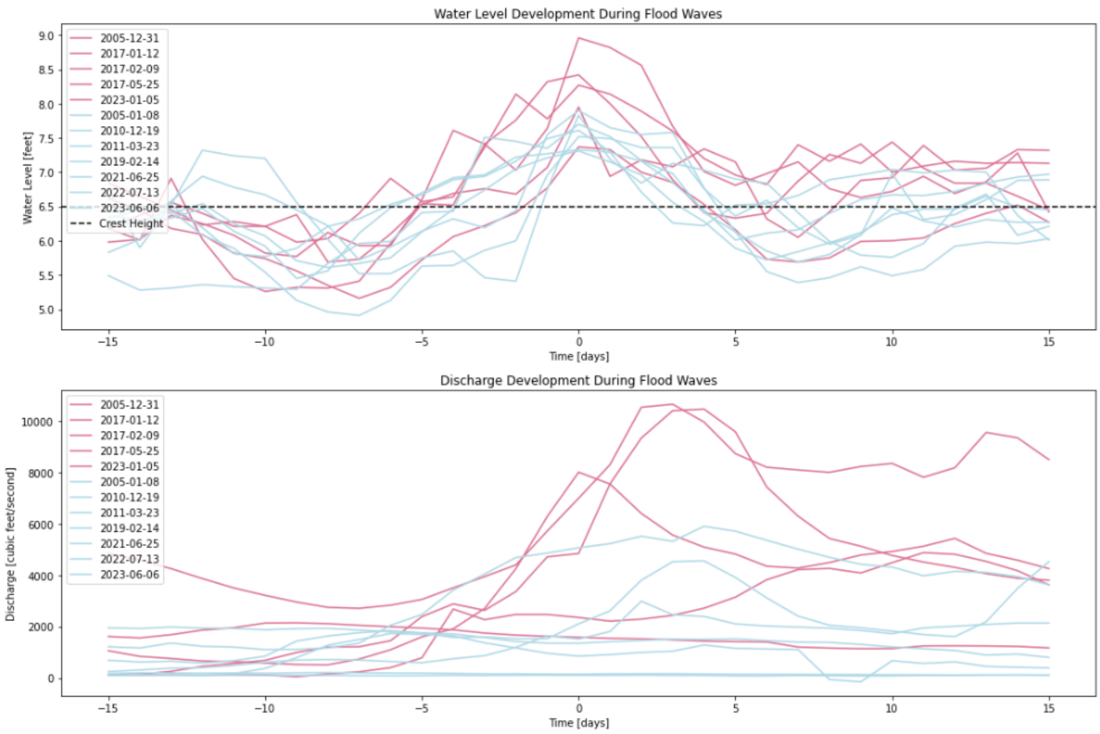


Figure E.1: Flood wave development during floods and near-misses

F

Tabulated Overview Data, Events and Extremes

In this appendix, tabulated overviews are shown of all the daily data prior to a flood event. Apart from the values of data, the P-rank of these values is calculated. The P-rank shows whether a value is considered statistically high or low. If for instance, the P rank is 95 this means that 95% of all recorded values for this variable are equal or lower than the value. If you reverse it, this means that this value is in the highest 5% of all of the recorded data.

The coloring format is defined in the following way:

1. If a value is higher than the 95th percentile the cell will be colored **green**
2. If a value is below the 25th percentile the cell will be colored **red**
3. If a breach has occurred on a date, the row is colored **yellow**

Flood of 2006

The events in 2006:

1. 2006-01-01 Breach occurred

Date	Discharge (cms)	P-Rank Discharge	Water level (feet)	P-Rank Water Level	HH-Tidal	P-Rank HH-Tidal	LH-Tidal	P-Rank LH-Tidal
2005-12-01 00:00:00	226.800000	48.801000	7.010000	97.335000	8.150000	94.269000	3.100000	1.926000
2005-12-02 00:00:00	333.400000	64.502000	6.500000	86.019000	8.180000	95.154000	2.600000	0.133000
2005-12-03 00:00:00	456.300000	73.661000	6.140000	65.907000	8.140000	93.882000	6.210000	61.485000
2005-12-04 00:00:00	618.300000	79.428000	5.850000	46.341000	8.030000	88.769000	6.280000	65.386000
2005-12-05 00:00:00	687.900000	81.161000	5.580000	28.205000	7.790000	70.754000	6.360000	69.409000
2005-12-06 00:00:00	578.900000	78.083000	5.350000	16.465000	7.440000	32.396000	2.800000	0.485000
2005-12-07 00:00:00	340.100000	65.653000	4.920000	3.392000	7.090000	8.772000	3.290000	3.429000
2005-12-08 00:00:00	255.900000	53.659000	4.990000	4.846000	6.970000	4.483000	3.970000	10.528000
2005-12-09 00:00:00	205.000000	43.530000	5.130000	8.251000	7.210000	14.732000	4.740000	22.207000
2005-12-10 00:00:00	181.400000	37.497000	5.440000	20.717000	7.500000	38.430000	5.510000	40.174000
2005-12-11 00:00:00	151.000000	30.373000	5.800000	42.610000	7.740000	65.677000	6.120000	57.197000
2005-12-12 00:00:00	139.800000	27.017000	6.000000	56.857000	7.840000	75.309000	5.580000	41.980000
2005-12-13 00:00:00	152.600000	31.003000	6.090000	63.036000	7.950000	83.887000	4.730000	22.026000
2005-12-14 00:00:00	153.100000	31.136000	6.200000	70.305000	7.890000	79.634000	3.910000	9.644000
2005-12-15 00:00:00	143.500000	28.398000	6.230000	72.207000	7.870000	77.914000	3.270000	3.198000
2005-12-16 00:00:00	140.600000	27.453000	6.180000	68.779000	7.750000	66.865000	2.920000	0.921000
2005-12-17 00:00:00	135.700000	25.030000	6.010000	57.451000	7.670000	57.972000	6.510000	78.083000
2005-12-18 00:00:00	255.700000	53.622000	6.910000	96.232000	7.570000	46.014000	6.590000	81.585000
2005-12-19 00:00:00	451.400000	73.467000	6.020000	58.032000	7.400000	29.004000	6.640000	83.450000
2005-12-20 00:00:00	559.500000	77.514000	5.450000	21.262000	7.170000	12.515000	6.650000	83.790000
2005-12-21 00:00:00	685.700000	81.052000	5.260000	12.430000	6.890000	2.459000	6.700000	85.789000
2005-12-22 00:00:00	990.100000	86.394000	5.320000	15.277000	6.810000	1.357000	3.650000	6.821000
2005-12-23 00:00:00	1204.300000	88.793000	5.310000	14.696000	6.920000	3.114000	4.010000	11.049000
2005-12-24 00:00:00	1215.300000	88.951000	5.410000	19.409000	7.160000	11.958000	4.520000	18.270000
2005-12-25 00:00:00	1449.500000	91.071000	5.960000	53.792000	7.390000	28.132000	5.130000	30.858000
2005-12-26 00:00:00	2388.900000	95.493000	6.540000	87.558000	7.660000	56.676000	5.680000	44.088000
2005-12-27 00:00:00	2885.500000	96.596000	6.520000	86.794000	7.890000	79.634000	5.400000	37.606000
2005-12-28 00:00:00	2635.100000	96.075000	7.410000	99.128000	8.120000	93.167000	4.740000	22.207000
2005-12-29 00:00:00	3367.900000	97.298000	7.030000	97.432000	8.240000	96.705000	3.980000	10.625000
2005-12-30 00:00:00	4720.200000	98.934000	7.640000	99.382000	8.300000	98.062000	3.260000	3.065000
2005-12-31 00:00:00	4850.700000	99.055000	8.960000	99.624000	8.310000	98.231000	2.690000	0.254000
2006-01-01 00:00:00	7561.600000	99.746000	8.820000	99.612000	8.340000	98.643000	6.540000	79.622000
2006-01-02 00:00:00	9353.200000	99.903000	8.560000	99.600000	8.200000	95.808000	6.690000	85.474000
2006-01-03 00:00:00	10419.300000	99.964000	7.670000	99.431000	7.930000	82.554000	6.820000	89.484000

Figure F.1: Tabulated data, flood 2006

Floods of 2017

The events in 2017:

1. 2017-01-11 Breach 1, section Sacramento river
2. 2017-01-27 Repair breach 1
3. 2017-02-15 Breach 1 failed again, section Sacramento river
4. 2017-05-25 Breach 2, failed, section Concord club

