Addicks & Barker Dams

An optimization to minimize damage due to flooding

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by

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DISCLAIMER

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NOTE

All calculations and analysis in this report are programmed in Excel and Python. These files are not attached to this report. If you are interested in the files, you can retrieve them by contacting one of the members of the project group.

Preface

This multidisciplinary project is set up by four Civil Engineering students as part of the MSc program at the Delft University of Technology. The project is conducted in cooperation with Rice University in Houston, United States of America.

The report contains information about the performance of Addicks and Barker Reservoirs. Initially, our research would be about the Texas City Levee in the Galveston Bay Area. However, the project started two weeks after Hurricane Harvey had hit Houston in the end of August. Because of the major impact of Harvey, where many people suffered damages, we decided to change our focus to the Addicks and Barker Reservoirs.

A special thanks is directed to the SSPEED Center for receiving us at their department and helping us throughout the project, the Netherlands Business Support Office because they provided us with work space in the Federal Bank, our supervisors Prof.dr.ir. S.N. Jonkman, Dr.B.L.M. Kothuis, Dr.ir.A.G. Sebastian and Ir.E.C. van Berchum for launching the project and providing us with their feedback and the companies who supported our project and which we could ask for advice: HKV Lijn in Water, Iv-Groep, Rijkswaterstaat and Royal HaskoningDHV.

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Abstract

The Addicks and Barker Reservoirs, built in the forties, are located in Houston and collect precipitation and run-off from upstream areas to reduce flood risks along Buffalo Bayou to protect downtown Houston. During Hurricane Harvey (August 25 - August 30, 2017), the precipitation reached a new record of 910 mm [36.2 inches] in a 4 day period in Houston. The gates of Addicks and Barker Reservoirs were opened during the night of 27-28 August which led to major damages due to downstream flooding. Besides, non-government owned land upstream was flooded due to high water levels in the reservoirs.

In this report, new design water levels for Addicks and Barker Reservoir are calculated based on inflowing discharge into the reservoirs and precipitation directly onto the reservoirs, including data of Hurricane Harvey. These calculated design water levels are compared with the critical water levels calculated based on the failure mechanisms of the dams. This study shows that the original design water level of the dams, based on the Probable Maximum Flood, are 2.83 m and 1.01 m higher than the critical water level for which failure of the dams can occur due to piping for Addicks and Barker Reservoir. However, the maximum allowed water level which is currently maintained by the United State Army Corps of Engineers, is 2.19 m and 2.46 m below the calculated critical water level. During Hurricane Harvey, these maximum allowed water levels were exceeded with 3.46 m and 1.93 m.

The damage of residential properties upstream and downstream of the reservoirs are minimized based on the distribution of excess volume from the inflow of creeks and precipitation onto the reservoirs. The ratio of the amount of volume which should remain upstream of the dams and the volume discharged into the Buffalo Bayou is calculated for every considered event with its duration and return period. The ratio of Addicks Reservoir is the dominant ratio, which should be used for both reservoirs. Run-off alone already produces damage, especially for the 12h and 24h precipitation, so the Addicks and Barker Reservoirs should not release discharge into the Buffalo Bayou for small durations. For events with a longer duration, it would cause less damage to open the outlets of the reservoirs than to keep them closed. However, if the water level in the reservoir exceeds the critical water level for piping, it is advised to discharge more to the downstream area to prevent breaching of the dams. Since the critical water level is reached for approximately 25% of the events at Addicks Reservoir, mitigations against piping should be taken to improve the minimization of damage. For Barker Reservoir, the critical water level is not reached in the optimization. During big events, people living upstream will be more affected by the flooding than people living downstream since this optimization is based on the damage minimization of residential properties.

List of Symbols

Sign	Description	Unit
$\$_A$	Total damage upstream of Addicks Reservoir	$million\ dollars$
$\$_B$	Total damage upstream of Barker Reservoir	$million\ dollars$
$\$_d$	Total damage downstream	$million\ dollars$
α	Angle of the slice shear stress	deg
α_f	Shape parameter of Fréchet	–
α_w	Shape parameter of Weibull	–
β	Angle between waves and levee	deg
β	Gumbel parameter	in
β_f	Scale parameter of Fréchet	–
β_w	Scale parameter of Weibull	–
$\delta\phi_{c,u}$	Critical head difference	m
$\delta\phi$	Head difference	m
δh	Wind set-up	m
δt	Duration time	h
δV	Total excess volume of water	m^3
η	Drag factor coefficient	–
η_f	Location parameter of Fréchet	–
η_w	Location parameter of Weibull	–
γ_β	Coefficient oblique wave attack	–
γ_b	Coefficient berm of levee	–
γ_f	Coefficient cover of the levee	–
γ_s	Volumetric weight of aquifer material	kN/m^3
γ_{sat}	Saturated volumetric weight	kN/m^3
γ_v	Coefficient horizontal wall on levee	–
γ_{veg}	Reducation factor vegetation	–
γ_w	Volumetric weight of water	kN/m^3
κ	Friction coefficient	–
λ	Damping factor	m
λ_h	Leakage factor	m
ϕ_{exit}	Potential at exit point	m
ρ_{air}	Density of air	kg/m^3
ρ_{water}	Density of water	kg/m^3
σ	Standard deviation	$same\ unit\ as\ data\ points$
θ	Bedding angle	deg
$\xi_{m-1,0}$	Iribarren number	–
d	Thickness hinterland blanket	m
d_{70}	70 percent fractal of the grain size distribution	m
d_{70m}	Reference value for d_{70}	m
e	Void ratio	–
$f(x)$	Probability density	$inverse\ of\ unit\ data\ points$
g	Gravitational constant	$9.81\ m/s^2$
h	Water level	m
$h_{10\%}$	Water elevation for 1/10 years discharge	m

h_d	Water level elevation downstream	m
h_p	Hinterland phreatic level	m
h_u	Water level elevation upstream	m
i	Exit gradient	m
$i_{c,h}$	Critical heave gradient	m
k_h	Hydraulic conductivity of the aquitard	m/s
k	Hydraulic conductivity of the aquifer	m/s
m	Rank of a certain value	–
n	Porosity	–
n	Amount of data points	–
u_{10}	Wind speed at 10 meter above the surface	m/s
v	Kinematic viscosity of water	$1.33 \times 10^{-6}\ m^2/s$
x_A	Incoming volume of water in Addicks Reservoir	m^3
x_B	Incoming volume of water in Barker Reservoir	m^3
B	Width of the levee	m
D	Aquifer thickness	m
F	Fetch	m
$F(x)$	Probability of exceedance	$inverse of unit of data points$
$H_{c,p}$	Critical head difference	m
H_{m0}	Wave height	m
H_s	Significant wave height	m
L	Seepage length	m
L_f	Length of effective foreshore	m
N	Precipitation	mm
Q	Discharge	m^3/s
Q_A	Discharge of Addicks Reservoir	m^3/s
Q_B	Discharge of Barker Reservoir	m^3/s
Q_{run}	Discharge from run-off	m^3/s
$R_{2\%}$	Free board wave Run-up	m
R_c	Free board wave overtopping	m
T_p	Wave period	s
T	Return Period	yr
V_d	Volume of water downstream	m^3
V_gov	Total volume of water in governmental land	m^3
V_{in}	Total inflowing volume of water	m^3
V_u	Volume of water upstream	m^3
X_0	Gumbel parameter	in
X_i	Annual maximum precipitation	in

Contents

1 Introduction

Houston is a city prone to flooding due to extensive rainfall, especially during the hurricane season. In the last few years, major floods have appeared in 2015, 2016 and even this year in 2017. Hurricane Harvey hit Houston on August 25 (2017) and flooded big parts of the city due to extreme rainfall. Many homes were flooded during this event. Not only homes that are located in the well-known floodplains in Houston, but even more neighborhoods.

The Addicks and Barker Reservoirs located in Harris County, North-West of Downtown Houston should be able to collect the rainfall and protect the downstream homes. However, during Hurricane Harvey both reservoirs seemed unable to perform their duty. Addicks and Barker both spilled their water into the Buffalo Bayou. During this flood, it seemed necessary to release more water than usually into Buffalo Bayou, flooding many homes to make sure that the dams from the reservoirs would not fail and create an even bigger disaster as a result.

After hurricane Katrina (2005), the Addicks and Barker Dams were checked and designated as “extremely high risk” when it comes to strength and safety of the dams [51]. The U.S. Army Corps of Engineers (USACE) took some measures to improve the conditions of the dams. However, the recent floods due to hurricane Harvey made clear that another close look should be taken into these dams, to guarantee the safety of Houston. In this report, the Addicks and Barker Dams will be discussed following the main research question:

> *What should the optimized performance of the Addicks and Barker Reservoirs be to minimize damage due to flooding in Houston?*

The report starts with introducing the area of interest, followed a literature study on the current condition of the Addicks and Barker Dams in Chapter 3. In this chapter, an explanation about the reservoirs system is given as well. In Chapter 4, the design water level of the dams will be redefined using a probabilistic approach. The maximum water level that the dams can handle before failing will be determined in Chapter 5, using Dutch failure mechanism calculations for earthen levees. In Chapter 6, an optimization between the water level in the reservoirs and the outflow will be made in terms of damage upstream and downstream of the dams. At last, Chapter 7 gives information about Hurricane Harvey and the performance of the reservoirs during this event followed by the conclusion and recommendations.

2

Area of Interest

As a response on the major flooding in December 1935 along the Buffalo Bayou in Houston, the USACE was authorized by the Congress to conduct the Buffalo Bayou and Tributaries Project (BBTP) [3]. The Addicks and Barker Reservoirs are part of this project as detention basins. Excessive amounts of rainfall and run-off from the Addicks and Barker watershed will be collected in the reservoirs to control the release in the Buffalo Bayou. Therefore term "watershed" encloses the area that drains into the reservoir.

These reservoirs are located in the southeast of Texas in the San Jacinto River basin about 17 miles west from downtown Houston. The Interstate Highway 10 (I-10), which stretches from the Pacific Ocean in California to the east all the way to Florida, runs between the two reservoirs with the Addicks Reservoir on the north and the Barker Reservoir on the south side. State Highway 6 (SH 6) is situated east from the Barker Reservoir and runs northwards through the Addicks Reservoir. See Figure 2.1 and 2.2 for an overview.

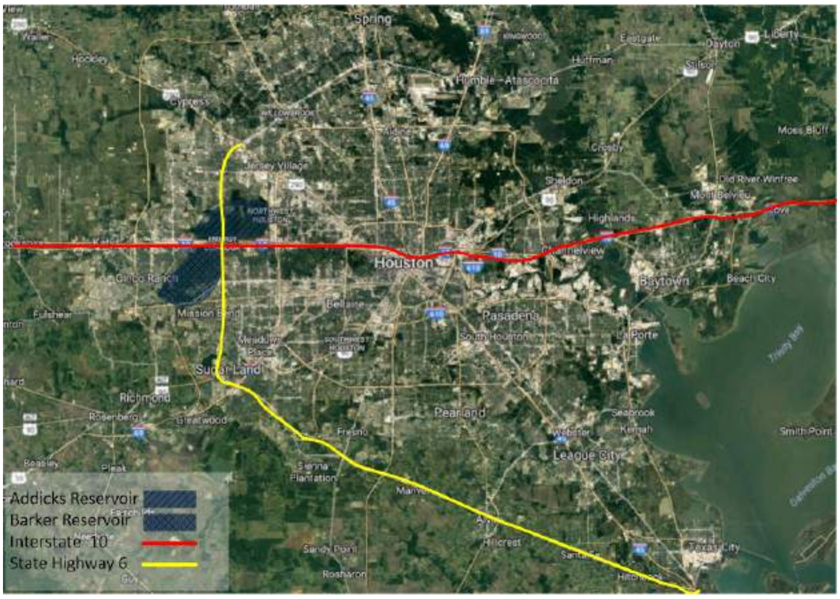


Figure 2.1: Project location

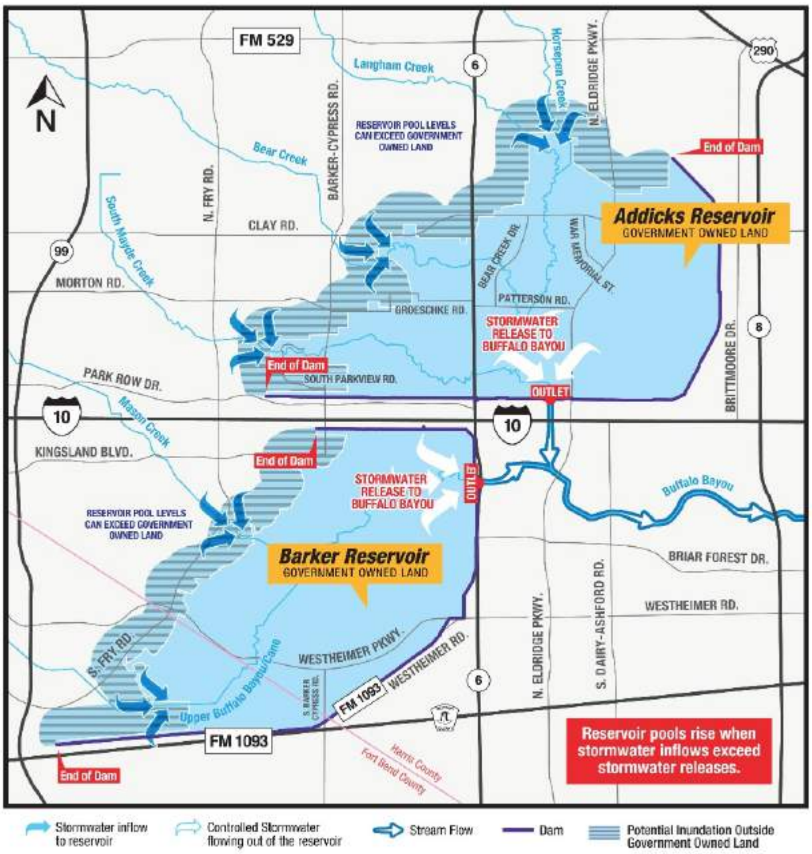


Figure 2.2: Close-up Addicks and Barker [HCFCD]

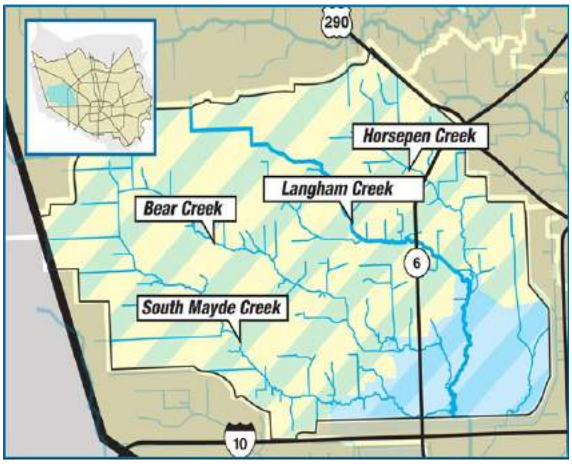
3 Current conditions

In this chapter, the water management system of the Addicks and Barker Reservoirs will be explained, followed by a summary of the history of the reservoirs, in which is looked at the original design plan which gives the boundary conditions used for the first design. Besides, an overview of adjustment made in the past is given followed by current information about the composition, land use and hydraulic boundary conditions. These two questions are answered:

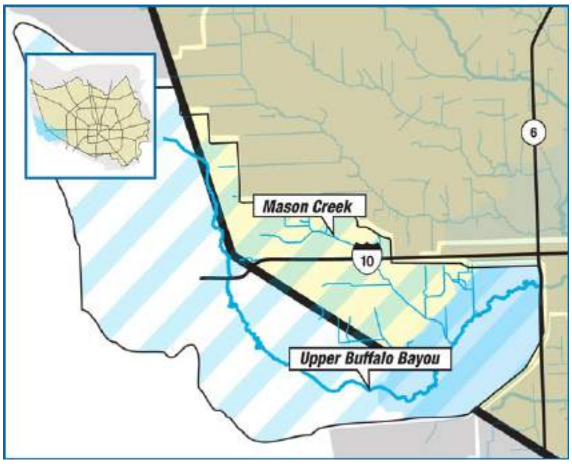
How does the Addicks and Barker Reservoir system work?
What is the current condition of the Addicks and Barker Reservoirs?

3.1. Addicks and Barker Reservoirs

The reservoirs are owned, operated and maintained by the USACE, because the construction in the forties was part of a federal project to reduce flood risks along Buffalo Bayou and protect downtown Houston. Leases and permits are given for recreational uses within the basin and residential development is growing upstream the reservoirs within the watershed. Nowadays, the estimated population of the Harris County portion in the watersheds is 295,694 in the Addicks and 88,895 in the Barker watershed. The Barker Reservoir watershed also covers a part of the Fort Bend County population. Besides, the undeveloped area in the watershed is used for agricultural purposes and the reservation of different wildlife habitats. New development needs to be approved by the USACE and may never impact the primary function flood risk management.



(a) Addicks [27]



(b) Barker [30]

Figure 3.1: Basin and watershed

As shown in Figure 7.3, the Addicks Reservoir watershed covers 357.42 km^2 [138.00 mi^2]. The rainfall within this area will be drained into the primary stream named Langham Creek and its tributaries Bear Creek, Horsepen Creek and South Mayde Creek, which are all open waterways which have a total length of 225.89 km [140.36 mi].
The Barker Reservoir has a drainage area of 326.34 km^2 [126 mi^2] and about 75.64 km [47.00 mi] of open streams, including tributary channels and the two primary streams: Upper Buffalo Bayou and Mason Creek[27] [30].

3.1.1. Outflow system

Figure 7.3 shows that the reservoirs are filled up with the water of the creeks in combination with the run-off and rainfall. The outlets in the reservoirs can discharge a certain amount of water into the Buffalo Bayou to prevent downstream Houston from flooding. In this watershed downstream the reservoirs live 444,602 people at a surface area of 246.18 km^2[102 mi^2], see Figure 3.2 [28]. When the reservoirs fill up too much and they cannot be emptied fast enough through the controlled outlets, the spillways will overflow in order to prevent overtopping of the dams. These spillways are uncontrolled and located at the tips of the dam [23].

Figure 3.2: Buffalo Bayou watershed [28]

According to the Interim reservoir control plan, the maximum allowed water levels are set on 29.72 m [97.5 ft] for the Addicks Reservoir and 28.53 [93.6 ft] for the Barker Reservoir. These water levels are the maximum water levels that are experienced in the past. In order to make sure that these water levels do not exceed the maximum allowed flow limit downstream, the joint discharge of the two reservoirs is measured at the gage at Piney Point in the Buffalo Bayou, located 17.22 km [10.70 miles] downstream in the channel below Barker dam. The combined discharge is raised from 56.64 m^3/s [2,000 cfs] to 113.27 m^3/s [4,000 cfs], including local runoff. These discharges are based on surveys done by the USACE. These surveys show that downstream of the reservoirs, property inundation will start when releasing more than 70.79 m^3/s [2,500 cfs] and when releasing more than 116.10 m^3/s [4,100 cfs] a large part of the structures in the downstream area (between North Wilcrest Drive and Chimney Rock Road) will experience flood damage [3].

For releases larger than 70.79 m^3/s [2,500 cfs], authorization from the District Engineer and District Dam Safety Officer (DSO) are required, however it is preferred that the District Engineer consults with the Division Engineer as well. Also areas prone to potential flooding downstream of the reservoirs will by visually monitored. For releases larger than 113.27 m^3/s [4,000 cfs] authorization by the Division Engineer is required [4]. The maximum discharge capacity for the outlets of the Addicks dam is 222.34 m^3/s [7,852 cfs] and for the Barker Dam 247.32 m^3/s [8,734 cfs], leading to a total maximum release capacity of 469.66 m^3/s [16,586 cfs] [3].

3.1.2. Land Use

The land use of the watershed can be divided into the area which is inside the reservoir or basin, owned by the government (from now one referred to as governmental land), and the area outside of the basin. Development is expanding from the edge of the reservoirs to the outer part of Houston. The governmental land of the reservoirs are partitioned in project lands which have a certain classification to meet the authorized purpose of the project: flood risk management. In Table 3.1, an overview is given of the area per Land Use Class in the Addicks and Barker Reservoirs. Residential properties are located close to border of the governmental land.

Project Operations (OPS) includes land for the safe and efficient operation and maintenance, primarily the earthen dams and land adjacent to the dam. The class of Recreation (Rec) and Proposed Recreation (PRec) introduced to identify the amount of land not related to the main purpose. Land with the label Environmentally Sensitive Area (ESA) requires special consideration and additional protection due to scientific, ecological, cultural or aesthetic features. Lands assigned to the Multiple Resource Management (MRM) Land-Use Class, consists of area for activities which do not interfere with the main purpose as Low Impact Recreation, Wildlife Management and Vegetative Management.

Table 3.1: Approximate area for each Land-Use Class by reservoir in hectares [3]

Land Use Class		Addicks	Barker	Total
Project Operations	OPS	769	927	1,717
Recreation	Rec	664	561	1,225
Proposed Recreation	PRec	45	138	182
Environmentally Sensitive Area	ESA	1,766	1,191	2,957
Multiple Resource Management	MRM	2,309	2,292	4,601
Total hectares				10,682

3.2. Addicks and Barker Dams

The Addicks and Barker Reservoirs are partly surrounded by dams, which are also owned by the USACE.The Barker Dam is built between 1942 and 1945 followed by the Addicks Dam between 1946 and 1948. Since 2009, these two dams are classified as 'Extremely High Risk'. In this paragraph, the current condition of the dams is described.

3.2.1. Original dam design in the forties

The original Addicks Dam is designed as a rolled earthen embankment with a length of 18.64 m [61.15 ft] and 14.78 m [48.49 ft] above the stream bed. The top of the dam has an elevation of 37.06 m [121.59 ft] above the National Geodetic Vertical Datum (NGVD) of 1929, which is comparable with the mean sea level datum of 1929, see Appendix B. The design is based on the maximum design water surface of 34.74 m [113.98 ft] NGVD. The Barker Dam design is a rolled earthen embankment as well with a length of 21.92 m [71.92 ft] and 11.13 m [36.50 ft] above the stream bed. The top of the dam has an elevation of 33.53 m [110.00 ft] and a maximum design water surface of 32.00 m [105 ft] NGVD [50].

The dams contain conduits to release water into the Buffalo Bayou. The Addicks Dam has five conduits with a length of 76.81 m [252.00 ft], width of 2.44 m [8.00 ft] and height of 1.83 m [6.00 ft]. The Barker Dam has five conduits as well with a length of 58.06 m [190.49 ft], width of 2.74 m [8.99 ft] and height of 2.13 m [6.99 ft]. The maximum capacity of the Addicks conduits is 222.34 m^3/s and of the Barker conduits is 247.32 m^3/s.

3.2.2. Improvements

Originally, one of the five conduits in both dams was gated to control the outflowing water. In 1948, two extra conduits in both dams are gated due to the rapid development of the city Houston [50]. In 1963, the last conduits were gated as well to control the outflowing water. Besides, there was decided to change the policy and limit the outflowing water 3.1.1.

In Table 3.2 an overview of all improvements made to the dams in the past years is given.

Table 3.2: Improvements made in past years

Year	Measure	Reference
1948	Two conduits in both dams gated	[50]
1963	Last conduits gated	[50]
1963	Change of policy, limit on outflow	3.1.1
1978-1982	Measures against seepage	[12]
1986-1991	Raised dam embankment	[12]
1987-1988	Roller-compacted-concrete at end of dams	[12]
1988-1991	T-wall Contract at Outlet	[5]
1998-1999	Outlet Structure Renovations	[5]
2009-2010	Fill Voids Under Conduits	[5]

3.2.3. Current dimensions

The Addicks Dam is 18.64 km [61,166 ft] long, the elevation of the top of the dam ranges from 35.78 m [117.40 ft] till 36.88 m [121.00 ft] and it has a crest of 3.66 m [12.00 ft] wide. The Barker Dam is 21.92 km [71,900 ft] long, the crest level ranges from 33.53 m [110.00 ft] till 34.47 m [113.1] and it has a crest width of 3.66 m [12.00 ft]. The dam embankment was raised in 1986 and 2013 to the current elevations [8], which are determined via the Lidar2008 map in ArcGIS.

The dams are divided into smaller dam sections as shown in Figure 3.3 and 3.4, which have the same characteristics in order to make clear conclusions about the failure mechanisms. The Addicks dam is divided into four dam sections (A1, A2, A3 and A4) and the Barker dam is divided into five dam sections (B1, B2, B3, B4 and B5).

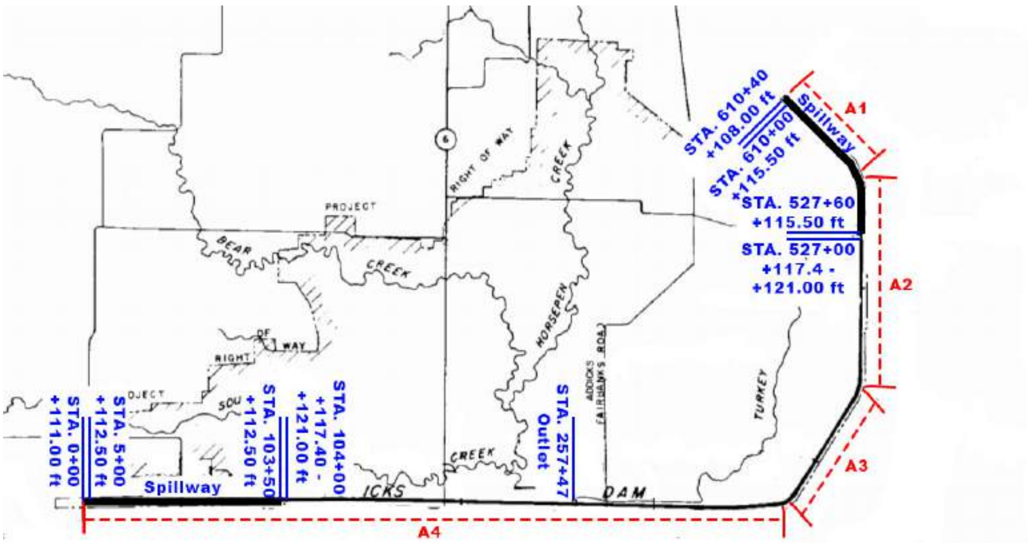


Figure 3.3: Map of the spillways of the Addicks dam

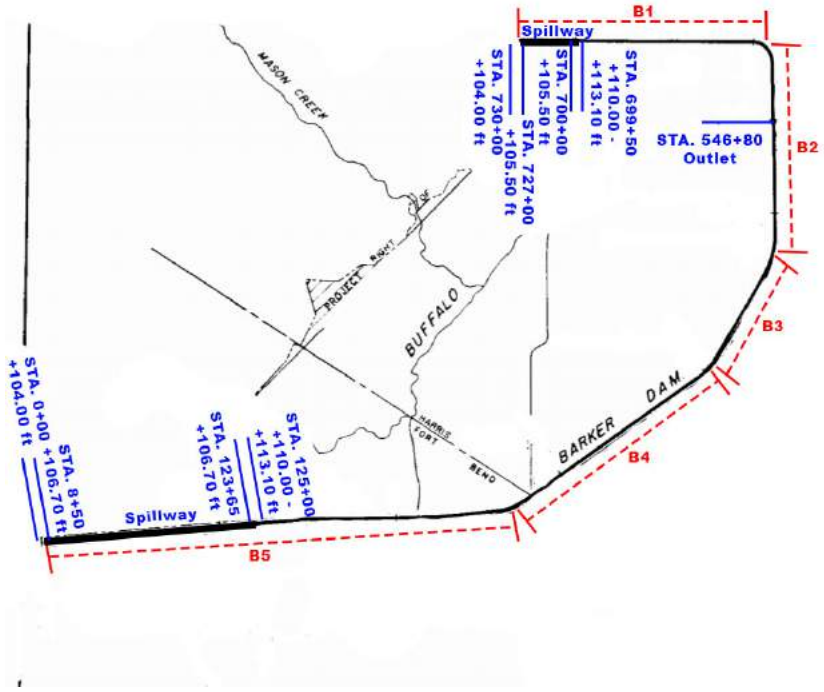


Figure 3.4: Map of the spillways of the Barker dam

The ends of the dams have a lower elevation than the crest height to serve as spillways during high water. In Figure 3.3, the locations of the spillways are shown along the different stations which have a distance of 30.48 m [100 ft] in between. So the spillway begins 125.4 m [STA 5+00*100 = 500 ft] from the lowest end of the dam. The elevation of the Addicks dam changes from STA 0+00 to 5+00 from the natural ground level of 32.92 m [108.00 ft] to 34.29 m [112.50 ft], which is the elevation of the spillway. This spillway runs to STA 103+00 and is at the crest elevation at STA 104+00. The spillway at the south end runs from STA 527+60 to STA 610+00 with an elevation of 35.20 m [115.50 ft] and back to natural ground elation at STA 610+40.