Date	Discharge (cms)	P-Rank Discharge	Water level (feet)	P-Rank Water Level	HH-Tidal	P-Rank HH-Tidal	LH-Tidal	P-Rank LH-Tidal
2016-12-15 00:00:00	943.600000	85.801000	7.130000	98.158000	8.020000	88.200000	2.650000	0.218000
2016-12-16 00:00:00	1247.800000	89.278000	6.960000	96.826000	7.860000	77.054000	6.810000	89.266000
2016-12-17 00:00:00	2006.200000	94.330000	6.090000	63.036000	7.610000	50.630000	6.880000	90.974000
2016-12-18 00:00:00	2584.300000	95.941000	5.520000	24.861000	7.280000	19.276000	6.960000	93.034000
2016-12-19 00:00:00	3016.800000	96.862000	5.360000	17.022000	7.100000	9.171000	3.300000	3.501000
2016-12-20 00:00:00	2832.600000	96.462000	5.470000	22.110000	7.240000	16.477000	3.840000	8.905000
2016-12-21 00:00:00	2222.800000	95.190000	5.840000	45.505000	7.360000	25.442000	4.450000	17.252000
2016-12-22 00:00:00	1859.900000	93.470000	6.050000	60.116000	7.450000	33.620000	5.100000	29.961000
2016-12-23 00:00:00	1650.200000	92.416000	6.360000	79.780000	7.480000	36.394000	5.740000	45.687000
2016-12-24 00:00:00	1505.500000	91.519000	6.360000	79.780000	7.550000	43.930000	6.120000	57.197000
2016-12-25 00:00:00	1300.200000	89.702000	6.060000	60.855000	7.530000	41.665000	5.570000	41.786000
2016-12-26 00:00:00	1197.200000	88.733000	5.850000	46.341000	7.580000	47.468000	4.920000	26.000000
2016-12-27 00:00:00	1149.800000	88.151000	5.950000	53.005000	7.610000	50.630000	4.320000	15.120000
2016-12-28 00:00:00	1059.400000	87.243000	5.980000	55.113000	7.710000	62.491000	3.810000	8.578000
2016-12-29 00:00:00	841.600000	84.129000	6.020000	58.032000	7.770000	68.742000	3.420000	4.483000
2016-12-30 00:00:00	755.900000	82.396000	6.370000	80.349000	7.850000	76.169000	3.170000	2.447000
2016-12-31 00:00:00	659.800000	80.470000	6.250000	73.916000	7.820000	73.201000	6.670000	84.662000
2017-01-01 00:00:00	632.500000	79.767000	6.080000	62.297000	7.730000	64.708000	6.690000	85.474000
2017-01-02 00:00:00	580.000000	78.120000	5.820000	43.870000	7.520000	40.732000	6.810000	89.266000
2017-01-03 00:00:00	522.800000	76.242000	5.770000	40.562000	7.210000	14.732000	6.910000	91.834000
2017-01-04 00:00:00	510.800000	75.830000	6.120000	64.817000	7.090000	8.772000	6.840000	90.041000
2017-01-05 00:00:00	734.000000	81.984000	5.930000	51.745000	7.290000	20.039000	3.930000	9.874000
2017-01-06 00:00:00	1097.600000	87.570000	5.930000	51.745000	7.480000	36.394000	4.630000	20.342000
2017-01-07 00:00:00	1602.100000	92.101000	6.600000	89.593000	7.670000	57.972000	5.460000	38.951000
2017-01-08 00:00:00	1914.000000	93.736000	7.610000	99.370000	7.910000	81.221000	6.180000	60.092000
2017-01-09 00:00:00	2705.400000	96.172000	7.410000	99.128000	8.090000	91.895000	5.720000	45.142000
2017-01-10 00:00:00	4262.400000	98.510000	7.760000	99.455000	8.190000	95.493000	4.920000	26.000000
2017-01-11 00:00:00	6296.100000	99.588000	8.320000	99.576000	8.220000	96.256000	4.090000	12.079000
2017-01-12 00:00:00	8019.000000	99.794000	8.420000	99.588000	8.170000	94.863000	3.420000	4.483000
2017-01-13 00:00:00	7559.900000	99.733000	8.000000	99.528000	8.090000	91.895000	3.040000	1.599000
2017-01-14 00:00:00	6411.800000	99.637000	7.520000	99.297000	7.950000	83.887000	7.070000	95.154000
2017-01-15 00:00:00	5569.000000	99.455000	6.890000	95.929000	7.710000	62.491000	7.200000	97.044000
2017-01-16 00:00:00	5098.900000	99.237000	6.410000	82.299000	7.400000	29.004000	7.280000	98.086000
2017-01-17 00:00:00	4828.800000	99.031000	6.330000	78.386000	7.310000	21.359000	7.050000	94.815000

Figure F.2: Tabulated data, flood 2017 number 1

2017-01-17 00:00:00	4828.800000	99.031000	6.330000	78.386000	7.310000	21.359000	7.050000	94.815000
2017-01-18 00:00:00	4354.000000	98.583000	6.400000	81.815000	7.350000	24.643000	3.840000	8.905000
2017-01-19 00:00:00	4283.700000	98.534000	6.870000	95.578000	7.410000	29.816000	4.330000	15.362000
2017-01-20 00:00:00	4498.400000	98.716000	7.260000	98.692000	7.450000	33.620000	4.910000	25.745000
2017-01-21 00:00:00	4791.200000	99.007000	7.130000	98.158000	7.460000	34.541000	5.550000	41.204000
2017-01-22 00:00:00	4928.300000	99.128000	7.440000	99.188000	7.540000	42.864000	6.140000	58.178000
2017-01-23 00:00:00	5133.300000	99.261000	7.060000	97.662000	7.600000	49.612000	5.960000	51.054000
2017-01-24 00:00:00	5440.200000	99.406000	6.840000	95.166000	7.670000	57.972000	5.400000	37.606000
2017-01-25 00:00:00	4857.800000	99.067000	6.840000	95.166000	7.780000	69.687000	4.850000	24.400000
2017-01-26 00:00:00	4589.300000	98.825000	6.650000	91.083000	7.830000	74.364000	4.370000	16.029000
2017-01-27 00:00:00	4261.100000	98.498000	6.450000	84.032000	7.930000	82.554000	3.950000	10.201000
2017-01-28 00:00:00	3872.000000	98.013000	6.230000	72.207000	7.970000	85.316000	3.610000	6.288000
2017-01-29 00:00:00	3507.700000	97.516000	6.280000	75.624000	7.910000	81.221000	3.410000	4.325000
2017-01-30 00:00:00	3219.300000	97.189000	6.210000	71.008000	7.740000	65.677000	7.060000	94.948000
2017-01-31 00:00:00	2958.100000	96.741000	5.980000	55.113000	7.510000	39.641000	7.220000	97.323000
2017-02-01 00:00:00	2751.600000	96.317000	6.030000	58.675000	7.340000	23.916000	7.170000	96.583000
2017-02-02 00:00:00	2713.800000	96.196000	6.400000	81.815000	7.500000	38.430000	6.810000	89.266000
2017-02-03 00:00:00	2835.300000	96.474000	6.910000	96.232000	7.620000	52.229000	4.160000	12.988000
2017-02-04 00:00:00	3059.100000	96.959000	6.570000	88.636000	7.730000	64.708000	4.770000	22.825000
2017-02-05 00:00:00	3511.500000	97.528000	6.640000	90.695000	7.830000	74.364000	5.540000	40.877000
2017-02-06 00:00:00	3949.400000	98.146000	7.360000	99.031000	7.930000	82.554000	6.270000	64.902000
2017-02-07 00:00:00	4406.500000	98.607000	8.140000	99.552000	8.030000	88.769000	6.070000	55.028000
2017-02-08 00:00:00	5727.500000	99.515000	7.780000	99.467000	8.080000	91.398000	5.400000	37.606000
2017-02-09 00:00:00	7005.800000	99.697000	8.270000	99.564000	8.080000	91.398000	4.710000	21.699000
2017-02-10 00:00:00	8312.800000	99.855000	8.140000	99.552000	8.020000	88.200000	4.130000	12.576000
2017-02-11 00:00:00	10551.500000	99.988000	7.890000	99.491000	7.950000	83.887000	3.740000	7.802000
2017-02-12 00:00:00	10669.800000	100.000000	7.600000	99.346000	7.810000	72.535000	3.570000	5.888000
2017-02-13 00:00:00	9975.800000	99.952000	7.200000	98.534000	7.580000	47.468000	7.350000	98.776000
2017-02-14 00:00:00	8745.100000	99.891000	6.960000	96.826000	7.420000	30.834000	7.290000	98.255000
2017-02-15 00:00:00	8212.800000	99.830000	6.820000	94.730000	7.390000	28.132000	6.970000	93.312000

Figure F.3: Tabulated data, flood 2017 number 2

	Discharge (cms)	P-Rank Discharge	Water level (feet)	P-Rank Water Level	HH-Tidal	P-Rank HH-Tidal	LH-Tidal	P-Rank LH-Tidal
Date								
2017-05-01 00:00:00	3383.700000	97.310000	6.220000	71.384000	7.530000	41.665000	4.810000	23.589000
2017-05-02 00:00:00	3119.200000	97.020000	6.030000	58.675000	7.210000	14.732000	5.350000	36.382000
2017-05-03 00:00:00	2854.400000	96.499000	5.810000	43.276000	7.040000	6.857000	6.050000	54.095000
2017-05-04 00:00:00	2507.100000	95.857000	6.070000	61.546000	7.120000	10.189000	6.750000	87.533000
2017-05-05 00:00:00	2342.000000	95.408000	6.120000	64.817000	7.240000	16.477000	6.740000	87.134000
2017-05-06 00:00:00	2220.600000	95.178000	6.150000	66.610000	7.370000	26.205000	6.440000	73.964000
2017-05-07 00:00:00	2090.900000	94.742000	6.030000	58.675000	7.520000	40.732000	6.040000	53.659000
2017-05-08 00:00:00	1937.800000	93.870000	6.280000	75.624000	7.520000	40.732000	5.700000	44.609000
2017-05-09 00:00:00	1744.900000	92.803000	6.410000	82.299000	7.480000	36.394000	5.410000	37.824000
2017-05-10 00:00:00	1614.100000	92.222000	6.620000	90.126000	7.400000	29.004000	5.170000	31.827000
2017-05-11 00:00:00	1560.300000	91.895000	6.450000	84.032000	7.400000	29.004000	4.990000	27.453000
2017-05-12 00:00:00	1685.300000	92.549000	6.180000	68.779000	7.450000	33.620000	4.910000	25.745000
2017-05-13 00:00:00	1865.400000	93.494000	6.090000	63.036000	7.480000	36.394000	6.280000	65.386000
2017-05-14 00:00:00	1957.400000	94.039000	5.820000	43.870000	7.560000	45.045000	6.290000	65.992000
2017-05-15 00:00:00	2139.000000	94.924000	5.740000	38.309000	7.500000	38.430000	6.320000	67.313000
2017-05-16 00:00:00	2147.500000	94.984000	5.560000	27.247000	7.320000	22.171000	6.400000	71.505000
2017-05-17 00:00:00	2114.100000	94.827000	5.350000	16.465000	7.100000	9.171000	5.330000	35.849000
2017-05-18 00:00:00	2050.200000	94.548000	5.160000	9.244000	6.840000	1.623000	5.750000	45.966000
2017-05-19 00:00:00	1995.600000	94.282000	5.320000	15.277000	6.930000	3.392000	6.270000	64.902000
2017-05-20 00:00:00	1942.300000	93.954000	5.720000	37.024000	7.060000	7.584000	6.500000	77.647000
2017-05-21 00:00:00	1874.700000	93.555000	6.060000	60.855000	7.130000	10.613000	6.450000	74.570000
2017-05-22 00:00:00	1748.400000	92.852000	6.220000	71.384000	7.360000	25.442000	6.460000	75.224000
2017-05-23 00:00:00	1668.500000	92.513000	6.410000	82.299000	7.590000	48.643000	6.470000	75.890000
2017-05-24 00:00:00	1623.700000	92.283000	6.770000	93.761000	7.780000	69.687000	6.010000	52.593000
2017-05-25 00:00:00	1583.600000	91.992000	7.370000	99.055000	7.970000	85.316000	5.340000	36.079000
2017-05-26 00:00:00	1542.100000	91.737000	7.330000	98.946000	8.040000	89.278000	4.810000	23.589000
2017-05-27 00:00:00	1522.800000	91.616000	7.000000	97.286000	8.070000	91.010000	6.580000	81.258000
2017-05-28 00:00:00	1482.900000	91.386000	6.850000	95.348000	8.010000	87.654000	6.650000	83.790000
2017-05-29 00:00:00	1436.100000	90.901000	6.530000	87.182000	7.810000	72.535000	6.740000	87.134000
2017-05-30 00:00:00	1414.900000	90.732000	6.160000	67.458000	7.500000	38.430000	4.760000	22.644000
2017-05-31 00:00:00	1403.300000	90.586000	5.730000	37.739000	7.150000	11.449000	5.220000	33.099000