In Barker, the spillways have an elevation of 32.15 m [105.50 ft] at the north side (STA 700+61 to 727+00) and 32.52 m [106.70 ft] at the south side (STA 8+50 to 123+65). From 0+00 to 6+50 and 730+00 to 727+00 changes the elevation to the natural ground elevation of 31.70 m [104.00 ft] [8] [50].

The crest of the embankment sections functioning as spillways, are covered with roller-compacter concrete (RCC) slabs to protect the crown from eroding. These slabs consists of unreinforced pavement with a thickness of 254 mm [10 in]. If the water in the reservoir becomes too high, the water will spill over these RCC slabs into the ditch parallel to the dam, which discharge into the Buffalo Bayou. Since the reservoir is not a fully closed dike-ring as used in the Netherlands, there is also the possibility that the water flows around the ends of the dams. This leads to uncontrolled discharge into the developed areas. This why the lowest elevation at the end of the dam 32.92 m [108.00 ft] is used as the top of the spillway of the Addicks Reservoir in the newest report of the USACE [5]. For Barker, this is 31.70 m [104.00 ft].

4
Design water level

To minimize the damage due to an extreme rain event, the design water level of the reservoirs needs to be updated which will result in necessary dam heights in the following chapters. In this chapter, the following question is answered:

What should the design water level of the dams be?

Two methods are applied in order to determine the design water level of the Addicks and Barker Reservoirs. The first method is based on the precipitation in the reservoirs and the inflow of the creeks into the reservoirs, the second method is based on the past water levels of the reservoirs. Using the second method, data from Hurricane Harvey is excluded, because water was released from the reservoirs during the rainfall event which will influence the 'natural' water levels. By using these two methods, the influence of Hurricane Harvey and the human interference on the new water levels can be made visible. Both methods are considered separately for the Addicks and the Barker Reservoir. The current design water level of the Addicks 34.74 m [113.98 ft] and Barker 32.00 m [105.00 ft] Reservoirs is based on the Probable Maximum Flood (PMF), which is based on the Probable Maximum Precipitation (PMP). These parameters are used for the design of many hydraulic structures to determine and manage the flood risk in the United States of America. In appendix C, the parameters and methods are elaborated in order to understand how the design water levels are set.

4.1. Method 1: Precipitation and inflow discharge

Using this method, all data is gathered from several gages in order to have relevant precipitation and inflow discharges of the reservoirs. The data of the gages are available via the databases of USGS [53] and HCFCD [31]. All the gages are numbered, see Appendix E.The gage used for precipitation is located between the two reservoirs and therefore used for both reservoirs. There is specifically chosen for rain gages as they give very accurate data at discrete time instances for a specific location, for example at the Addicks and Barker Reservoirs. Other measurements, such as radar data, could be used as well, but is more often used in larger areas. Only raw data is used, except for the outliers in the precipitation data. The values larger than 0.25 m [10 in] per hour, are considered not possible and therefore removed from the data.

The precipitation and inflow data are re-sampled to get hourly data. An extreme value analysis is applied to receive the needed maximums of the two datasets per duration (1h, 2h, 4h, 8h, 12h, 24h, 48h, 72h and 96h). The General Extreme Value-distribution (GEV-distribution) and the Log-Pearson type III are fitted onto the maximum data in Python, see Appendix D.1 and D.2. Gumbel, Weibull and Fréchet are fitted within the GEV-distribution and because Log-Pearson type III is used commonly in Harris County on hydrological data, this distribution is taken into account as well. The root mean square error method is applied to see which distribution fits the data best. The data is extrapolated to include the return periods of 10, 25, 50, 100, 250, 500, 1,000 and 10,000 years. See Appendix E.2 for further information about the analysis. The results for the precipitation and inflow will be given in the following paragraphs.

4.1.1. Precipitation

The precipitation is summed up per duration using a moving window. The precipitation data ranges from 1987 till 2017. The measurements starting from 1987 till 2002 are measured every hour and after 2002 every 15 minutes. For each duration, the annual maximum value is calculated. The GEV-distribution and the Log-Pearson type III for each duration are fitted onto this data and are compared by applying the root mean square error method, see Appendix E.3. The Weibull distribution overall fits the data best, but fits the 48h, 72h and 96h poorly. Therefore the decision is made to use the Gumbel distribution for all durations, see Figure 4.1 for the results. The values of precipitation for all durations and return periods can be seen in Appendix E.3.

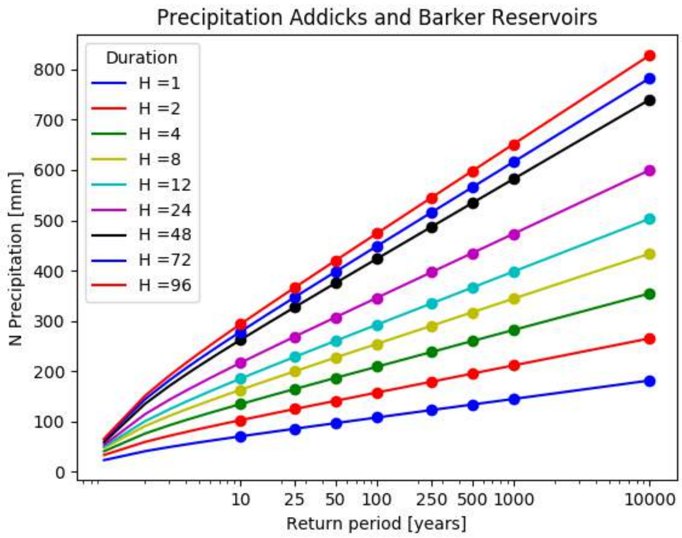


Figure 4.1: Results of the precipitation for different durations and return periods using a Gumbel distribution

Harvey is estimated to be a more than 1/1,000 year event for most locations in Harris County. Harris County uses three hydraulic regions within Houston, the Addicks and Barker Reservoirs are both located in Region 1. The design rainfall for this region for a 1/100 year rainfall event with a duration of 24h is 314.96 mm [12.40 inches] and for a duration of 4 days (96h) 378.46 mm [14.90 inches]. These values where calculated by USGS with data until 1998 [54]. The results from the Gumbel fit for a 1 /100 year rain event with a duration of 24h are 346.02 mm [13.62 inches] and for a duration of 4 days (96h) 474.11 mm [18.67 inches]. This is significantly higher than the currently used 1/100 year values by the USGS. The Gumbel fit results are including the recent big rainfall events in the analysis. However, the Gumbel fit for the precipitation data may still underestimate the rainfall which occurred during Hurricane Harvey. For a duration of 96 hours, the fitted rainfall is 812.8 mm [32 inches] for a 1/10,000 year event, while 927.1 mm [36,5 inches] was observed during Hurricane Harvey.

4.1.2. Inflow

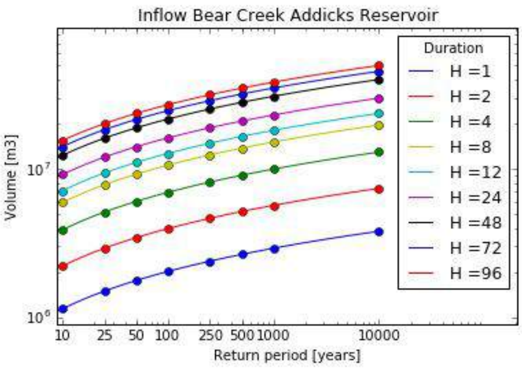
The extreme value analysis for the discharge differs from the analysis used for the precipitation data, because the discharge is a value at a certain moment in time and cannot be summed up hourly like the precipitation data. All discharges from the inlets are available per 15 or 30 minutes. For the discharge, the maximum values per hour are used and no duration is included.

For the Barker Reservoir, the inflow data is available from 1990 (27 years) and for the Addicks Reservoir the discharge data is available for Langham Creek from 2001 (16 years), for Bear Creek from 1993 (24 years) and South Mayde Creek from 2015 (2 years).For a reliable extreme value analysis, the data should be at least 25 years. The South Mayde Creek is too short to do an extreme value analysis. Because no other data is available for the inflow into the Addicks Reservoir, the short time series of Langham and Bear Creek are still used for the extreme value analysis. When more data is available, the analysis can be made more accurate and thus more reliable.

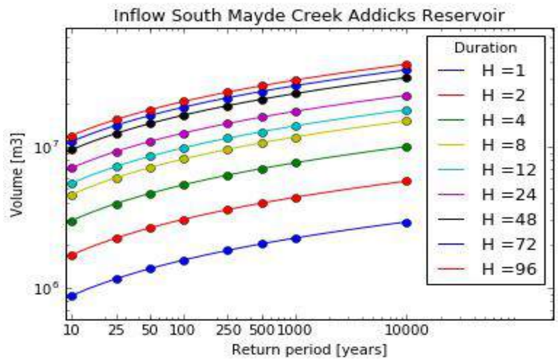
The hourly discharge data is converted to volumes in acre feet, because these volumes are necessary in order to find the water elevations in the next step following a volume-elevation graph. The discharge per hour is converted to a Volume. Using the moving window approach with the same durations used as for the precipitation, the maximum volumes per duration are calculated. In Appendix E, the distributions and errors per duration are given. The chosen distributions are:

- Langham Creek: Fréchet
  Log-Pearson type III fits best for Langham Creek according to the root mean square error, but does not give a good fit. Therefor the second best fit is used: Fréchet.
- Bear Creek: Gumbel
  Weibull gave the best results in the root mean square method, but did not give a good fit for half of the durations. Fréchet underestimates the data, because it is too low at the upper right tail. Therefore Gumbel is preferred, it performs better at the longer durations than Log-Pearson type III.
- South Mayde Creek: None
  Because South Mayde Creek has only 2 years of data, extrapolating this data is not reliable. Therefore, Bear Creek and South Mayde Creek are compared with the discharges of Mayde Creek, this resulted in a ratio of (1.3). South Mayde Creek is extrapolated using the extrapolated data of Bear Creek multiplied with a factor of (0.77). It is also checked with Langham Creek, but the data of Langham is shorter than Bear Creek (2001 vs 1993) and secondly, the standard deviation was greater.
- Upper Buffalo Bayou: Fréchet
  Log-Pearson type III did not fit well, the estimated volume at 1/10,000 years was reasonably too high. Fréchet is the second best fit following the root mean square error and is therefore used for the fit of the Upper Buffalo Bayou.

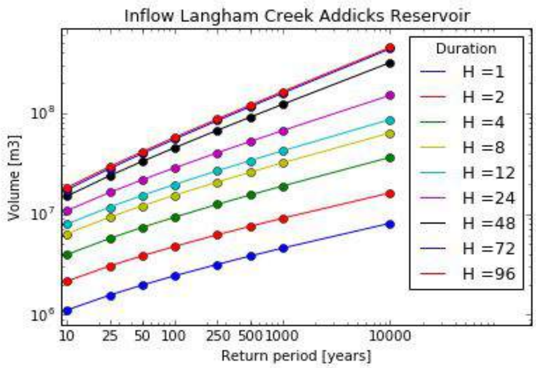
In Figure 4.2, the extrapolated volumes are given for the Addicks and Barker reservoirs.



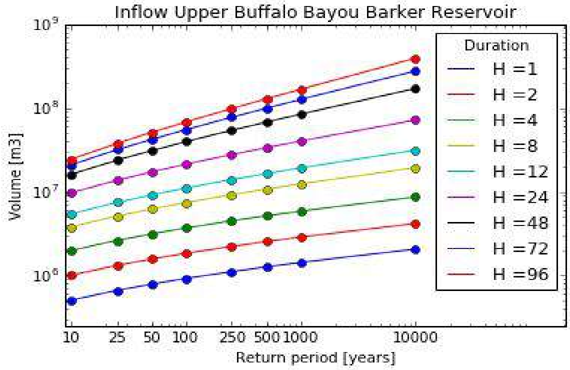
(a) Bear Creek



(c) South Mayde Creek



(b) Langham Creek



(d) Upper Buffalo Bayou

Figure 4.2: Inflow Volumes in the Addicks and Barker Reservoirs

4.1.3. Elevations

The calculated volumes of the inflowing creeks give in combination with the rain that falls directly into the reservoirs, an elevation of the water levels in the reservoirs. The precipitation is transformed to a volume by multiplying the precipitation with the total volume of the reservoirs. The incoming volume from the creeks and from the precipitation is summed up and conferred to an elevation via the Volume-Elevation Graphs. The Volume-Elevation Graphs are derived from the USACE [5], See Appendix E.4 for the used Volume-Elevation Graphs.

Normally, the inflowing creeks are (almost) empty and are filled up with water from the rainfall during the same rain events as used for the Addicks and Barker Reservoirs. Therefore, it is assumed that the elevations from the inflow and precipitation can be combined. However, during big storms, overflow from Cypress Creek can come into the creeks connected to the reservoirs, resulting in a bigger discharge than expected. This area is located north the reservoirs and could have a different amount of rainfall. So for bigger storms, summing up the discharge and precipitation is less reliable.

See Tables 4.1 and 4.2 for the design water level per duration of Addicks Reservoir and Barker Reservoir.