Figure F.4: Tabulated data, flood 2017 number 3

Floods of 2023

The events in 2023:

1. 2023-01-06 Breach 1, section Sacramento river
2. 2023-03-04 Breach 2, section Sacramento river

Date	Discharge (cms)	P-Rank Discharge	Water level (feet)	P-Rank Water Level	HH-Tidal	P-Rank HH-Tidal	LH-Tidal	P-Rank LH-Tidal
2022-12-15 00:00:00	495.200000	75.248000	4.810000	1.854000	6.710000	0.363000	3.630000	6.506000
2022-12-16 00:00:00	197.700000	41.350000	4.900000	3.065000	6.940000	3.732000	4.070000	11.837000
2022-12-17 00:00:00	144.500000	28.616000	5.280000	13.194000	7.140000	10.952000	4.700000	21.577000
2022-12-18 00:00:00	118.400000	18.052000	5.600000	29.307000	7.450000	33.620000	5.390000	37.364000
2022-12-19 00:00:00	142.000000	27.974000	5.520000	24.861000	7.690000	60.116000	5.890000	49.297000
2022-12-20 00:00:00	132.700000	23.916000	5.850000	46.341000	7.990000	86.516000	5.300000	35.098000
2022-12-21 00:00:00	123.200000	20.184000	6.170000	68.185000	8.170000	94.863000	4.470000	17.470000
2022-12-22 00:00:00	113.300000	16.029000	6.370000	80.349000	8.300000	98.062000	3.590000	6.155000
2022-12-23 00:00:00	132.400000	23.770000	6.560000	88.357000	8.380000	99.297000	2.860000	0.606000
2022-12-24 00:00:00	107.600000	13.315000	6.400000	81.815000	8.360000	99.007000	2.480000	0.024000
2022-12-25 00:00:00	130.600000	23.128000	6.210000	71.008000	8.230000	96.535000	6.560000	80.397000
2022-12-26 00:00:00	115.100000	16.683000	6.210000	71.008000	7.990000	86.516000	6.660000	84.214000
2022-12-27 00:00:00	44.900000	1.030000	6.380000	80.640000	7.660000	56.676000	6.760000	87.824000
2022-12-28 00:00:00	154.000000	31.318000	5.690000	35.292000	7.250000	17.325000	3.130000	2.084000
2022-12-29 00:00:00	236.500000	50.618000	5.730000	37.739000	7.200000	14.163000	3.730000	7.657000
2022-12-30 00:00:00	401.100000	70.778000	6.100000	63.448000	7.430000	31.718000	4.430000	16.925000
2022-12-31 00:00:00	783.200000	83.026000	6.530000	87.182000	7.610000	50.630000	5.160000	31.548000
2023-01-01 00:00:00	2683.300000	96.135000	6.690000	92.016000	7.790000	70.754000	5.770000	46.402000
2023-01-02 00:00:00	2269.700000	95.251000	6.760000	93.506000	7.850000	76.169000	6.010000	52.593000
2023-01-03 00:00:00	2474.900000	95.760000	6.680000	91.798000	7.820000	73.201000	5.350000	36.382000
2023-01-04 00:00:00	2473.900000	95.748000	7.070000	97.686000	7.790000	70.754000	4.600000	19.796000
2023-01-05 00:00:00	2357.200000	95.420000	7.950000	99.515000	7.700000	61.425000	3.950000	10.201000
2023-01-06 00:00:00	2212.700000	95.154000	6.940000	96.596000	7.690000	60.116000	3.510000	5.197000
2023-01-07 00:00:00	2291.600000	95.323000	7.180000	98.449000	7.650000	55.682000	3.280000	3.320000
2023-01-08 00:00:00	2439.000000	95.614000	7.080000	97.783000	7.680000	58.953000	3.230000	2.811000
2023-01-09 00:00:00	2717.800000	96.208000	7.340000	98.982000	7.640000	54.737000	6.730000	86.806000
2023-01-10 00:00:00	3147.100000	97.068000	7.150000	98.280000	7.490000	37.255000	6.730000	86.806000
2023-01-11 00:00:00	3828.000000	97.953000	6.320000	77.708000	7.250000	17.325000	6.810000	89.266000
2023-01-12 00:00:00	4222.200000	98.473000	6.050000	60.116000	6.920000	3.114000	6.910000	91.834000
2023-01-13 00:00:00	4269.600000	98.522000	6.420000	82.663000	7.130000	10.613000	6.490000	77.005000
2023-01-14 00:00:00	4092.700000	98.304000	6.880000	95.760000	7.360000	25.442000	4.060000	11.716000
2023-01-15 00:00:00	4493.300000	98.704000	6.920000	96.329000	7.630000	53.501000	4.600000	19.796000
2023-01-16 00:00:00	4885.300000	99.091000	7.390000	99.067000	7.850000	76.169000	5.270000	34.432000
2023-01-17 00:00:00	4821.600000	99.019000	7.030000	97.528000	8.040000	89.278000	5.870000	48.655000

Figure F.5: Tabulated data, flood 2023 number 1

2023-01-18 00:00:00	4542.400000	98.788000	7.060000	97.662000	8.220000	96.256000	5.380000	37.146000
2023-01-19 00:00:00	4179.900000	98.413000	7.330000	98.946000	8.360000	99.007000	4.660000	20.960000
2023-01-20 00:00:00	3629.000000	97.686000	7.320000	98.910000	8.450000	99.830000	3.880000	9.305000
2023-01-21 00:00:00	2994.100000	96.789000	7.180000	98.449000	8.500000	99.988000	3.230000	2.811000
2023-01-22 00:00:00	2467.700000	95.711000	7.110000	97.977000	8.480000	99.964000	2.830000	0.557000
2023-01-23 00:00:00	2234.300000	95.214000	6.790000	94.160000	8.350000	98.885000	6.900000	91.483000
2023-01-24 00:00:00	2026.500000	94.427000	6.310000	77.175000	8.090000	91.895000	7.090000	95.433000
2023-01-25 00:00:00	1744.900000	92.803000	5.800000	42.416000	7.710000	62.491000	7.240000	97.577000
2023-01-26 00:00:00	1546.300000	91.798000	5.700000	35.777000	7.400000	29.004000	7.250000	97.698000
2023-01-27 00:00:00	1407.200000	90.635000	5.850000	46.341000	7.580000	47.468000	3.790000	8.311000
2023-01-28 00:00:00	1273.700000	89.496000	6.060000	60.855000	7.680000	58.953000	4.370000	16.029000
2023-01-29 00:00:00	1165.900000	88.297000	6.290000	76.278000	7.710000	62.491000	5.040000	28.677000
2023-01-30 00:00:00	1001.300000	86.588000	5.920000	51.090000	7.740000	65.677000	5.730000	45.384000
2023-01-31 00:00:00	888.500000	85.086000	5.950000	53.005000	7.730000	64.708000	6.310000	66.852000
2023-02-01 00:00:00	799.400000	83.244000	5.930000	51.745000	7.680000	58.953000	5.780000	46.559000
2023-02-02 00:00:00	716.500000	81.669000	6.040000	59.607000	7.610000	50.630000	5.110000	30.264000
2023-02-03 00:00:00	652.600000	80.288000	6.330000	78.386000	7.620000	52.229000	4.520000	18.270000
2023-02-04 00:00:00	606.700000	79.004000	6.300000	76.581000	7.630000	53.501000	4.120000	12.467000
2023-02-05 00:00:00	596.600000	78.713000	6.430000	83.184000	7.610000	50.630000	3.900000	9.535000
2023-02-06 00:00:00	598.400000	78.750000	5.750000	39.084000	7.650000	55.682000	3.790000	8.311000
2023-02-07 00:00:00	616.600000	79.307000	5.510000	24.303000	7.580000	47.468000	3.750000	7.960000
2023-02-08 00:00:00	638.000000	79.937000	5.250000	12.127000	7.380000	27.320000	3.770000	8.105000
2023-02-09 00:00:00	616.900000	79.343000	5.030000	5.585000	7.200000	14.163000	7.140000	96.123000
2023-02-10 00:00:00	607.100000	79.053000	5.140000	8.529000	7.440000	32.396000	6.770000	88.139000
2023-02-11 00:00:00	574.200000	77.938000	5.530000	25.612000	7.640000	54.737000	6.350000	68.827000
2023-02-12 00:00:00	555.800000	77.417000	5.690000	35.292000	7.870000	77.914000	6.070000	55.028000
2023-02-13 00:00:00	528.000000	76.496000	5.940000	52.338000	7.970000	85.316000	4.830000	24.025000
2023-02-14 00:00:00	509.300000	75.757000	5.680000	34.517000	8.020000	88.200000	5.420000	38.091000
2023-02-15 00:00:00	495.300000	75.273000	5.780000	41.119000	8.060000	90.514000	6.020000	52.883000
2023-02-16 00:00:00	474.600000	74.497000	6.050000	60.116000	8.120000	93.167000	5.800000	46.983000
2023-02-17 00:00:00	459.800000	73.746000	6.330000	78.386000	8.210000	95.978000	5.210000	32.772000
2023-02-18 00:00:00	494.200000	75.224000	6.610000	89.956000	8.300000	98.062000	4.580000	19.385000
2023-02-19 00:00:00	474.600000	74.485000	6.640000	90.695000	8.350000	98.885000	4.040000	11.352000
2023-02-20 00:00:00	446.800000	73.249000	6.320000	77.708000	8.290000	97.831000	3.640000	6.639000
2023-02-21 00:00:00	432.100000	72.510000	6.280000	75.624000	8.120000	93.167000	3.470000	4.955000
2023-02-22 00:00:00	397.200000	70.463000	5.830000	44.645000	7.810000	72.535000	7.410000	99.176000
2023-02-23 00:00:00	401.200000	70.790000	5.830000	44.645000	7.570000	46.014000	7.410000	99.176000