Table 4.1: Addicks Elevation in meters

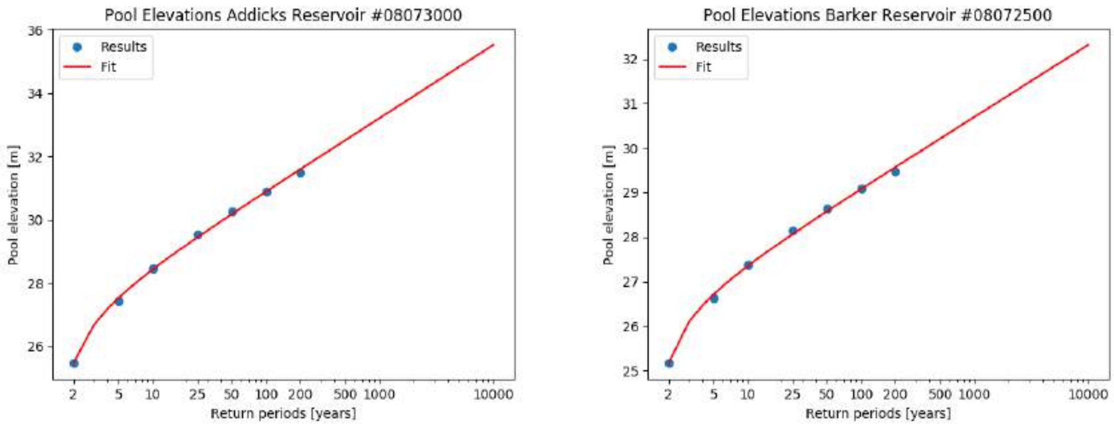
	1h	**2h**	**4h**	**8h**	**12h**	**24h**	**48h**	**72h**	**96h**
10	26.71	27.26	27.77	28.21	28.45	28.81	29.01	29.24	29.58
25	26.98	27.56	28.12	28.62	28.90	29.33	29.92	30.18	30.31
50	27.15	27.76	28.37	28.91	29.23	29.73	30.44	30.77	30.91
100	27.32	27.96	28.62	29.21	29.57	30.15	31.01	31.44	31.59
250	27.53	28.20	28.93	29.61	30.02	30.75	31.89	32.48	32.65
500	27.68	28.38	29.18	29.92	30.39	31.25	32.68	33.43	33.63
1,000	27.83	28.56	29.44	30.26	30.79	31.81	33.61	34.58	34.80
10,000	28.32	29.19	30.40	31.59	32.44	34.32	38.10	40.19	40.48

Table 4.2: Barker Elevation in meters

	1h	2h	4h	8h	12h	24h	48h	72h	96h
10	25.82	26.12	26.38	26.62	26.79	27.07	27.43	27.61	27.75
25	25.97	26.28	26.56	26.83	27.02	27.36	27.81	28.06	28.25
50	26.06	26.39	26.68	26.97	27.19	27.57	28.10	28.42	28.67
100	26.15	26.49	26.80	27.10	27.34	27.78	28.41	28.81	29.14
250	26.26	26.61	26.94	27.27	27.54	28.07	28.86	29.40	29.86
500	26.33	26.69	27.04	27.40	27.69	28.29	29.23	29.91	30.52
1,000	26.40	26.77	27.13	27.52	27.85	28.52	29.65	30.51	31.31
10,000	26.61	27.01	27.43	27.93	28.37	29.43	31.51	33.40	35.40

4.2. Method 2: Water levels

Using this method, the past water levels are considered in order to get the design water level for the reservoirs. The water levels during Hurricane Harvey will be excluded, because the releases during the rainfall event will influence the 'natural' water levels. The water levels are available from 2007 and 2008 for the Addicks and Barker Reservoirs. This is not enough date to do a reliable extreme value analysis, nevertheless to compare the data, the water level data is still considered. The distribution of Log-Pearson type III fits the elevation data best and is used for extrapolating, see Appendix E.5 for the reason, the distribution plots and errors. The results of the analysis are shown in Figures 4.3 and Tables 4.3, 4.4.



(a) Addicks

(b) Barker

Figure 4.3: Water elevation in reservoirs according to a Log-Pearson type III distribution

Table 4.3: Elevations in Addicks Reservoir

	Ret. Periods							
Elevation	**10**	**25**	**50**	**100**	**250**	**500**	**1,000**	**10,000**
feet	93,27	96.63	99.02	101.35	103.65	106.68	108.96	116.53
m	28,43	29.45	30.18	30.89	31.59	32.51	33.21	35.52

Table 4.4: Elevations in Barker Reservoir

	Ret. Periods							
Elevation	**10**	**25**	**50**	**100**	**250**	**500**	**1,000**	**10,000**
feet	89.73	92.08	93.76	95.39	97.00	99.11	100.71	106.00
m	27.35	28.07	28.58	29.07	29.56	30.21	30.70	32.31

4.3. Conclusion and discussion

Methods 1 and 2 gave design water levels for different durations and return periods. Method 1 includes hurricane Harvey and has more available time series and is therefore considered more reliable than method 2. The results of method 1, shown in Table 4.1 and 4.2, will be used in Chapter 6.

Note that the calculated design water levels contain some uncertainty because of multiple reasons. Firstly, Mason Creek and Horsepen Creek are not included in the analysis, so more inflow is expected. Besides, the discharges of the gages are used in the analysis, but behind the gages there is also some local rainfall with run-off, so the inflow is actually higher when entering the reservoirs; Because South Mayde Creek did not have enough data, it is assumed to be 0.77 times Bear Creek. The GEV-analysis is done even with some datasets shorter than 25 years, with more data available, the analysis can be made more reliable. Additionally, the rainfall analysis is done for only one gage, which limits the way it can be used for comparison with the analysis of Harris County. For a full understanding of what Hurricane Harvey did to the design water level, an analysis should be done for all gages in Harris County, which is out of the scope of this project. Furthermore, a proper joint distribution function should be derived to verify how the discharge and the precipitation are related to make future results more reliable.

5

Failure of the dams

Currently, the maximum allowed water level is based on the highest water level observed in the past, because it is known that the dams can handle this water level. However, it could be possible that the dams can stand an even higher water level before failure will occur. By knowing when the dams will fail, a maximum possible water level for the reservoirs can be determined. In this chapter, the following sub question is answered:

What water level will cause failure of the dams?

The water level which causes failure of the dams, will be determined by looking into the failure mechanisms. The dams consist of earthen embankments with spillways and outlets where the water can discharge from the reservoirs. The outlets and spillways have already been redesigned and are under construction by the USACE, so in this project only the dams itself will be considered. The assumption is made to consider these sections of the dams as levees for the structural design since the failure mechanisms of these sections will be similar to the ones of an earthen levee in the Netherlands. Besides, the water level for most dams is fairly constant; the water level in Addicks and Barker Reservoir varies significantly over time which is more similar for levees.

First, the current conditions of the dams are elaborated including the structural design and the boundary conditions determined by the soil and hydraulics. Next, there is looked into individual failure mechanisms for the current cross-sections and conditions, which lead to a failure water level per failure mechanism. These critical water levels can be compared to the water levels from Chapter 4 to find the safety level of the dam.

5.1. Boundary conditions and loads

5.1.1. Still water level

The still water level, SWL, is the unknown in this chapter, so taking the current embankment design as starting point, the still water level in an extreme event is calculated. This level is caused by a combination of natural phenomena such as precipitation and wind, and human interventions such as development and management of the outlets.
The inflow equals the outflow plus the amount of water that is retained in the reservoir, which can been measured as the rise or fall of the water level. On top of that, the wind has a big influence on the water level as it tilts the water surface. Especially during a hurricane with high wind speeds, the water can be pushed up till an amount of 0.23 m for the Addicks dam and 0.18 m for the Barker dam. The calculation of the wind set-up can be found in Appendix G.1.

5.1.2. Wind waves

On top of the tilting of the water surface which is included as wind set-up, the wind can also cause waves. These wave conditions are determined by using the simplified Donelan/JONSAWP method (1996), because this is a conservative method which can only give a small over-prediction [1]. The height of the waves depends on the fetch, gravitational acceleration and the design wind speed, using the basic wind speed as explained in Appendix G.1. The height and wave period are determined for each dam section per wind direction with

the adapted formula of the Young and Verhagen (1996) by Breugem and Holthuijsen (2006). This formula is a more accurate approximation for lakes and reservoirs than the original formula [7].
For the determination of the wind set-up 5.1.1, overtopping and run-up 5.2.2, it should be taken into account that the reservoirs have no free fetch. Using Google Earth [25] 80%, it can be seen that the area in the Addicks and Barker Reservoirs is covered with trees. The wave height reduction due to these trees and other vegetation located in the reservoirs is estimated using research done by USACE [**?**]. According to this report, the wave height reduction factor is 0.74%/m for each 100 m for a mangrove area. Mangroves have very densely packed small stems and are therefore a better wave height reducer than the big trees standing in the Addicks and Barker reservoirs. On the other hand, the trees in the reservoirs can block the wind and therefore the development of wind waves. The assumption is made that the wave height reduction in total would be the same as that of a mangrove area.

5.1.3. Geotechnical boundary conditions

Both the Addicks as Barker Dams are rolled earth embankments which means that they consist of consecutive thin layers which are compacted at optimum moisture content with rollers. Because of this construction method, there is some inhomogeneity and anisotropy along the length of the dams. Since the majority of the layers consists of lean clay (CL), clayey sand (SC) and fat clay (CH), the dams are considered as clay embankments. There are slightly more silt and silty sand (SM) layers in the top of the Addicks levee than the Barker levee, but in the General Design Memorandum, it is still considered as a clay levee built on a silty sand foundation layer [50]. See Figure 5.1 for a cross-section of Addicks Dam and see Table 5.1 for the corresponding soil layers and their properties.

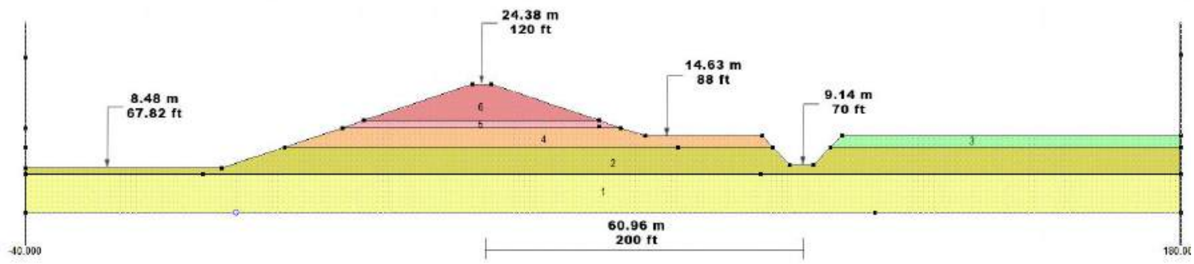


Figure 5.1: Cross-section Addicks Dam section A4, for legend see Appendix F

Table 5.1: Soil layers with their corresponding soil properties

Layers	Elevation [m]	γ_s [kN/m^3]	γ_d [kN/m^3]	ϕ [Deg]	c [kN/m^2]
V: V/Stiff Sandy Clay (CL)	h_c - 29.72	20.89	11.08	15	43.09
IV: V/Stiff Clay (CH)	29.72 - 28.35	19.48	11.96	11	38.30
III: V/Stiff Sandy Clay (CL)	28.35 - 24.69	21.36	11.55	11	43.09
II: Dense Silty Sand (SM)	24.69 - 19.51	20.89	11.08	33	0
I: Hard Clay (CH)	19.51 - 12.19	19.64	9.82	11	38.36

In early 1976, the HCFCD had excavated a 4.57 m [15.00 ft] deep ditch adjacent to the Addicks Dam, which intercepted the sand layer of the dams foundation. This created a large seepage problem through the dams followed by emergency modifications to avoid this seepage. For the reaches in the Addicks and Barker Dam which have a ditch adjacent to one of the dams, three measures are taken against seepage. These measures are shown in Figure 5.2 and were completed in 1982. First, bentonite slurry trench seepage barriers were installed from the crest level into the relatively impermeable clay layer for approximately 1.52 m [5.00 ft] deep. Secondly, earthen stability berms were placed on the outer slope and at last a clay cover was placed on the inner slope of the dams [9]. These slurry walls are installed from STA 164+00 (A4) until STA 450+00 (A2) at Addicks Dam, so have a length of 8.72 km and from STA 499+00 (B2) until STA 600+00 (B1) at Barker Dam, so have a length of 3.08 km [1.91 mi]. This means that the slurry wall is installed in 11.80 km [7.33 mi], which is only a part of the 40.56 km [25.20] total length [5] [50].

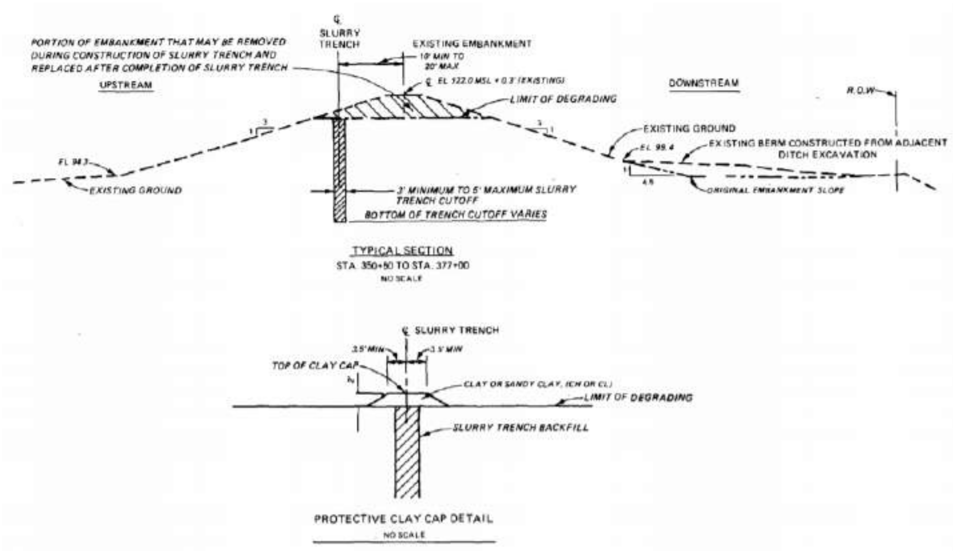


Figure 5.2: Addicks Dams remedial slurry trench for embankment and foundation seepage control [50]

Nowadays, there still is a ditch parallel to some of the dam sections, but little information is known about this ditch. The elevations of the ditch and the adjacent land is taken from the Lidar2008 in ArcGIS. There is a probability of water staying in the ditch, which gives a higher value of the bottom elevation. Because the ditch has a small depth and only the most critical section is taken, this error will be negligible.
Since the built-up of the Addicks Dam is comparable in every section, sections of the dams without slurry wall need to be checked on stability and piping using the soil composition shown in Figure 5.1. For the soil underlaying the Barker Dam, no cross-sections or Soil Penetration Tests are available. Because of the small distance between the two reservoirs and the same construction method of the dams, the assumption is made to use the same soil composition for both Addicks and Barker Dam to check the failure mechanisms. For each dam section the typical cross-section is given in Appendix F.