Figure F.6: Tabulated data, flood 2023 number 1

2023-02-24 00:00:00	404.700000	71.032000	6.370000	80.349000	7.690000	60.116000	6.980000	93.543000
2023-02-25 00:00:00	424.600000	72.098000	6.150000	66.610000	7.740000	65.677000	6.620000	82.675000
2023-02-26 00:00:00	474.100000	74.449000	6.030000	58.675000	7.660000	56.676000	6.400000	71.505000
2023-02-27 00:00:00	494.200000	75.212000	5.940000	52.338000	7.560000	45.045000	5.150000	31.245000
2023-02-28 00:00:00	606.900000	79.016000	5.880000	48.340000	7.430000	31.718000	5.800000	46.983000
2023-03-01 00:00:00	748.000000	82.251000	5.870000	47.710000	7.330000	23.165000	6.420000	72.656000
2023-03-02 00:00:00	948.400000	85.874000	5.490000	23.431000	7.290000	20.039000	6.300000	66.453000
2023-03-03 00:00:00	1023.900000	86.843000	5.470000	22.110000	7.320000	22.171000	5.750000	45.966000
2023-03-04 00:00:00	978.800000	86.310000	6.210000	71.008000	7.370000	26.205000	5.250000	33.947000
2023-03-05 00:00:00	882.700000	84.965000	6.150000	66.610000	7.390000	28.132000	4.930000	26.205000
2023-03-06 00:00:00	860.300000	84.517000	5.850000	46.341000	7.360000	25.442000	4.760000	22.644000

Figure F.7: Tabulated data, flood 2023 number 2

50 Most Extreme Discharges

In this section, the data for the 50 most extreme discharges are shown.

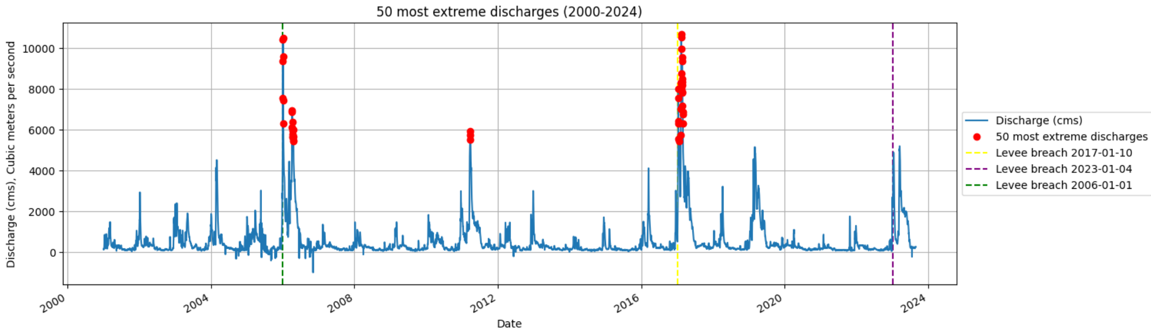


Figure F.8: Graph 50 most extreme discharge

Date	Discharge (cms)	P-Rank Discharge	Water level (feet)	P-Rank Water Level	HH-Tidal	P-Rank HH-Tidal	LH-Tidal	P-Rank LH-Tidal
2017-02-12 00:00:00	10669.800000	100.000000	7.600000	99.346000	7.810000	72.535000	3.570000	5.888000
2017-02-11 00:00:00	10551.500000	99.988000	7.890000	99.491000	7.950000	83.887000	3.740000	7.802000
2006-01-04 00:00:00	10482.100000	99.976000	7.010000	97.335000	7.530000	41.665000	6.950000	92.803000
2006-01-03 00:00:00	10419.300000	99.964000	7.670000	99.431000	7.930000	82.554000	6.820000	89.484000
2017-02-13 00:00:00	9975.800000	99.952000	7.200000	98.534000	7.580000	47.468000	7.350000	98.776000
2006-01-05 00:00:00	9595.000000	99.939000	6.810000	94.584000	7.190000	13.678000	3.290000	3.429000
2017-02-22 00:00:00	9571.000000	99.927000	7.130000	98.158000	7.540000	42.864000	5.980000	51.636000
2017-02-23 00:00:00	9361.500000	99.915000	7.140000	98.207000	7.610000	50.630000	5.540000	40.877000
2006-01-02 00:00:00	9353.200000	99.903000	8.560000	99.600000	8.200000	95.808000	6.690000	85.474000
2017-02-14 00:00:00	8745.100000	99.891000	6.960000	96.826000	7.420000	30.834000	7.290000	98.255000
2017-02-24 00:00:00	8509.500000	99.879000	7.130000	98.158000	7.710000	62.491000	5.120000	30.591000
2017-02-19 00:00:00	8363.200000	99.867000	7.000000	97.286000	7.340000	23.916000	5.460000	38.951000
2017-02-10 00:00:00	8312.800000	99.855000	8.140000	99.552000	8.020000	88.200000	4.130000	12.576000
2017-02-18 00:00:00	8243.000000	99.843000	7.410000	99.128000	7.330000	23.165000	4.910000	25.745000
2017-02-15 00:00:00	8212.800000	99.830000	6.820000	94.730000	7.390000	28.132000	6.970000	93.312000
2017-02-21 00:00:00	8194.100000	99.818000	7.160000	98.328000	7.450000	33.620000	6.380000	70.620000
2017-02-16 00:00:00	8108.300000	99.806000	7.400000	99.091000	7.390000	28.132000	6.680000	85.050000
2017-01-12 00:00:00	8019.000000	99.794000	8.420000	99.588000	8.170000	94.863000	3.420000	4.483000
2017-02-17 00:00:00	8015.100000	99.782000	7.160000	98.328000	7.350000	24.643000	6.480000	76.399000
2017-02-25 00:00:00	7861.900000	99.770000	7.210000	98.558000	7.780000	69.687000	4.720000	21.856000
2017-02-20 00:00:00	7822.800000	99.758000	7.090000	97.856000	7.380000	27.320000	6.060000	54.592000
2006-01-01 00:00:00	7561.600000	99.746000	8.820000	99.612000	8.340000	98.643000	6.540000	79.622000
2017-01-13 00:00:00	7559.900000	99.733000	8.000000	99.528000	8.090000	91.895000	3.040000	1.599000
2006-01-06 00:00:00	7445.500000	99.721000	6.980000	96.995000	7.420000	30.834000	3.880000	9.305000
2017-02-26 00:00:00	7192.300000	99.709000	7.430000	99.152000	7.790000	70.754000	4.310000	14.999000
2017-02-09 00:00:00	7005.800000	99.697000	8.270000	99.564000	8.080000	91.398000	4.710000	21.699000
2006-04-06 00:00:00	6932.000000	99.685000	6.050000	60.346000	6.840000	1.623000	6.470000	75.890000
2017-02-27 00:00:00	6882.300000	99.673000	7.320000	98.910000	7.700000	61.425000	3.970000	10.528000
2006-04-07 00:00:00	6879.000000	99.661000	6.410000	82.299000	6.880000	2.241000	6.430000	73.298000
2017-02-28 00:00:00	6744.500000	99.649000	6.930000	96.450000	7.510000	39.641000	3.790000	8.311000
2017-01-14 00:00:00	6411.800000	99.637000	7.520000	99.297000	7.950000	83.887000	7.070000	95.154000
2006-04-08 00:00:00	6399.100000	99.624000	6.530000	87.182000	7.020000	6.106000	5.990000	52.011000
2017-03-01 00:00:00	6318.200000	99.612000	6.640000	90.695000	7.550000	43.930000	7.250000	97.698000
2006-01-07 00:00:00	6307.400000	99.600000	7.150000	98.280000	7.600000	49.612000	4.570000	19.118000
2017-01-11 00:00:00	6296.100000	99.588000	8.320000	99.576000	8.220000	96.256000	4.090000	12.079000
2006-04-05 00:00:00	6113.400000	99.576000	6.780000	93.809000	7.080000	8.226000	5.880000	48.994000
2006-04-16 00:00:00	5985.300000	99.564000	7.260000	98.692000	8.050000	89.969000	5.990000	52.011000
2006-04-09 00:00:00	5963.000000	99.552000	6.250000	73.916000	7.150000	11.449000	5.620000	42.852000
2011-03-27 00:00:00	5911.600000	99.540000	6.410000	82.299000	7.120000	10.189000	6.290000	65.992000
2006-04-15 00:00:00	5788.100000	99.528000	7.170000	98.413000	7.880000	78.713000	4.940000	26.411000
2017-02-08 00:00:00	5727.500000	99.515000	7.780000	99.467000	8.080000	91.398000	5.400000	37.606000
2011-03-28 00:00:00	5721.800000	99.503000	6.130000	65.241000	7.060000	7.584000	6.700000	85.789000
2006-04-17 00:00:00	5675.300000	99.491000	6.990000	97.117000	8.100000	92.270000	5.970000	51.393000
2006-04-13 00:00:00	5667.100000	99.479000	6.860000	95.420000	7.390000	28.132000	5.160000	31.548000
2006-04-14 00:00:00	5623.600000	99.467000	7.060000	97.662000	7.620000	52.229000	5.030000	28.471000
2017-01-15 00:00:00	5569.000000	99.455000	6.890000	95.929000	7.710000	62.491000	7.200000	97.044000
2011-03-25 00:00:00	5523.200000	99.443000	7.170000	98.413000	7.620000	52.229000	4.980000	27.223000
2006-04-20 00:00:00	5474.600000	99.431000	6.380000	80.797000	7.560000	45.045000	5.660000	43.627000
2006-04-19 00:00:00	5443.200000	99.418000	6.520000	86.794000	7.840000	75.309000	6.180000	60.092000
2017-01-24 00:00:00	5440.200000	99.406000	6.840000	95.166000	7.670000	57.972000	5.400000	37.606000

Figure F.9: Table 50 most extreme discharge

50 Most Extreme Water Levels

In this section, the data for the 50 most extreme water levels are shown.

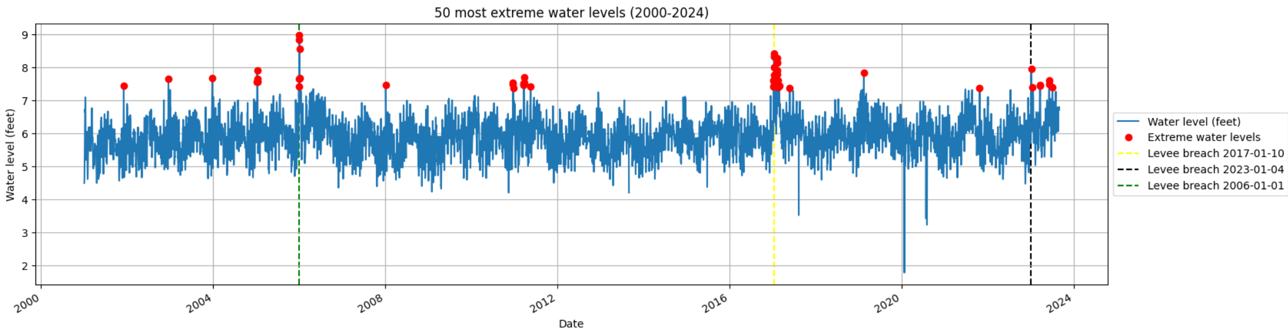


Figure F.10: Graph 50 most extreme water levels

Date	Water level (feet)	P-Rank Water Level	Discharge (cms)	P-Rank Discharge	HH-Tidal	P-Rank HH-Tidal	LH-Tidal	P-Rank LH-Tidal
2005-12-31 00:00:00	8.960000	99.624000	4850.700000	99.055000	8.310000	98.231000	2.690000	0.254000
2006-01-01 00:00:00	8.820000	99.612000	7561.600000	99.746000	8.340000	98.643000	6.540000	79.622000
2006-01-02 00:00:00	8.560000	99.600000	9353.200000	99.903000	8.200000	95.808000	6.690000	85.474000
2017-01-12 00:00:00	8.420000	99.588000	8019.000000	99.794000	8.170000	94.863000	3.420000	4.483000
2017-01-11 00:00:00	8.320000	99.576000	6296.100000	99.588000	8.220000	96.256000	4.090000	12.079000
2017-02-09 00:00:00	8.270000	99.564000	7005.800000	99.697000	8.080000	91.398000	4.710000	21.699000
2017-02-10 00:00:00	8.140000	99.552000	8312.800000	99.855000	8.020000	88.200000	4.130000	12.576000
2017-02-07 00:00:00	8.140000	99.552000	4406.500000	98.607000	8.030000	88.769000	6.070000	55.028000
2017-01-13 00:00:00	8.000000	99.528000	7559.900000	99.733000	8.090000	91.895000	3.040000	1.599000
2023-01-05 00:00:00	7.950000	99.515000	2357.200000	95.420000	7.700000	61.425000	3.950000	10.201000
2005-01-08 00:00:00	7.900000	99.503000	848.700000	84.311000	8.330000	98.534000	4.360000	15.883000
2017-02-11 00:00:00	7.890000	99.491000	10551.500000	99.988000	7.950000	83.887000	3.740000	7.802000
2019-02-14 00:00:00	7.830000	99.479000	2106.500000	94.778000	7.940000	83.366000	5.880000	48.994000
2017-02-08 00:00:00	7.780000	99.467000	5727.500000	99.515000	8.080000	91.398000	5.400000	37.606000
2017-01-10 00:00:00	7.760000	99.455000	4262.400000	98.510000	8.190000	95.493000	4.920000	26.000000
2011-03-23 00:00:00	7.700000	99.443000	5074.200000	99.225000	7.980000	85.837000	6.650000	83.790000
2003-12-24 00:00:00	7.670000	99.431000	591.200000	78.471000	8.270000	97.371000	2.560000	0.085000
2006-01-03 00:00:00	7.670000	99.431000	10419.300000	99.964000	7.930000	82.554000	6.820000	89.484000
2002-12-16 00:00:00	7.650000	99.406000	607.300000	79.065000	7.540000	42.864000	5.130000	30.858000
2005-01-09 00:00:00	7.650000	99.406000	909.000000	85.340000	8.430000	99.697000	3.550000	5.694000
2005-12-30 00:00:00	7.640000	99.382000	4720.200000	98.934000	8.300000	98.062000	3.260000	3.065000
2017-01-08 00:00:00	7.610000	99.370000	1914.000000	93.736000	7.910000	81.221000	6.180000	60.092000
2023-06-06 00:00:00	7.610000	99.370000	1352.500000	90.041000	8.200000	95.808000	6.370000	70.039000
2017-02-12 00:00:00	7.600000	99.346000	10669.800000	100.000000	7.810000	72.535000	3.570000	5.888000
2005-01-11 00:00:00	7.580000	99.334000	1041.700000	87.061000	8.410000	99.600000	2.620000	0.157000
2005-01-10 00:00:00	7.550000	99.322000	993.900000	86.479000	8.460000	99.879000	2.920000	0.921000
2005-01-07 00:00:00	7.550000	99.322000	961.200000	86.055000	8.220000	96.256000	5.160000	31.548000
2011-03-24 00:00:00	7.520000	99.297000	5234.200000	99.322000	7.860000	77.054000	6.530000	79.137000
2017-01-14 00:00:00	7.520000	99.297000	6411.800000	99.637000	7.950000	83.887000	7.070000	95.154000
2010-12-19 00:00:00	7.520000	99.297000	1513.300000	91.580000	7.980000	85.837000	4.150000	12.915000
2011-03-20 00:00:00	7.510000	99.261000	4048.100000	98.255000	7.630000	53.501000	4.090000	12.079000
2010-12-20 00:00:00	7.490000	99.249000	1809.100000	93.228000	8.070000	91.010000	3.540000	5.573000
2023-06-05 00:00:00	7.490000	99.249000	1356.000000	90.102000	8.290000	97.831000	5.340000	36.079000
2008-01-04 00:00:00	7.470000	99.225000	301.000000	60.007000	7.660000	56.676000	5.370000	36.928000
2023-03-19 00:00:00	7.450000	99.213000	5194.600000	99.309000	7.880000	78.713000	4.440000	17.046000
2011-03-21 00:00:00	7.450000	99.213000	4692.600000	98.873000	7.860000	77.054000	7.120000	95.820000
2001-12-02 00:00:00	7.440000	99.188000	732.100000	81.948000	8.100000	92.270000	6.230000	62.551000
2017-01-22 00:00:00	7.440000	99.188000	4928.300000	99.128000	7.540000	42.864000	6.140000	58.178000
2023-03-22 00:00:00	7.440000	99.188000	4603.200000	98.837000	7.500000	38.430000	3.780000	8.178000
2017-02-26 00:00:00	7.430000	99.152000	7192.300000	99.709000	7.790000	70.754000	4.310000	14.999000
2011-05-18 00:00:00	7.420000	99.140000	1386.100000	90.356000	7.990000	86.516000	4.920000	26.000000
2017-01-09 00:00:00	7.410000	99.128000	2705.400000	96.172000	8.090000	91.895000	5.720000	45.142000
2005-12-28 00:00:00	7.410000	99.128000	2635.100000	96.075000	8.120000	93.167000	4.740000	22.207000
2017-02-18 00:00:00	7.410000	99.128000	8243.000000	99.843000	7.330000	23.165000	4.910000	25.745000
2023-07-04 00:00:00	7.400000	99.091000	237.600000	50.824000	8.420000	99.649000	5.360000	36.588000
2017-02-16 00:00:00	7.400000	99.091000	8108.300000	99.806000	7.390000	28.132000	6.680000	85.050000
2023-01-16 00:00:00	7.390000	99.067000	4885.300000	99.091000	7.850000	76.169000	5.270000	34.432000
2021-10-24 00:00:00	7.370000	99.055000	220.800000	47.625000	7.550000	43.930000	6.170000	59.620000
2017-05-25 00:00:00	7.370000	99.055000	1583.600000	91.992000	7.970000	85.316000	5.340000	36.079000
2010-12-21 00:00:00	7.360000	99.031000	2991.500000	96.777000	8.160000	94.536000	3.040000	1.599000

Figure F.11: Table 50 most extreme water levels

G

Multivariate Analysis

The following Deepnote link will provide access to the Python code, used for the bivariate analysis. This code outputs the corresponding graphs for the bivariate analysis.