The ground water level varies over the years, mostly depending on the high precipitation or floods. Looking at the well near the Addicks Dam over the last ten years, the water table had a level between 1.52 m [5.00 ft] - 5.49 m [18.00 ft] below natural ground (elevation of 94.00 ft) with an extreme value of 0.91 m [3.00 ft] during Hurricane Harvey. According to the Water Control Manual (2012), the water table along Barker Dam can even drop until 7.62 m [25.00 ft] below natural grounds [11].

The area has suffered from large regional subsidence of the land due to groundwater extraction from the Gulf Coast aquifer and the compression of the clayey layers. The Harris-Galveston Subsidence District provided regulation of this groundwater withdrawal and predicts that, with the current regulation, the land-surface subsidence will be approximately 0.61 m [2.00 ft] in the area of the reservoirs in the period between 2010 till 2050 [2]. Since this land subsidence affects the dams and the surrounding areas, it is not taken into account by the investigation of the failure mechanisms; the weight of the water in the reservoirs could increase the effect of subsidence, but the reservoirs are most of the time empty so this effect is assumed negligible.

5.2. Failure mechanisms

The most probable failure mechanism for the dams need to be determined, which will be investigated individually according to a Dutch approach on levee design. This approach includes using the formulas and norms used in the Netherlands for an earthen embankment for each of the failure mechanisms. Appendix G gives a schematic overview of the most relevant failure mechanisms for this dam with a short description, which are overflow, overtopping, stability and piping. The spillways are not taken into account in the calculations of these failure mechanism, therefore the height of the spillway is considered to be the same height as the lowest crest level of the dam.

5.2.1. Overflow

If the still water level is higher than the crest level of the dam, the term *overflow* is used. The water can flow over the crest into the protected area, where the discharge itself can cause flooding. Even worse, the overflow can cause infiltration and erosion at the inner slope, which can eventually lead to a breach. In this case, overflow of the dams are prevented by the spillways which have a lower elevation than the crest level of the dams. As mentioned before, this is not taken into account in the calculation of the critical water level.

5.2.2. Wave overtopping and run-up

If the still water level is lower than the crest level of the dam, but the waves run over the crest, the term *overtopping* is used. This is normally expressed in a critical discharge to prevent the inner slope from erosion, this critical discharge together with the used formulas can be found in Appendix G.2. In Tables 5.2 and 5.3, the critical water levels for run-up and overtopping per dam section can be found. As can be seen, the overall critical water level for Addicks is 35.15 m [115.32 ft] and for Barker 32.80 m [107.61 ft].

Table 5.2: Maximum wave run-up per dam section incl. wave set-up

Wave run-up	**A1**	**A2**	**A3**	**A4**	**B1**	**B2**	**B3**	**B4**	**B5**
Crest level [m]	35.66	35.66	35.66	36.58	33.83	34.14	34.14	33.83	33.83
Critical water level [m]	35.39	35.40	35.39	36.31	33.11	33.42	33.44	33.83	33.83
Required free board [m]	0.28	0.27	0.27	0.26	0.738	0.72	0.7	0.71	0.72

Table 5.3: Maximum wave overtopping per dam section incl. wave set-up

Wave overtopping	**A1**	**A2**	**A3**	**A4**	**B1**	**B2**	**B3**	**B4**	**B5**
Crest level [m]	35.66	35.66	35.66	36.58	33.83	34.14	34.14	33.83	33.83
Critical water level [m]	35.15	35.18	35.18	36.23	32.80	33.10	33.21	32.90	32.82
Required free board [m]	0.51	0.48	0.48	0.32	1.03	1.04	0.93	0.94	1.02

5.2.3. Stability

Stability includes both *inner* as *outer slope failure* and *horizontal shearing.* Due to changes in hydrological condition, the embankment can become unstable and sliding planes occur, which lead to failure of the dam.

Inner and outer slope failure

For both inner and outer slope failure, there are two extreme situations: one situation in which the dam is completely saturated and one in which the dam is completely dry. For inner slope failure the chance of failing in saturated conditions is rare, because most of the time the water creates a counter momentum against sliding of the soil. Only when the water retreats fast while the dam is still saturated, a slip plane can appear.

The calculations for macro stability for both the inner and outer slope are done according Bishop with the program D-Geo stability. More information about the calculation and D-Geo stability can be found in Appendix G.3 [16]. In Table 5.4, the results from the calculations can be found as safety factors for the critical sliding planes per dam. It can be seen that dam section A1 was not considered, because it only consists out of spillway. This spillway is made of concrete and therefore it is unlikely to have problems with macro stability. Section A4 has the most critical sliding plane for the inner slope with a safety factor of 1.81 and section B2 has the most critical sliding plane for the outer slope with a safety factor of 2.16. Both factors are well above one and therefore considered to be safe. This means that the dams will not fail due to macro stability.

Table 5.4: Safety factors macro stability inner- and outerslope

Macro stability	A1	A2	A3	A4	B1	B2	B3	B4	B5
Inner slope dry	-	1.86	1.86	1.81	2.06	2.15	2.15	2.06	2.06
Inner slope saturated	-	2.54	2.53	2.46	2.82	2.92	2.92	2.82	2.82
Outer slope dry	-	3.10	2.61	2.40	3.28	2.16	3.11	3.13	3.56
Outer slope saturated	-	4.33	3.51	3.25	5.36	2.96	4.18	4.24	4.81

Horizontal sliding

If the horizontal pressure of the water inside the reservoir becomes higher than the maximum possible shear stress of the base of the levee, the base can *shear* towards the protected area. The shear depends on the cohesion, the angle of internal friction and effective strength according to Formula G.18 of Coulomb 1776. The latter of these three is caused by the weight of the levee itself. As there are three different soil layers inside the levee, shearing will be checked at the base of the levee and at the interfaces of layer V and IV, and of layer IV and III. Since the dams are not made out of a very light material, such as peat, it is not probable the dam will fail due to horizontal sliding or shearing. Therefore the choice is made to calculate the factors of safety for the different dike sections in G.3. Dividing the horizontal force by the shear force gives a safety factor of maximum 0.19 when the water level is as high as the crest of the dam, so the dams are safe against horizontal shearing.

5.2.4. Piping

Piping, or *Backward Internal Erosion*, is the process of forming channels in the subsoil of the dam due to high hydraulic gradients. If these 'pipes' grow underneath the whole cross-section, the dam can collapse or slide. The dams itself are considered as clay levees with underneath a layer of silty sand. Since silty clay is semi-permeable, this will behave as aquifer, so water can seep underneath the levee through this layer. Piping becomes a problem if there is a combination of uplift, heave and backward internal erosion, so if it expands to a continuous pipe and the aquifer will collapse.

The piping calculation is made on the two most vulnerable locations: the toe, because of the shortest distance and the ditch, because of the smallest thickness of the aquitard. For the latest of the two, the ditch reaches the aquifer. This means only piping needs to be checked, since there is already a direct connection between the water in the reservoir and the water in the ditch. For the three main phases, the critical heights are shown in Tables 5.5 and 5.6, how it is determined is explained in Appendix G.4.

Table 5.5: Piping at the toe per dam section

Critical elevations [m]	A1	A2	A3	A4	B1	B2	B3	B4	B5
Crest level	35.66	35.66	35.66	36.58	33.83	34.14	34.14	33.83	33.83
Critical water level Uplift	35.31	32.22	30.15	30.15	32.22	30.15	30.84	30.49	30.84
Critical water level Heave	32.82	30.74	29.36	29.36	30.74	29.36	29.82	29.59	29.82
Critical water level Sellmeijer	-	-	-	-	-	-	-	-	-
Critical water level Bligh	31.60	31.60	31.60	31.91	30.99	31.09	31.09	30.99	30.99
Critical water level Lane	32.90	32.90	32.90	31.11	32.58	32.65	32.65	32.58	32.58

Table 5.6: Piping at the ditch per dam section

Critical elevations [m]	A1	A2	A3	A4	B1	B2	B3	B4	B5
Crest level	35.66	35.66	35.66	36.58	33.83	34.14	34.14	33.83	33.83
Critical water level Sellmeijer	-	-	-	-	-	-	-	-	-
Critical water level Bligh	-	36.60	33.07	33.57	33.38	32.99	33.43	33.33	33.38
Critical water level Lane	-	36.42	33.94	34.28	34.27	33.99	34.30	34.23	34.27

Uplift and heave

Uplift is the process of rupture of the aquitard due to a higher pore pressure in the aquifer than the weight of the blanket. If erosion of the silty sand layer occurs, it is called *heave*. These checks are made in Appendix G.4 by calculating the critical water level for both uplift and heave. The critical water levels for these phases of piping are lower than the crest levels for all the dam sections, so uplift and heave are not an issue at the toe.

Backward internal erosion

If the head difference cannot be resisted by the erosion resistance of the sand grains in a partially developed piping channel, it is called *backward internal erosion*, which can grow until a continuous pipe is formed and the aquifer collapses. This is checked with the formulas of Bligh (1910), Lane (1935) and Sellmeijer (1988). The theory of Blight and Lane are both focusing on the lane of creep, where Blight only have taken the horizontal distance and Lane also takes the vertical distance into account. For Sellmeijers formula, more detailed information on the grains and voids are needed. The soil composition is known for the dike sections, but since the exact information of the existing soils are not available, standardized values are taken for those properties. Nevertheless, the critical water levels are unrealistic high compared with the water levels of Bligh and van Lane, so are left out for the further research.

Piping will only occur if uplift, heave and backward internal erosion occur together which is the case for piping at the toe according to Bligh and Lane. Because of the many assumptions made on the soil characteristics, the most conservative elevation of Bligh is chosen and compare with uplift and heave. At the toe this gives a critical water level of 31.60 m [103.67 ft] for Addicks Reservoir and 30.99 m [101.67 ft] for Barker Reservoir and at the ditch is this 33.07 m [108.50 ft] for Addicks Reservoir and 32.99 m [108.23 ft] for Barker Reservoir.

5.2.5. Special structures

The majority of the dams consists of an earthen structure, but there are a few exceptions, such as the outlet work structures and spillways. Because of the concrete parts and the potential consequence to the Houston metropolitan area, the Addicks and Barker dams are designed by the USACE as "extremely high risk" [8]. Since there are no structural drawings of the spillways and outlet structures available, this failure mechanisms are only elaborated but not calculated in this report.

Outlet work structures

The outlet work structures are concrete conduits which allow the water to discharge from the reservoir to the Buffalo Bayou. If the hydraulic pressure inside of the conduits becomes higher than the surrounded soil pressure, fractures can occur in the soil. This is the case when there is too much compaction energy beneath the pipe or when there is poor compaction. These fractures cause erosion along the conduits and can develop as voids which direct water from downstream to upstream of the reservoir through the dam until the process of backward internal erosion has caused a pipe which collapses. To prevent this, a filter collar or chimney filter zone can be installed which interrupts the flow along the conduit [19].

Spillways

The spillway embankments are covered with RCC slabs, which could fail in the following manner: uplift of the revetment or erosion around the transition zone. During high water, the water will saturate the embankment and cause a water pressure upwards underneath the slabs. The slabs are impermeable so cannot release this water and needs to withstand these force. If the water level becomes even higher, an extra drag force due to the flow velocity will work on the slabs. If the combination of these two forces becomes higher than the weight of the slabs, the revetment can be lifted up from the embankment and washed away. When water is flowing over the remaining part of the dam, eroding will start start quickly until a breach occurs.

The second way the dam can fail around the spillways is at the transition zone between the concrete and the grass cover or the concrete and the natural ground. Due to the difference in revetment, from a soft permeable to a hard impermeable type of revetment, the flow of the water will be concentrated along the concrete edges and can cause erosion at this location. The transition from concrete to grass can be seen as a smooth transition, in contrast to the transition from concrete to the natural ground where is an abrupt difference in height. As a general rule, the smoother the transition from one to the other revetment, the more the chance of failure will be reduced. This gradual transition can be settled by placing open concrete blocks where grass can be grown through.

5.3. Conclusion

The maximum water level that the dams can manage differs per failure mechanism, see Table 5.7. Since the critical water elevation for stability is higher than the crest elevation, those water elevations are not included in the table. The slurry wall in the Addicks dam is installed from STA 164+00 till STA 450+00, so in part of the section there is no slurry wall. For the Addicks Dam is this only the 3.03 km [1.88 mi] long section A2 from STA 450+00 till 550+00. For A4 there is no slurry wall from the beginning of the dam till STA 164+00 and in this part there is no ditch, so only piping at the toe is considered. For Barker Dam a length of 13.79 km [8.57 mi] of the total 21.92 km [13.62 mi] is remaining, since the ditch is only absent in section B5 for 5.05 km [3.14 mi] and the slurry wall has a length of 3.08 km [1.91 mi]. Since the slurry wall is placed in only a part of B2, all the sections will be taken into consideration [50],[5].