Link to the Deepnote for the multivariate analysis

H

Fault Tree

Tabulated overview volume overflow

In this section a table is shown that consists of all events where the water level has exceeded the 7 feet water level mark. The volume of overflow per meter width was calculated and shown in the last column of this table. Notice that the red color is events where failure occurred and yellow the identified near misses.

	Water level (feet)	Water level (meters)	Discharge	Water volume
Date				
2023-08-01 00:00:00	7.260000	2.212848	0.037732	295.955719
2023-06-06 00:00:00	7.610000	2.319528	0.135596	2370.315900
2023-03-20 00:00:00	7.450000	2.270760	0.085915	1280.366471
2023-01-06 00:00:00	7.950000	2.423160	0.263534	3358.559391
2022-07-13 00:00:00	7.310000	2.228088	0.049124	429.996397
2022-01-04 00:00:00	7.100000	2.164080	0.009000	58.482601
2021-10-25 00:00:00	7.370000	2.246376	0.064055	433.028407
2021-08-21 00:00:00	7.310000	2.228088	0.049124	698.130478
2021-06-25 00:00:00	7.330000	2.234184	0.053954	1015.051711
2019-06-18 00:00:00	7.030000	2.142744	0.001479	5.323964
2019-02-14 00:00:00	7.830000	2.386584	0.215213	3302.981114
2018-07-12 00:00:00	7.030000	2.142744	0.001479	5.323964
2017-06-23 00:00:00	7.280000	2.218944	0.042168	589.576278
2017-05-25 00:00:00	7.370000	2.246376	0.064055	623.797305
2017-02-11 00:00:00	8.140000	2.481072	0.346424	16534.702596
2017-01-13 00:00:00	8.420000	2.566416	0.481596	15896.831887
2014-12-20 00:00:00	7.110000	2.167128	0.010383	63.533474
2013-07-22 00:00:00	7.000000	2.133600	0.000000	0.000000
2012-12-13 00:00:00	7.240000	2.206752	0.033463	179.537628
2011-06-16 00:00:00	7.080000	2.157984	0.006440	39.963300
2011-05-18 00:00:00	7.420000	2.261616	0.077468	479.102462
2011-03-23 00:00:00	7.700000	2.346960	0.166685	3875.910136
2010-12-19 00:00:00	7.520000	2.292096	0.106722	2275.287550
2010-02-28 00:00:00	7.100000	2.164080	0.009000	32.400612
2010-01-30 00:00:00	7.140000	2.176272	0.014909	146.641298
2009-06-23 00:00:00	7.050000	2.148840	0.003182	14.353345
2008-01-04 00:00:00	7.470000	2.276856	0.091706	801.502277
2006-07-11 00:00:00	7.100000	2.164080	0.009000	55.584602
2006-06-12 00:00:00	7.200000	2.194560	0.025456	150.638421
2006-04-29 00:00:00	7.340000	2.237232	0.056425	948.347776
2006-03-31 00:00:00	7.170000	2.185416	0.019949	417.896123
2006-02-28 00:00:00	7.120000	2.170176	0.011831	45.489709
2006-01-01 00:00:00	8.960000	2.731008	0.780971	28014.492274
2005-12-02 00:00:00	7.010000	2.136648	0.000285	1.024597
2005-07-20 00:00:00	7.180000	2.188464	0.021735	233.601040
2005-05-26 00:00:00	7.130000	2.173224	0.013340	48.025095
2005-01-09 00:00:00	7.900000	2.407920	0.243005	6284.637679
2004-07-30 00:00:00	7.000000	2.133600	0.000000	0.000000
2004-02-26 00:00:00	7.030000	2.142744	0.001479	5.323964
2003-12-25 00:00:00	7.670000	2.337816	0.156086	1629.210935
2002-12-17 00:00:00	7.650000	2.331720	0.149149	1948.094241
2002-11-09 00:00:00	7.000000	2.133600	0.000000	0.000000
2001-12-03 00:00:00	7.440000	2.267712	0.083067	595.627170

Figure H.1: Tabulated data, volume of water due to overflow

I

Appendix Logic Tree

As stated in the previous section, a logic tree consists of states of nature. When the initiating event is described as a certain high water level, the states of nature will be the physical processes that influence that water level. These physical processes have been identified in the multivariate analysis: the air pressure, wind set-up, tidal influence, and discharge.

The hydrodynamical model takes these four processes into account and calculates the water level at Mallard based on their individual magnitudes. If this model is applicable the situation is referred to as a 'Transparent'. However as explained in REFERENCE, under certain conditions the model is invalid. These conditions will be referred to as 'Opaque'. Using these definitions the water level can either be caused by 'transparent' processes or 'opaque' processes (see Equation I.1 for a mathematical definition). This distinction will form the basis of the logic tree (Figure I.1)

$$P(I) = P(I|\text{Transparent}) + P(I|\text{Opaque}) \tag{I.1}$$

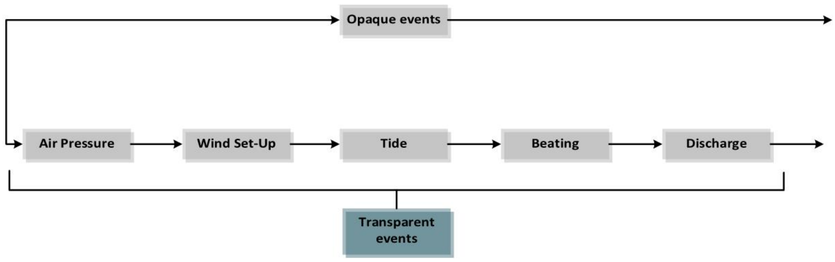


Figure I.1: Visualisation Opaque and Transparent processes

I.0.1. Construction logic tree

The columns of a logic tree are states of nature, so in this case, the 4 processes used in the hydrodynamic model. To allow for computational simplicity, the order of the columns should be based on dependency. The earlier the dependent events occur in the tree, the less computational power is needed. Since the multivariate analysis showed that air pressure and wind setup are dependent, these processes are put first. The order of the other independent ones is irrelevant but is defined as follows: first tide (whether HH-LH-HL-LL occurs), then the beating of the tidal cycle, and lastly the discharge.

Apart from this 'transparent' part of the event tree, the 'opaque' part is described by using a separate branch. As the columns in the transparent branches do not apply to the opaque branch, this branch immediately goes to a probability of occurrence of an initiating event. A visualization of all unique components of the logic tree is shown in Figure I.2. The choice of thresholds per column will be described in the next section.

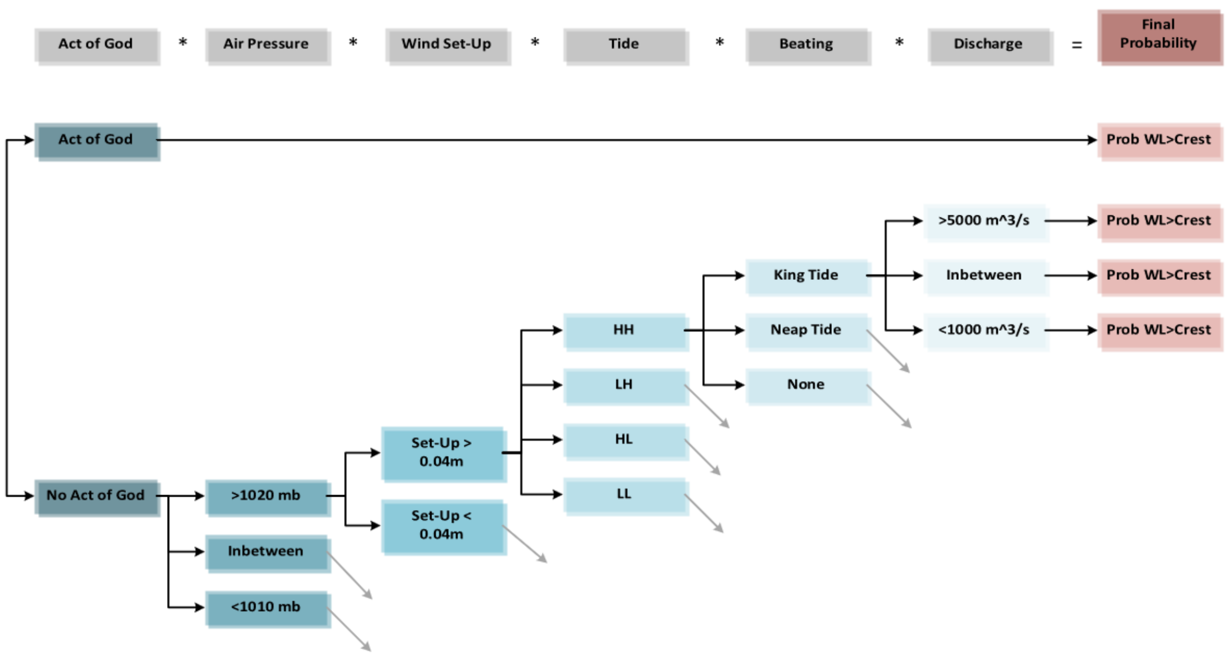


Figure I.2: Schematization of the logic tree for the probability of the water level exceeding the crest height.

I.0.2. Determining threshold

Air Pressure and Wind Setup

Three distinct events have been identified for the air pressure process. One with the mean of the air pressure (1015 mb) is set as the zero mark, the other two differ from the mean with ± 5 millibar. These thresholds are set as markers for low and high air pressures, based on visual inspection of the data.

The probabilities per day are calculated, by dividing the number of data points associated with a threshold by the total number of data points. These probabilities are denoted as $P(AP)$, for mathematical convenience, and given in Table I.1.

$P(AP > 1020mb) =$
$P(1010mb < AP < 1020mb) =$
$P(AP < 1010mb) =$

Table I.1: Probability a certain air pressure.

As the wind setup and air pressure are dependent, the daily probability of wind setup has to be calculated based on the outcome of air pressure. For example, lower air pressure results in a higher probability of wind setup than higher air pressure. The threshold for significant wind setup is set at 0.04 m, based on visual inspection of the data. Again for mathematical convenience, the probability of wind setup is denoted as $P(W)$. The results are given in Table I.2.

$P(W > 0.04m \mid AP > 1020mb) =$ $P(W < 0.04m \mid AP > 1020mb) =$
$P(W > 0.04m \mid 1010mb < AP < 1020mb) =$ $P(W < 0.04m \mid 1010mb < AP < 1020mb) =$
$P(W > 0.04m \mid AP < 1010mb) =$ $P(W < 0.04m \mid AP < 1010mb) =$

Table I.2: Probability of occurrence wind setup given a certain air pressure.

Tide and Beating

The tidal components are semi-diurnal effects, which are not influenced by the other physical processes in the tree. Therefore this branch is considered an independent event. As was noticed before initiating events always occurred during High-High tide. Since the sampling frequency Therefore the probability of occurrence.

The tide branch is still split into two branches, a tide branch, and a beating branch. A division into High High (HH), High Low (HL), Low High (LH), and Low Low (LL) tide, would not suffice to include the tidal component, as it is known that all of the floods have occurred during king tides. therefore next to the tide branch, a beating branch is included, where King tides and Neap tides are taken into account.

The probabilities of the tides occurring are pretty straightforward. As each branch has an equal probability of happening, because they occur in chronological order every time, the probability of each of them is 0.25. For the occurrence of king tide and neap tide, a similar reasoning occurs, as their appearance does not change per year.

$$\text{Probability of king tide} = 0.13 \tag{I.2}$$
$$\text{Probability of neap tide} = 0.13 \tag{I.3}$$
$$\text{Rest} = 0.74 \tag{I.4}$$

However, even though the probabilities do not change, their contribution to the water level does change, As the water level of king tide at Low Low gives a different contribution to the total water level than a water level with king tide at High High. So even though the probabilities do not change, the contribution of the tide to the water level does change with each branch.

I.0.3. Discharge

The final branch is the discharge branch. As stated in chapter 4, discharge can be considered an independent event of the other components, as the other components which would make it dependent are removed in its definition.

To determine the probabilities of certain discharges, a similar approach to the threshold selection of the air pressures is used, namely via visual inspection of the data. Based on this inspection, an upper boundary of 5000 cms is chosen, and a boundary of 1000 cms. These thresholds give the following probabilities.

$$\text{Probability of Q} > 5000 \text{ cms} = 0.003 \tag{I.5}$$
$$\text{probability of Q} < 1000 \text{ cms} = 0.872 \tag{I.6}$$
$$\text{Probability of rest} = 0.124 \tag{I.7}$$

Apart from these thresholds to determine probabilities, discrete discharges are chosen per branch, to make up for their contribution to the water level. For the highest discharge, a threshold of 5000 cms is chosen, for the middle branch, 1000 cms, and the lowest branch 200 cms. These values are based on the bivariate analysis.

With all these probabilities filled in, the total probability of a water level exceeding crest height can be calculated, as well as the actual height of that water level per branch.

talk about 3 not quantified events and how we got to that, already written!

I.0.4. Correlation water level and a chain of events

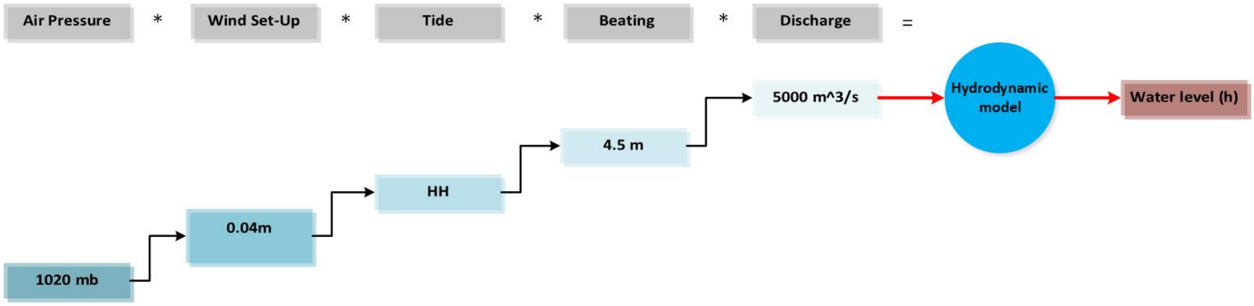


Figure I.3: Schematization of the logic tree for the probability of the water level exceeding the crest height.

$$P(\mathrm{AP} \geq 1020 \cap \mathrm{W} \geq 0.04 \cap \mathrm{T} \ \geq 1.87 \cap \mathrm{Q} \geq 5000) \tag{I.8}$$

I.0.5. Probability chain of events

The probability of a chain of events will be explained by The hydrodynamic model can calculate a water level given a set of conditions for the physical processes. So the probability of a water level can be calculated by using the probabilities associated to these conditions. An example of this in a mathematical form is shown in Equation I.9. The conditions are defined as thresholds.

$$P(\text{Arbitrary h}) = P(\mathrm{A} \geq \mathrm{th}_A \cap \mathrm{W} \geq \mathrm{th}_W \cap \mathrm{T} \ \geq \mathrm{th}_T \cap \mathrm{B} \geq \mathrm{th}_B \cap \mathrm{Q} \geq \mathrm{th}_Q) \tag{I.9}$$

Where A = Air pressure, W = Wind setup, T = Tide, B = Tidal beating and Q = Discharge

The joint probability of this set of conditions can be calculated. This is done in Equation I.10.

$$P(\mathrm{A} \cap \mathrm{W} \cap \mathrm{T} \ \cap \mathrm{B} \cap \mathrm{Q}) = P(\mathrm{A}|\mathrm{W} \cap \mathrm{T} \ \cap \mathrm{B} \cap \mathrm{Q}) * P(\mathrm{W}|\mathrm{T} \ \cap \mathrm{B} \cap \mathrm{Q}) * P(\mathrm{T}|\mathrm{B} \cap \mathrm{Q}) * P(\mathrm{B}|\mathrm{Q}) * P(\mathrm{Q}) \tag{I.10}$$

However since the tidal effects and the discharge are assumed independent of both air pressure, wind-setup and one another this can be simplified to Equation I.11

$$P(\mathrm{A} \cap \mathrm{W} \cap \mathrm{T} \ \cap \mathrm{B} \cap \mathrm{Q}) = P(\mathrm{A}) * P(\mathrm{W}|\mathrm{A}) * P(\mathrm{T}) * P(\mathrm{B}) * P(\mathrm{Q}) \tag{I.11}$$

For every branch the end result is a probability of occurrence for combination of parameter values based on the thresholds defined for this branch. So for example if one follows the upper branch this set of combinations is

I.0.6. Probability Opaque event occurring

$P(\text{Air pressure} > 1020\text{mb}, \text{Wind Set-up} > 0.04\text{m}, \text{HH-Tide}, \text{King Tide occurring}, \text{Discharge} > 5000 m^3/s)$ For every chain of events, a final probability is calculated.

J

Budget of Van Sickle Island

The following Deepnote link will provide access to the Python code, used to perform calculations on the budget for Van Sickle Island.

Link to the Deepnote for the budget

K

(Re)building costs Levee

Breach Size Estimates 2023

The 2023 breaches consist of two separate breaches, where the second one is cut up into three parts.

Figure K.1: Breach size estimates 2023 flood, based on field measures. [24]

Table K.1: Breach widths in feet, estimated via Google Maps and field measurements.

Breach Size Estimate	**First Breach**	**Second Breach**		
		Part 1	Part 2	Part 3
Field Measurements	51	19	71	108,8
Google Earth Measurement	53	22,5	73	105,8

Emergency Repair Quantities

Multiple methods have been used to estimate the quantities of the placed riprap during the repair work in early September. For the estimation of the placed dirt, the data on the freeboard is considered. All methods will be elaborated in the sections below.

Rip Rap Quantities

Bucket Counting

General info: The bucket used to distribute the riprap was 5 cubic yards.

Table K.2: Estimation of used cubic yards by the amount of riprap buckets per section.

	First breach	**Second breach**
Buckets	60	66
total (cubic yards)	300	330

Bucket Timing

General info: It was estimated the barge delivered 10 buckets per 15 minutes, equal to 50 cubic yards per 15 minutes. It should be noted that this method is deemed the least accurate.

Table K.3: Estimation of used cubic yards by filling duration per breach.

	First breach	**Second breach**
filling time (min)	115	130
total (cubic yards)	383,3333	433,3333

Estimation Cost of (Re)building Levee

The total cost of riprap was \$220.000, and the total cost of dirt was \$157.000. Therefore, if the dimensions of the new levee are known, the cost per meter cubed can be estimated. The total volume of the repaired levee was estimated as 5220 m^3. For a graphical explanation of this number see figure K.2.

Dirt

It is estimated that the total volume is filled by dirt for estimating the total price. This total price consists of the cost of the material and the placement of the material.
The price of dirt per cubic meter is calculated as follows: \$157.000/5200m^3 = \$30/m^3.

Riprap

It is difficult to quantify the actual volume of riprap used per meter width of levee. This is because of large irregularities in placement. Moreover, it is unknown how much riprap would be required to raise the levee by a certain increment. It was thus decided to compute the price of riprap per meter width per meter height. This was done by dividing the total cost of rip rap by the width and then dividing by the height. The calculations is as follows: \$220.000/180m/2.35m = \$520/m/m.