Table 5.7: Critical water level per failure mechanism per dike section [m]

	A1	**A2**	**A3**	**A4**	**B1**	**B2**	**B3**	**B4**	**B5**
Overtopping	**35.15**	35.18	35.18	36.23	**32.80**	33.10	33.21	32.90	32.82
Stability	-	-	-	-	-	-	-	-	
Piping at the toe	35.31	32.22	-	**31.91**	32.22	31.09	31.09	**30.99**	**30.99**
Piping at the ditch	-	**36.42**	-	-	33.38	**32.99**	33.43	33.33	33.83

The problem is for the ditch is the connection of the inside water with the outer water through the silty sand layer, so solutions which add to the effective stresses, for example berms, or release pressure in the aquitard such as relief wells, can be disregarded for the parts where only piping through the ditch is observed. This "open" connection can be fixed by extending the seepage length or block the seepage entirely by the following mitigations, which need to be applied only in the sections of the dams which have a ditch, but no slurry wall is already installed. If mitigations against piping are taken, the critical water level is 3.24 m [10.63 ft] higher for the Addicks Reservoir and 1.81 m [5.94 ft]

- *Seepage wall*
  As used in the past, placing a seepage or cut-off wall from the crest level through the silty sand layer into the impermeable layer underneath, block the seepage entirely. Assuming the aquifer infinitively deep, blocking the aquifer of the Addicks Dam until an elevation of 17.77 m [58.30 ft], the critical water level is the same as for overtopping. For Barker Dam, this is 17.31 m [56.79 ft].
- *Granular filter*
  To prevent erosion of the aquifer material, a granular filter or geomembrane is a solution, but this does not prevent water to seep through the aquifer. This filter can be placed vertically at the outer slope of the dam to close off the impermeable layer. Another location for this filter is to cover up the bottom of the ditch with the granular filter.
- *Closing the ditch*
  By closing of the ditch, the length of the hinterland becomes significantly longer. A reasonable function of the ditch is to discharge the overflow from the spillways to the Buffalo Bayou. If the choice is made to preserve this ditch, this can be done by placing a impermeable blanket which is prevented by uplift and heave by its own weight to cover the aquifer for 6.5 million dollar. Another option is to create a concrete flume for the discharge.
- *Impermeable layer at the inner side of the dam*
  To extend the seepage length at the inner side of the dam, can be seen as creating a foreshore. A thin layer of impermeable material needs to be placed on top of the silty sand layer inside the reservoir to prevent the water from infiltrating into the aquifer.

For the parts of the dams where piping can occur at the toe of the dams, heave and uplift are a problem as well and therefore the following mitigation can also be taken to prevent piping.

- *Berm at the out side of the dam*
  By constructing an impermeable berm on the out side of the dam the weight of the soil layer increases. If this weight is larger than the pressure from the water under this soil layer in the aquifer, uplift and therefore heave will be prevented.

- *Relief wells*
  Normally a relief well is installed close to the dam. Relief wells are vertically inserted by drilling techniques till they permeated the aquifer. The relief well makes sure the water pressure is reduced, so uplift and therefor heave are less likely to occur.

6

Minimize flooding damage

Both upstream and downstream of the reservoirs are developed areas, which are prone to flooding as a consequence of operation of the reservoirs. If the combined outflow of the reservoirs exceeds a certain amount, the houses downstream of the reservoirs will be flooded. On the other hand, when more water is retained in the reservoirs, the water level of the reservoir raises and houses at the upstream side of the reservoirs will be flooded. In this sub question, an optimization to minimize the damage will be made between these two options in case of an extreme rain event based on the design events determined before. The following question is answered in this chapter:

What is the optimized ratio between the retained volume in the reservoirs and released discharge into the Buffalo Bayou to minimize the damage due to flooding?

6.1. Scenarios

Currently, the aim of the USACE is to not open the outlets during an extreme event. However, when too much water comes into the reservoirs by the inflow of the creeks and the precipitation, three different situations could occur:

1. *Flooding of upstream area*
   To prevent the downstream area from flooding, a limited amount of water can be discharged into the Buffalo Bayou. If more water is entering the reservoirs than is discharged through the outlets, the water elevation in the reservoirs becomes higher. The developed areas upstream from the reservoirs in the Addicks and Barker watershed will flood. In this situation, there is only damage in the upstream area and the downstream area is saved. The floodplains of the area upstream of the reservoirs are comparable with the elevation of the area, because the water is retained by the reservoirs.

2. *Flooding of downstream area*
   Until an elevation of 31.39 m [103.00 ft] in Addicks Reservoir and 28.96 m [95.00 ft] in Barker Reservoir, the water only floods governmental land [5]. To prevent the upstream area from flooding, the excess of water can be discharged into the Buffalo Bayou. However, if too much water is discharged through the outlets to prevent the upstream area from flooding, the Buffalo Bayou bursts its banks and floods houses along the river. This situation saves the upstream area, but causes damage in the downstream area.

3. *Breaching of the dams*
   If too many water is retained in the reservoirs to prevent flooding of downstream area, the hydraulic forces on the dams can be too much and cause a breach. Breaching is an uncontrolled situation where even more damage could occur since areas can be flooded which normally are not in the floodplain.

In this report only the first two situations are considered in the optimization of damage. However, the water level elevation before the dams will fail as calculated in Chapter 5.3 cannot be exceeded to prevent the third option of breaching of the dams.

6.2. Mapping of damage

In this part, the damages upstream and downstream of Addicks and Barker Reservoir are determined in order to optimize the damage estimates. Only the residential properties will be considered for the first damage estimation. Therefore the amount of houses and its values have to be analyzed in the area around the reservoirs. The costs of the total damage have to be estimated following the depth of inundation. The areas around the Addicks and Barker Reservoirs and the Buffalo Bayou are divided into sub areas, see Appendix H.3. These sub areas are chosen in such a way they correspond with sub areas that are given in maps used later in this chapter.

The surface area of the sub areas are measured using ArcGIS. Per sub area, the amount of houses is estimated using Formula 6.1. Combining the amount of houses with the median residential property value will give a total value for all the residential properties per sub area. The information about population density, household size and residential property values of 2017 are gathered from maps made by Esri and can be opened in ArcGIS Online[18]. These maps give per sub area a minimum, an average and a maximum value, therefore the results are given in a range. From Figure H.5, the total damage (in percentage of property+content value) of a residential property per feet inundation can be found. The graphs will show a maximum inundation of 9 feet, for inundation more than 9 feet the USACE states that the damage will no further increase and top out[43]. In Figure H.6, it is depicted that there is almost no difference in damage for houses with or without basement and therefore only the graph without basement is used. The content value of a residential property is considered to be 50% of the residential property value, since this is the estimated value that is used by the USACE [43][44]. The residential properties are considered to consist out of one story houses for 55% and two story houses for 45% in Texas[36].

$$Amount\,of\,houses\,per\,area = \frac{Population\,Density\,[persons/mi^2] * Surface\,Area\,[mi^2]}{Household\,Size\,[persons]} \tag{6.1}$$

Upstream, the above information is combined with the elevation maps which are made using the elevation contour maps in ArcGIS.The elevation contour maps have a 5 feet accuracy and are provided by the Houston Galveston Area Council[14]. The elevation maps are depicted in Figures 6.1, 6.2 and 6.3.

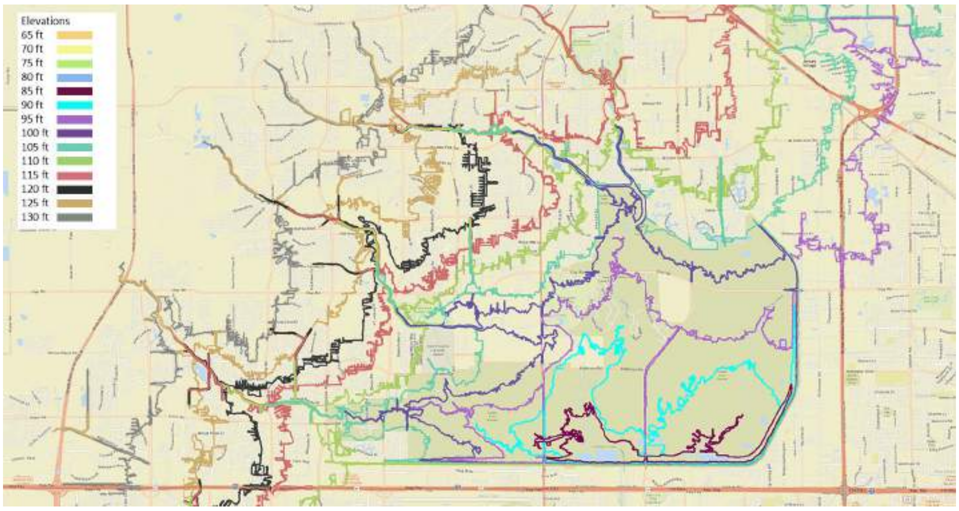


Figure 6.1: Elevation map Addicks

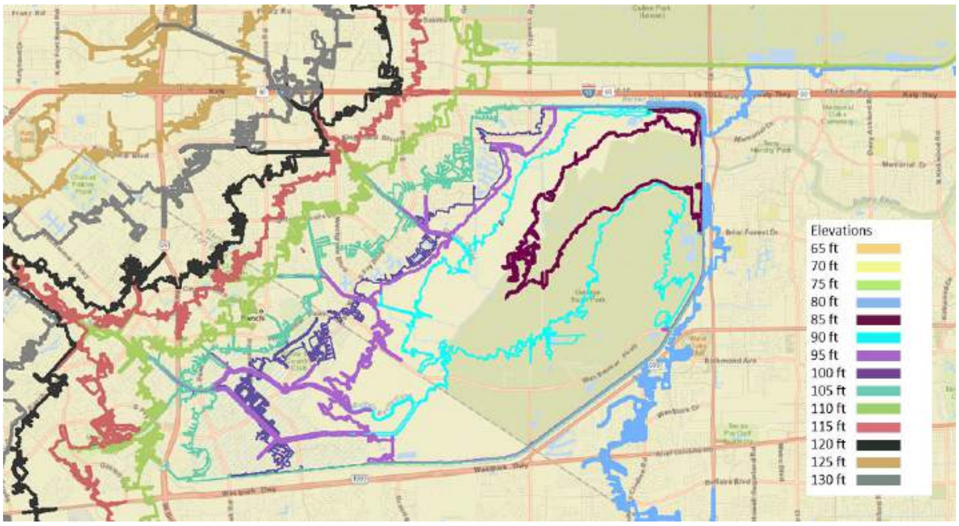


Figure 6.2: Elevation map Barker

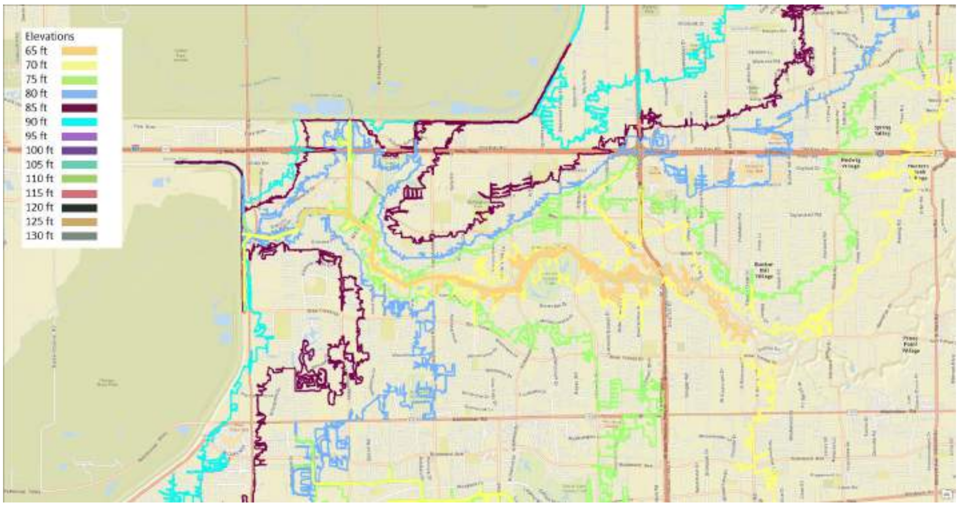


Figure 6.3: Elevation map Buffalo Bayou

To indicate the damages downstream, the floodplains are used, see Figure 6.4. The downstream area is considered until the location where the Buffalo Bayou merges with the White Oak Bayou in downtown Houston. The downstream floodplains depend on the elevation and gradient of both the land around and water in the Buffalo Bayou. With the Model and Map Management (M3) system of the HCFCD ,the effective floodplains are modeled in HEC-RAS [32]. In this 1D-model, different cross-sections along the stream can be drawn, each with its own elevation, gradient and roughness coefficient. By adding a discharge through the river, the water level at different locations of the stream can be determined. The floodplains which belong to these discharges can be mapped. Two floodplains are already available for the area around the Buffalo Bayou. The first one is the baseline probability (100-year or 1%-annual-change floodplain), this is called Special Flood Hazard Area (SFHA) and is based on the 1%-annual-change flood discharge. The second floodplain is the 500-year floodplain and is mapped based on the 0.2%-annual-change flood discharge. Both of these discharges consist of the run-off in the watershed.

In the optimization, the Addicks and Barker Reservoirs can release water in the Buffalo Bayou, so an even higher discharge could occur. The higher discharge is used as input for the model to compose another, larger floodplain than the already composed 100-year and 500-year floodplains. Due to the limitations of HEC-RAS, there are some errors running this larger discharge through the Buffalo Bayou. In Appendix H.1, these limitations are described. Because of the limitations, these floodplains are probably underestimated, but it helped giving a first estimate and it increases the accuracy of the damage calculation with the 100- and 500-year floodplain.

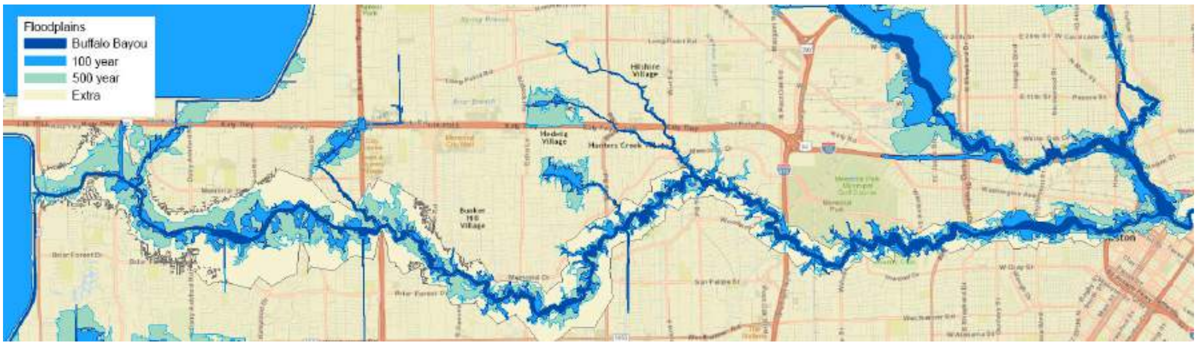


Figure 6.4: Floodplains Buffalo Bayou (Adjusted illustration from Esri[18])

6.3. Damage optimization

The damage due to an extreme event depends on the distribution of the total volume of water V_{in} from precipitation and inflow due to an extreme event and the discharge Q into the Buffalo Bayou. In the optimal situation, the combined damage in the upstream area of Addicks $\$_A$, the upstream area of Barker $\$_B$ and the downstream area along the Buffalo Bayou $\$_d$ is at its minimum. Damage will only occur if there is more incoming volume V_{in} than can be retained in governmental land V_{gov}. The ratio x determines which part of the total excess volume will remain upstream ΔV_u and the excess volume which will be discharged downstream ΔV_d. If this ratio is 1, all the excess volume remains upstream, when this ratio is 0, the volume will be discharged downstream. These ratios are the unknown parameters in this optimization and are calculated per return period of the event per duration of the precipitation. Only the higher durations of 12, 24, 48, 72 and 96 h are considered, because there is only an excess of volume during these durations.

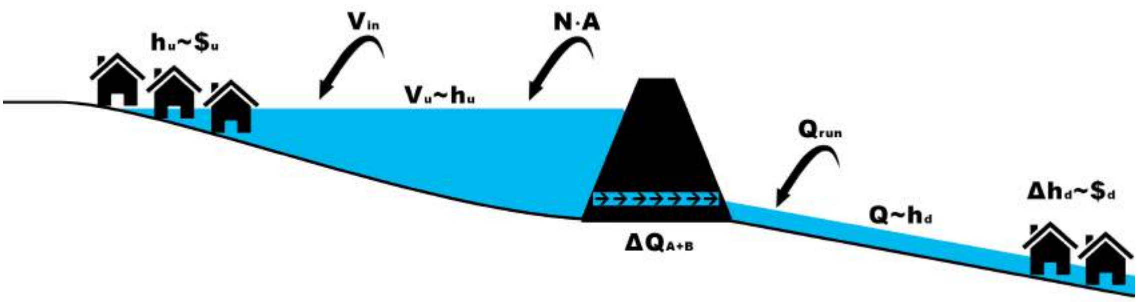


Figure 6.5: Optimization method

The following assumptions are made in the optimization model:

- The Barker Reservoir is directly discharging into the Buffalo Bayou, while the Addicks Reservoir discharges into the Langham Creek before it flows into the Buffalo Bayou. Only the Buffalo Bayou after the confluence is described by the model.
- The Buffalo Bayou is modeled with a uniform cross-section and gradient, so in this model there is a uniform discharge. The gradient of the water surface is equal to the gradient of the riverbed, so uniform as well.
- A constant discharge is assumed to be released through the outlets into the Buffalo Bayou during the duration of the precipitation.

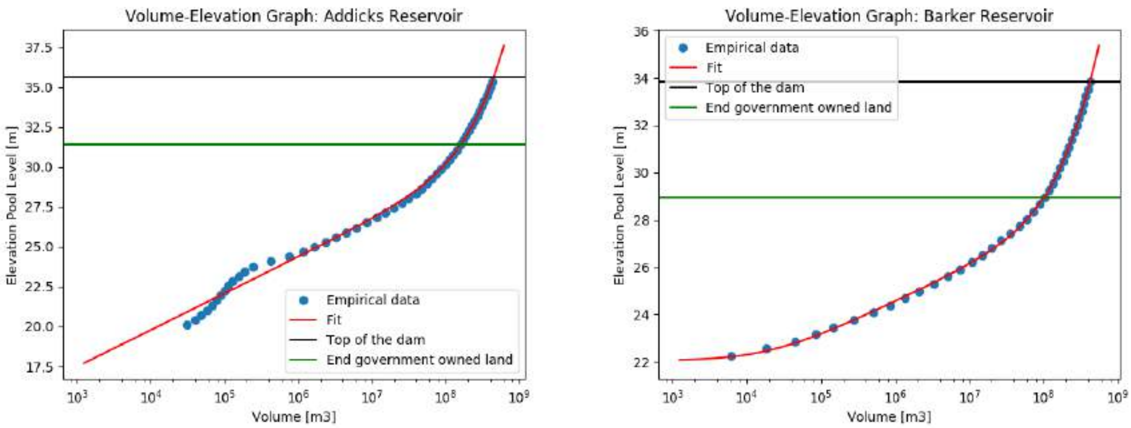
- The discharge of the Turkey Creek is a result of the runoff in the Buffalo Bayou watershed, so is direct dependent of the precipitation in this watershed. The precipitation per return period per duration determined in Chapter 4.3 is assumed to be uniform with the precipitation in the Buffalo Bayou since this is measured at the gage with a negligible distance to the watershed.

Steps optimization:

1. *Volume ~ Discharge (V ~ Q)*
   When the volume V_{in} (from precipitation and inflow) is more than the reservoirs can retain without exceeding the governmental area, a certain volume ΔV will spread over the non-governmental area upstream ΔV_u or needs to be discharged into Buffalo Bayou downstream ΔV_d. This incoming volume is determined by the volume per return period of the event per duration.

$$\begin{aligned} \Delta V &= \Delta V_u + \Delta V_d \\ \Delta V &= V_{in} - V_{gov} \end{aligned} \quad \begin{cases} \Delta V_u = x * \Delta V \\ V_u = V_{gov} + \Delta V_u \\ \\ \Delta V_d = (1-x) * \Delta V \\ \Delta Q = \Delta V_d / \Delta t \end{cases} \quad \begin{cases} V_u = V_{gov} + x * (V_{in} - V_{gov}) \\ \\ \Delta Q = \frac{(1-x)*(V_{in}-V_{gov})}{\Delta t} \end{cases} \tag{6.2}$$

2. *Volume upstream ~ Water level upstream (V_u ~ h_u)*
   The relations between the total volume upstream V_u of the reservoirs and the upstream water levels h_u, which are calculated in Appendix E.4, are used in this step of the optimization.



(a) Addicks

(b) Barker

Figure 6.6: Volume-Elevation graph

3. *Discharge ~ Water level downstream (Q ~ h_d)*
   The ratio between the excess of volume upstream ΔV_u and the volume discharged downstream ΔV_d, results in a discharge per reservoir. Combined with the discharge from the run-off Q_{run}, gives the total discharge Q, which results in a water level downstream h_d of the reservoirs of the Buffalo Bayou. The run-off is derived per duration and per return period in Appendix H.4, the optimum discharge into the Buffalo Bayou can be determined per return period per duration. The relation between the discharge and the water level can be seen in Figure 6.7 and how it was made can be found in Appendix H.4 as well. From a discharge of 116.11 m^3 [4,100 cfs] and higher at Piney Point, there is damage to residential property [4]. This discharge is comparable with the 10-percent-annual-flood-discharge [20], which causes a height at the used STA of 19.89 m [65.24 ft].

$$Q = \Delta Q_A + \Delta Q_B + Q_{run} \tag{6.3}$$

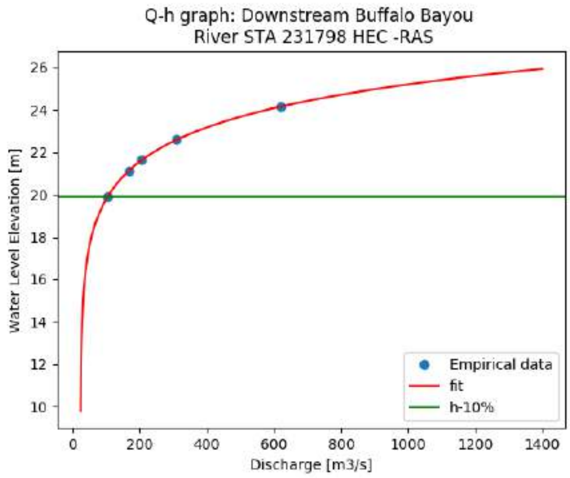


Figure 6.7: Q~h_d

4. *Water level ~ damage (h ~ \$)*
The ratio between the volumes which belongs to the minimum damage costs can be found combining the relation between the water level both upstream as downstream with the damage costs. Upstream damage corresponds directly with the elevation height. Downstream damage occurs above the $h_{10\%}$, see Figure 6.7, so a water level elevation increases Δh_d around the Buffalo Bayou. This is used for the downstream damage. The damage from the urban planning part is used to fit a relation between the water level elevation and the corresponding damage due to flooding. This is done for the minimum, average and maximum flood damage.

$$\Delta h_d = h_d - h_{10\%} \tag{6.4}$$

$$min(\$_{total}) = min(\$_A + \$_B + \$_d) \tag{6.5}$$

6.4. Results

To minimize the total damage, it is clear that Addicks should be the leading reservoir for determining the ratio of the volume retaining upstream and discharging downstream to minimize the total damage. These damages are shown in Table 6.1 and are lower for every event compared to the situation when Barker is dominant, see Table H.1 in Appendix H.5. All the events highlighted in gray are the events where Addicks and/or Barker Reservoir are discharging into the Buffalo Bayou. In the other events, the discharge only consists of run-off which already gives damage downstream.

Table 6.1: Totale damage (million $), Addicks Reservoir dominant

	12	**24**	**48**	**72**	**96**
10	231.29	1.12	0.00	0.00	0.00
	1,002.09	1.68	0.00	0.00	0.00
	2,182.12	2.24	0.00	0.00	0.00
25	496.52	1.99	0.00	0.00	0.00
	2,181.83	2.98	0.00	0.00	0.00
	4,768.44	3.97	0.00	0.00	0.00
50	693.28	73.65	0.50	0.00	0.00
	3,064.46	317.33	0.75	0.00	0.00
	6,708.31	691.79	1.00	0.00	0.00
100	878.10	163.23	1.03	0.00	108.95
	3,896.63	703.27	1.54	0.00	505.64
	8,539.34	1,529.65	2.05	0.00	699.79
250	1,101.00	309.13	211.51	408.69	483.99
	4,903.04	1,346.46	1,063.88	1,954.89	2,203.92
	10,755.60	2,935.85	2,480.89	3,679.25	3,983.24
500	1,253.41	427.17	475.93	688.72	827.28
	5,592.52	1,871.91	2,413.91	3,411.62	3,988.14
	12,274.90	4,088.04	4,485.37	6,191.12	6,986.63
1,000	1,392.60	753.00	767.36	1,097.03	1,331.37
	6,223.07	3,450.93	3,963.91	5,791.37	6,891.27
	13,664.80	7,716.44	7,482.19	10,834.90	12,436.10
10,000	2,084.26	1,780.97	2,531.95	3,341.30	4,095.37
	9,580.01	8,903.43	14,263.20	17,372.40	21,066.50
	20,785.70	18,343.20	29,180.70	33,429.20	38,539.30

As shown in Tables 6.2 and 6.3, run-off has a big influence on the discharge downstream in the Buffalo Bayou, especially for the 12 hours and 24 hours precipitation. The Addicks and Barker Reservoirs should not release water during these events (not the gray highlighted values) and store the run-off from the upstream area. In this way, they have a positive influence on the flooding problem of the downstream area, so fulfill their function of preventing Houston from flooding.

When the events have a longer duration, it would cause less damage to open the outlets of the reservoirs than to keep them closed. Especially for the events with a 72 and 96 hours duration and a return period from 100 years and up, a part of the excess of volume in the reservoirs should be discharged downstream to minimize the amount of damage. Only in the most extreme events, 10,000 year return period in combination with a 48 hour duration and up, all the volume should be discharged downstream. The ratios of all the considered events can be found in Table 6.4, for the case in which the Addicks Reservoir is dominant and the Barker Reservoir is dominant. A ratio of 1 means that the excess volume needs to be kept upstream and a ratio of 0 means that the Addicks and Barker reservoirs should release according to their maximum capacity. In the case the Addicks Reservoir is dominant, the damage upstream of the Addicks Reservoir in relation with the damage downstream determines the ratio for both of the reservoirs.

For the average and maximum values of the damages, the discharges and ratios are the same. However, for the minimum values the ratio is changed from 0 to 1 for the events with a return period of 10,000 year in combination with a 48, 72 and 96 hours duration. The corresponding discharges are 263.97, 177.30 and 166.78

m^3/s, the water level in the Addicks Reservoir is 38.10, 40.19, 40.42 m and the water level in the Barker Reservoir is 31.30, 32.86, 34.29 m and the increase in water level in the Buffalo Bayou is 2.32, 1.37 and 1.22 m.