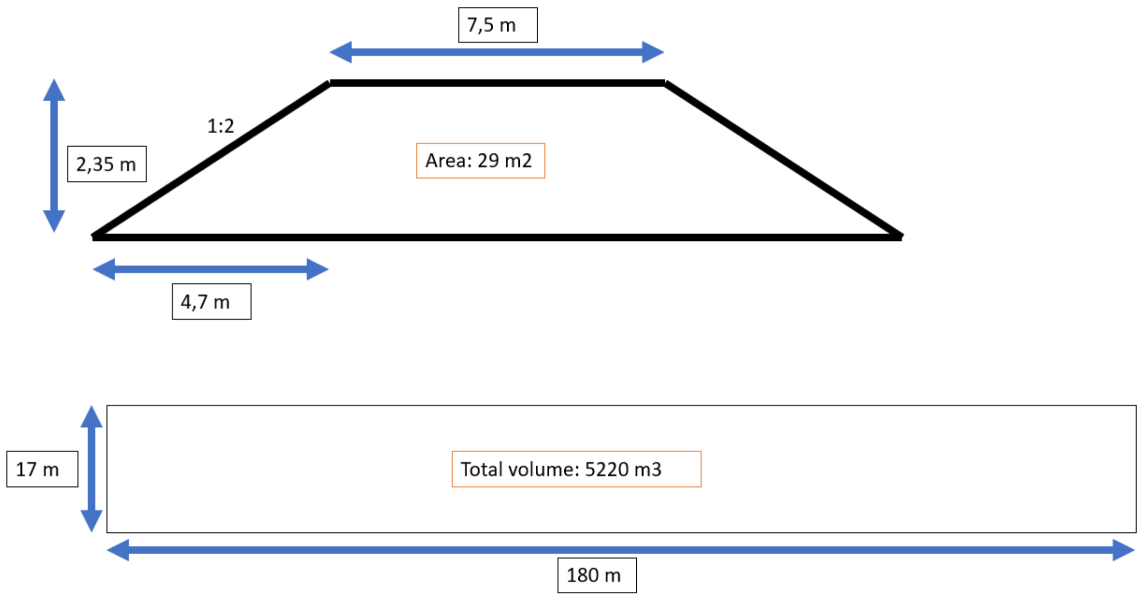


Figure K.2: Cross section and top view of total levee repairs.

Derivation of Equation 7.1

To calculate the economic cost of raising the levee, an estimate of the additional material placed on the levee must be made. This was done by estimating the material on the current levee. This assumes a crest height of y, a levee body that is 0.4 meters higher than the datum, a crest width of 3 meters, and a side slope of 3:1.

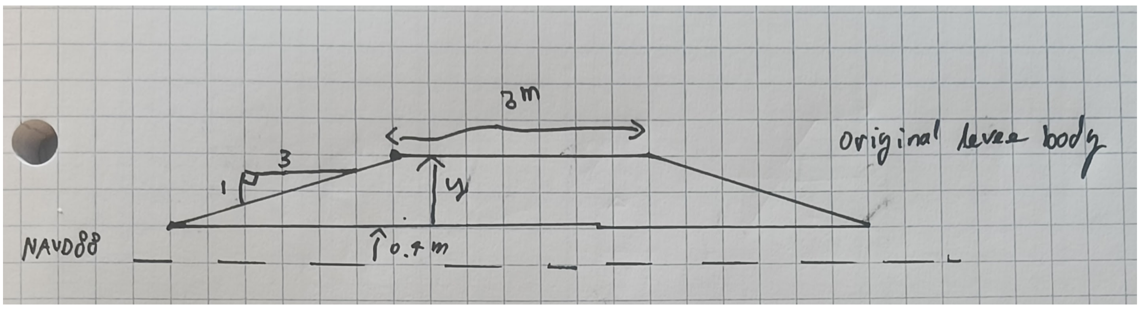


Figure K.3: Cross section assumed for original levee body.

The current volume per meter length can be calculated using Equation K.1

$$3(y-0.4)+3(y-0.4)^2 \tag{K.1}$$

The volume of a raised levee can be calculated by raising the crest height by x and assuming all other variables remain constant.

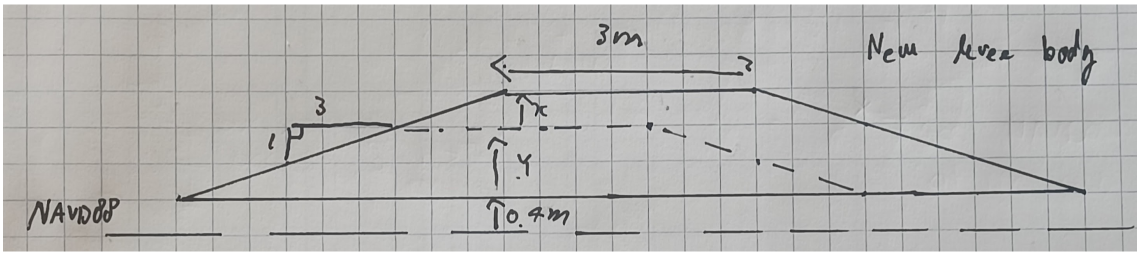


Figure K.4: Cross section assumed for new levee body.

The volume per meter length for a new levee body can be calculated using Equation K.2

$$3(x+(y-0.4))+3(x+(y-0.4))^2 \tag{K.2}$$

Therefore the additional material per meter length can be calculated by subtracting Equation K.2 from Equation K.1.

$$3x+3x^2+6x(y-0.4) \tag{K.3}$$

Since the cost of dirt per volume, and the cost of rip rap per meter height, per meter length are known, the total cost per meter length can be calculated using Equation K.4.

$$30(3x+3x^2+6x(y-0.4))+520x \tag{K.4}$$

L

Species Selection

Species Credits per Bank

Burke Ranche Conservation Bank	
Burrowing Owl (BUOW)	112,29
California Tiger Salamander (CTS)	710,426
Conservancy Fairy Shrimp (CFS)	10,73
Swainson's hawk (SWHA)	112,29
Vernal Pool Fairy Shrimp (VPFS) - preservation	259,186
Vernal Pool Tadpole Shrimp (VPTS) - preservation	266,476

Cayetano Creek Mitigation Bank	
Burrowing Owl (BUOW)	59,04
California Tiger Salamander (CTS)	71,88
California red-legged frog (CRF)	70,37
Congdon's Tarplant (CT)	7,64
San Joaquin Kit Fox (SJKF)	67,65

Elsie Gridley Mitigation Bank	
Burrowing Owl (BUOW)	1500,22
California Tiger Salamander (CTS)	1486,48
Conservancy Fairy Shrimp (CFS)	83,89
Swainson's hawk (SWHA)	1500,22
Vernal Pool Fairy Shrimp (VPFS) - preservation	56,31

North Bay Highlands Conservation Bank	
California red-legged frog (CRF)	441,14

Noonan Ranch Conservation Bank	
California Tiger Salamander (CTS)	152,06
Contra Costa Goldfields (LACO) - preservation	21,73
Vernal Pool Fairy Shrimp (VPFS) - preservation	3,33

Muzzy Ranch Conservation Bank	
Burrowing Owl (BUOW)	839,5
California Tiger Salamander (CTS)	877,14
Conservancy Fairy Shrimp (CFS)	38,1
San Joaquin Orcutt Grass (SJOG)	1
Vernal Pool Fairy Shrimp (VPFS) - preservation	327,3
Vernal Pool Tadpole Shrimp (VPTS) - preservation	38,1
North Delta Fish Conservation Bank	
AB360	56,46
Riparian SRA	13,92
Salmonid (Preservation)	253,56
Tule Marsh SRA	3,7
delta smelt/ longfin smelt	379,49
North Suisun Mitigation Bank	
California Tiger Salamander (CTS)	424,2
Contra Costa Goldfields (LACO) - preservation	41,2
Vernal Pool Fairy Shrimp (VPFS) - preservation	126,9
Vernal Pool Tadpole Shrimp (VPTS) - preservation	126,9
Ohione West Conservation Bank	
Alameda whipsnake (AWS)	629,08
California Tiger Salamander (CTS)	756,74
California red-legged frog (CRF)	630,36
Callippe Silverspot Butterfly	77,19
Oursan Ridge Conservation Bank	
Alameda whipsnake (AWS)	425,46
California red-legged frog (CRF)	429,9
Ridge Top Ranch Wildlife Conservation Bank	
California red-legged frog (CRF)	739,3
Callipe Silverspot Butterfly	662,6
River Ranch VELB Conservation Bank	
Valley Elderberry Longhorn beetle (VELB)	4542,1
East Austin Creek Conservation Bank	
Coho Salmon	144
Steelhead	144

Relevant Species Selection

To determine for which species credits, Van Sickle Island could potentially qualify, their habitat is compared to the potentially created habitat on Van Sickle Island and the current habitat on Van Sickle Island in dry conditions. Two conditions are applied to view the chances of having the species in the area. The count column represents the number of banks that got species credits per species and the total column represents the total amount of credits given out per species.

Species	**Habitat**	**Compatible with Van Sickle (Dry)**	**Compatible with Van Sickle (Flooded)**	**Count**	**Total**
BUOW	Dessert, plains, fields	NO	NO	4	2613,05
CTS	Annual Grassland, Oak woodlands	YES	NO	7	4478,926
CFS	Playa pools	NO	NO	3	132,72
SWHA	Grassland, flats	YES	NO	2	1612,51
VPFS	vernal pools (temporary wetlands	NO	NO	5	773,026
VPTS	vernal pools (temporary wetlands	NO	NO	3	431,476
CRF	streams, stock ponds	NO	NO	5	2311,07
CT	Seasonal wetland	NO	YES	1	7,64
SJKF	Desert grassland	NO	NO	1	67,65
SJOG	arid, semi-arid	NO	NO	1	1
LACO	vernal pools	NO	NO	2	62,93
Salmonid	Fresh/Salt water	NO	YES	2	507,12
delta smelt/ longfin smelt	San Francisco estuary	NO	YES	2	758,98
AWS	coastal scrubs, annual grassland, woodland habitat	YES	NO	2	1054,54
CSB	Native grasslands	YES	NO	1	77,19
VELB	riparian areas, oak woodlands	NO	NO	1	4542,1
Coho Salmon	coastal streams	NO	YES	1	144
Steelhead	cold water streams	NO	YES	1	144