Table 6.2: Run-off discharge [m^3/s]

	12	**24**	**48**	**72**	**96**
10	264.33	142.16	74.26	43.83	28.38
25	333.13	183.62	100.11	62.02	42.84
50	384.17	214.38	119.28	75.51	53.57
100	434.84	244.91	138.31	88.90	64.22
250	501.54	285.10	163.37	106.53	78.24
500	551.91	315.46	182.29	119.84	88.83
1,000	602.25	345.79	201.20	133.14	99.40
10,000	769.36	446.49	263.97	177.30	134.53

Table 6.3: Total discharge [m^3/s]

(a) Addicks Reservoir dominant

	12	**24**	**48**	**72**	**96**
10	264.33	142.16	74.26	43.83	28.38
25	333.13	183.62	100.11	62.02	42.84
50	384.17	214.38	119.28	75.51	53.57
100	434.84	244.91	138.31	103.63	108.42
250	501.54	285.10	184.05	183.15	183.21
500	551.91	315.46	182.29	179.18	182.69
1,000	602.25	345.79	201.20	176.93	178.98
10,000	769.36	446.49	3905.22	3849.41	3327.16

(b) Barker Reservoir dominant

	12	**24**	**48**	**72**	**96**
10	264.33	142.16	74.26	43.83	28.38
25	333.13	183.62	100.11	62.02	42.84
50	384.17	214.38	119.28	75.51	53.57
100	434.84	244.91	138.31	103.63	112.48
250	501.54	285.10	163.37	183.15	183.21
500	551.91	315.46	182.29	179.18	182.69
1,000	602.25	345.79	201.20	176.93	178.98
10,000	769.36	446.49	263.97	177.30	166.78

Table 6.4: Ratio [-]

(a) Addicks Reservoir dominant

	12	**24**	**48**	**72**	**96**
10	1.00	1.00	1.00	1.00	1.00
25	1.00	1.00	1.00	1.00	1.00
50	1.00	1.00	1.00	1.00	1.00
100	1.00	1.00	1.00	0.00	0.12
250	1.00	1.00	0.88	0.76	0.69
500	1.00	1.00	1.00	0.91	0.85
1,000	1.00	1.00	1.00	0.96	0.92
10,000	1.00	1.00	0.00	0.00	0.00

(b) Barker Reservoir dominant

	12	**24**	**48**	**72**	**96**
10	1.00	1.00	1.00	1.00	1.00
25	1.00	1.00	1.00	1.00	1.00
50	1.00	1.00	1.00	1.00	1.00
100	1.00	1.00	1.00	0.00	0.04
250	1.00	1.00	1.00	0.76	0.69
500	1.00	1.00	1.00	0.91	0.85
1,000	1.00	1.00	1.00	0.96	0.92
10,000	1.00	1.00	1.00	1.00	0.99

As mentioned before, the average and maximum values resulted in the same discharges, ratios and damages, Tables 6.5 until 6.8 are based on these values. The tables which correspond with the minimum values are shown in Appendix H.4. The water level per event is shown is Tables 6.5 and 6.6, where the orange parts show where the water level in the reservoir is higher than the critical water level determined in Chapter 5, so where piping can occur in the dams. This is especially a problem for the Addicks Dam at the larger return periods and durations. When mitigations against piping are taken and overtopping is the critical failure mechanism, only the events highlighted in red are still a problem. When the Barker Reservoir is dominant, the red highlighted elevations happen for durations with return period 10,000 years. However, when the Addicks Reservoir is dominant, these water levels only occur if the minimum damage is taken into account for the optimization, see Appendix H.4.
The advise would be to discharge the excess of volume downstream to secure the water level from not rising above the critical water level. The line is drawn at the location where the water in the reservoir exceeds the governmental land, so when it starts flooding houses upstream. This line is not drawn for the Addicks Reservoir in Figure 6.5, because these are the same as the orange highlighted events.

Table 6.5: Water level of Addicks Reservoir [m]

(a) Addicks - Addicks Reservoir dominant

	12	24	48	72	96
10	28.45	28.81	29.01	29.24	29.58
25	28.90	29.33	29.92	30.18	30.31
50	29.23	29.73	30.44	30.77	30.91
100	29.57	30.15	31.01	31.37	31.37
250	30.02	30.75	31.89	32.22	32.27
500	30.39	31.25	32.68	33.26	33.32
1,000	30.79	31.81	33.61	34.46	34.55
10,000	32.44	34.32	31.37	31.37	31.37

(b) Addicks - Barker Reservoir dominant

	12	24	48	72	96
10	28.45	28.81	29.01	29.24	29.58
25	28.90	29.33	29.92	30.18	30.31
50	29.23	29.73	30.44	30.77	30.91
100	29.57	30.15	31.01	31.37	31.37
250	30.02	30.75	31.89	32.22	32.27
500	30.39	31.25	32.68	33.26	33.32
1,000	30.79	31.81	33.61	34.46	34.55
10,000	32.44	34.32	38.10	40.19	40.42

Table 6.6: Water level of the Barker Reservoir [m]

(a) Barker - Addicks Reservoir dominant

	12	24	48	72	96
10	26.79	27.07	27.43	27.61	27.75
25	27.02	27.36	27.81	28.06	28.25
50	27.19	27.57	28.10	28.42	28.67
100	27.34	27.78	28.41	28.81	28.96
250	27.54	28.07	28.86	29.23	29.53
500	27.69	28.29	29.16	29.75	30.20
1,000	27.85	28.52	29.57	30.34	30.97
10,000	28.37	29.35	28.95	28.95	28.95

(b) Barker - Barker Reservoir dominant

	12	24	48	72	96
10	26.79	27.07	27.43	27.61	27.75
25	27.02	27.36	27.81	28.06	28.25
50	27.19	27.57	28.10	28.42	28.67
100	27.34	27.78	28.41	28.81	28.96
250	27.54	28.07	28.86	29.23	29.53
500	27.69	28.29	29.16	29.75	30.20
1,000	27.85	28.52	29.57	30.34	30.97
10,000	28.37	29.35	31.30	32.86	34.29

Table 6.7: Increase of water level in the Buffalo Bayou [m]

(a) Addicks Reservoir dominant

	12	24	48	72	96
10	2.33	0.82	0.00	0.00	0.00
25	2.86	1.46	0.00	0.00	0.00
50	3.19	1.83	0.37	0.00	0.00
100	3.47	2.15	0.75	0.00	0.11
250	3.79	2.50	1.46	1.45	1.45
500	4.00	2.74	1.44	1.40	1.45
1,000	4.20	2.95	1.68	1.37	1.40
10,000	4.74	3.53	8.25	8.22	7.91

(b) Barker Reservoir dominant

	12	24	48	72	96
10	2.33	0.82	0.00	0.00	0.00
25	2.86	1.46	0.00	0.00	0.00
50	3.19	1.83	0.37	0.00	0.00
100	3.47	2.15	0.75	0.00	0.21
250	3.79	2.50	1.17	1.45	1.45
500	4.00	2.74	1.44	1.40	1.45
1000	4.20	2.95	1.68	1.37	1.40
10000	4.74	3.53	2.32	1.37	1.22

The ratio which corresponds the total damage, differs from the ratio of the amount of people that are affected by the flood, this ratio is shown in Table 6.8 with the absolute numbers. This ratio is the same for the situation when the Addicks Reservoir is dominant as when the Barker Reservoir is dominant, so only one is shown. All gray ratios are larger than 0.50 which means that more water is kept upstream, so it is clear that during the big events, more people upstream will be affected by the the flood when the optimization is based on the damage minimization.

Table 6.8: Upstream people affected by the flood: Addicks Reservoir dominant

(a) Amount of people

	12	**24**	**48**	**72**	**96**
10	0	0	0	0	0
	0	0	0	0	0
	0	0	0	0	0
25	0	0	6,713	6,713	6,713
	0	0	18,587	18,587	18,587
	0	0	60,924	60,924	60,924
50	0	0	6,713	14,776	14,776
	0	0	18,587	41,070	41,070
	0	0	60,924	60,924	60,924
100	0	6,713	14,776	14,776	14,776
	0	18,587	41,070	41,070	41,070
	0	60,924	60,924	60,924	60,924
250	6,713	6,713	28,414	28,414	28,414
	18,587	18,587	80,916	80,916	80,916
	60,924	60,924	193,030	193,030	193,030
500	6,713	6,713	28,414	40,894	48,245
	18,587	18,587	80,916	118,895	141,606
	60,924	60,924	193,030	319,986	319,986
1000	6,713	28,414	40,894	64,430	64,430
	18,587	80,916	118,895	193,162	193,162
	60,924	193,030	319,986	493,842	493,842
10000	28,414	40,894	124,059	147,886	166,863
	80,916	118,895	353,461	370,285	370,285
	193,030	319,986	60,924	60,924	60,924

(b) Ratio

	12h	**24h**	**48h**	**72h**	**96h**
10	0.00	-	-	-	-
	0.00	-	-	-	-
	-	-	-	-	-
25	0.00	-	1.00	1.00	1.00
	0.00	-	1.00	1.00	1.00
	-	-	1.00	1.00	1.00
50	0.00	0.00	1.00	1.00	1.00
	0.00	0.00	1.00	1.00	1.00
	-	-	1.00	0.62	0.62
100	0.00	0.77	1.00	1.00	1.00
	0.00	0.76	1.00	1.00	1.00
	-	1.00	0.62	0.62	0.62
250	0.44	0.44	1.00	1.00	1.00
	0.42	0.42	1.00	1.00	1.00
	1.00	1.00	0.84	0.84	0.84
500	0.28	0.44	1.00	1.00	1.00
	0.25	0.42	1.00	1.00	1.00
	1.00	1.00	0.84	0.90	0.81
1000	0.28	0.77	0.95	1.00	1.00
	0.25	0.76	0.95	1.00	1.00
	1.00	0.84	0.90	0.87	0.87
10000	0.62	0.83	0.98	1.00	1.00
	0.60	0.82	0.87	0.87	0.87
	0.84	0.90	0.62	0.62	0.62

6.5. Mitigation

More mitigations on minimizing the damage around the reservoirs are possible. In this report, an optimization of the performance of the current Addicks and Barker Reservoirs is made without making chances to the boundary conditions. However, if changes of the future boundary conditions are taken into consideration, the following options can be designed:

- *Increase capacity Buffalo Bayou downstream*
  By increasing the capacity of the Buffalo Bayou, more volume can be discharged through the channel with a lower water level elevation. This can be done by increasing the cross-section of the mostly natural river by widening or deepening it, what could cause buy-outs of private properties along the bayou downstream. Also changes of the profile can cause other problems to the flow of the river, such as aggregation or degradation of sediment at several locations along the bayou. These changes needs to be modeled to see the overall effect over time.

- *Increase capacity of the reservoirs*
  By increasing the capacity of the reservoirs, more water can be retained before it needs to be discharged downstream. By heighten the dams or dredging the reservoir no buy-outs or relocation of homes has to happen upstream. If the choice is made to extend the dams and create a larger area, this can be a problem upstream. Also applying soil improvement, so more water can be infiltrated into the ground,

increases the capacity.
If the dike-ring will be closed and will not have a U-shape anymore, the retained water cannot uncontrolled flow out of the area as well. However, inlets need to be created at the upstream side to let the run-off from the upstream area enter the reservoirs.

- *Decrease run-off from upstream areas into the reservoirs*
  If less water is entering the reservoirs, less water needs to be discharged through the city to the bay. When an extra reservoir is build upstream, less run-off will enter the Addicks and Barker Reservoirs. Another plan, which dates back to the forties, is constructing a levee upstream that prevent the overflowing water from Cypress Creek to flow into the Addicks watershed. Smaller alternatives to decrease the run-off upstream, is applying infiltration measures upstream, such as green roofs or an infiltration area.

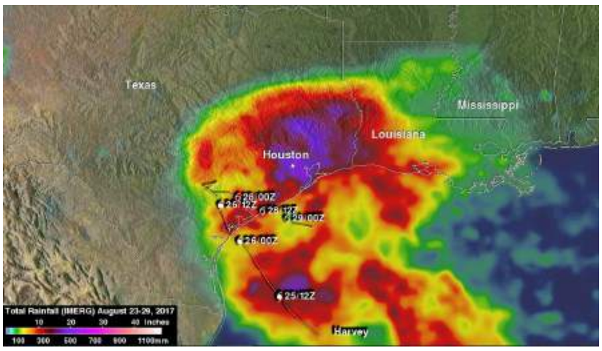
7 Hurricane Harvey

In this chapter, the focus is the performance of the Addicks and Barker Reservoirs during Hurricane Harvey. In Appendix A photos are added made shortly after the hurricane in and around the reservoirs which show the impact of hurricane Harvey in the area.

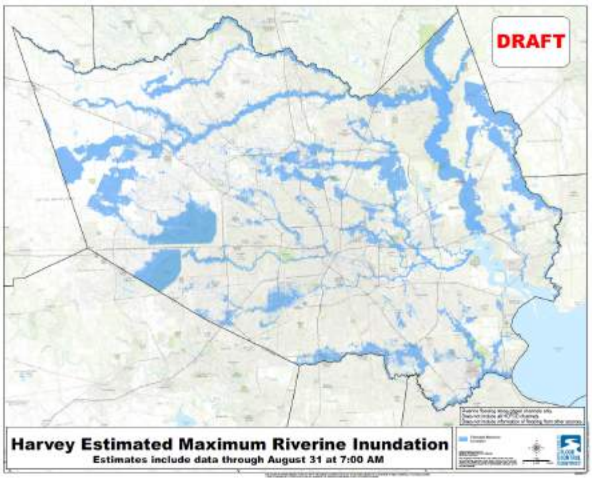
The data used in this chapter is derived from the same gages as used in Chapter 4, see Appendix E for the exact gage numbers and corresponding locations. Besides, information announced in the media by the US-ACE Galveston District and the Harris Country Flood Control District (HCFCD) is used as several gages were damaged and stopped performing during Hurricane Harvey.

7.1. General information

Hurricane Harvey made landfall in Texas near Corpus Christi and reached Houston on August 25, 2017. Hurricane Harvey was devastating due to the enormous amount of rainfall and not because of wind or storm surges. In Figure 7.1a the rainfall and path of Harvey is shown. Harvey stalled above Houston for 5 days in which the rainfall in Houston and in Harris County reached a new record averaging on approximately 910 mm [36 inches] of rain in a 4 day period. In some locations, the rainfall was even bigger up till 1250 mm [49 inches]. This amount of rainfall led to flooding of approximately 136,000 structures and the death of approximately 30 people in Harris County alone [34] [38]. The damage estimates for all affected areas is around 108 billion dollars [35].The effected areas as published by by the HCFCD can be found in Figure 7.1b.



(a) Hurricane Harvey's path and rainfall [40]



(b) Flooding due to Hurricane Harvey in Houston [31]

Figure 7.1: Hurricane Harvey

7.2. Performance of Addicks and Barker Reservoirs

The Addicks and Barker Reservoirs are designed to temperately store run-off and rainfall of the upstream areas. Normally when a big rain event is expected, the outlets of the dams are closed to reduce the risk of flooding of downstream residents and are opened again after the event. During the night of 27-28 August 2017, during the even of Hurricane Harvey in Houston, the outlets were opened during the event, because the USACE was afraid for risk on failure of the dams. This resulted in more downstream flooding, on top of the already exciting flooding caused by the runoff in the Buffalo Bayou watershed due to the rainfall. At the same moment, the pool elevations in Addicks and Barker reached a new record height, leading to flooding extending the government owned land upstream of the reservoirs. The precipitation directly in the reservoirs combined with runoff from upstream areas, led the pool levels to this record despite some intermediate releases.

In Figure 7.2 a time line is given which contains some general information about the performance of the Addicks and Barker Reservoirs which the USACE Galveston District and the Harris Country Flood Control District (HCFCD) have announced via the media. Specific information about releases are derived from posts of the USACE Galveston District and Jeff Lindner (HCFCD) on their Twitter accounts.

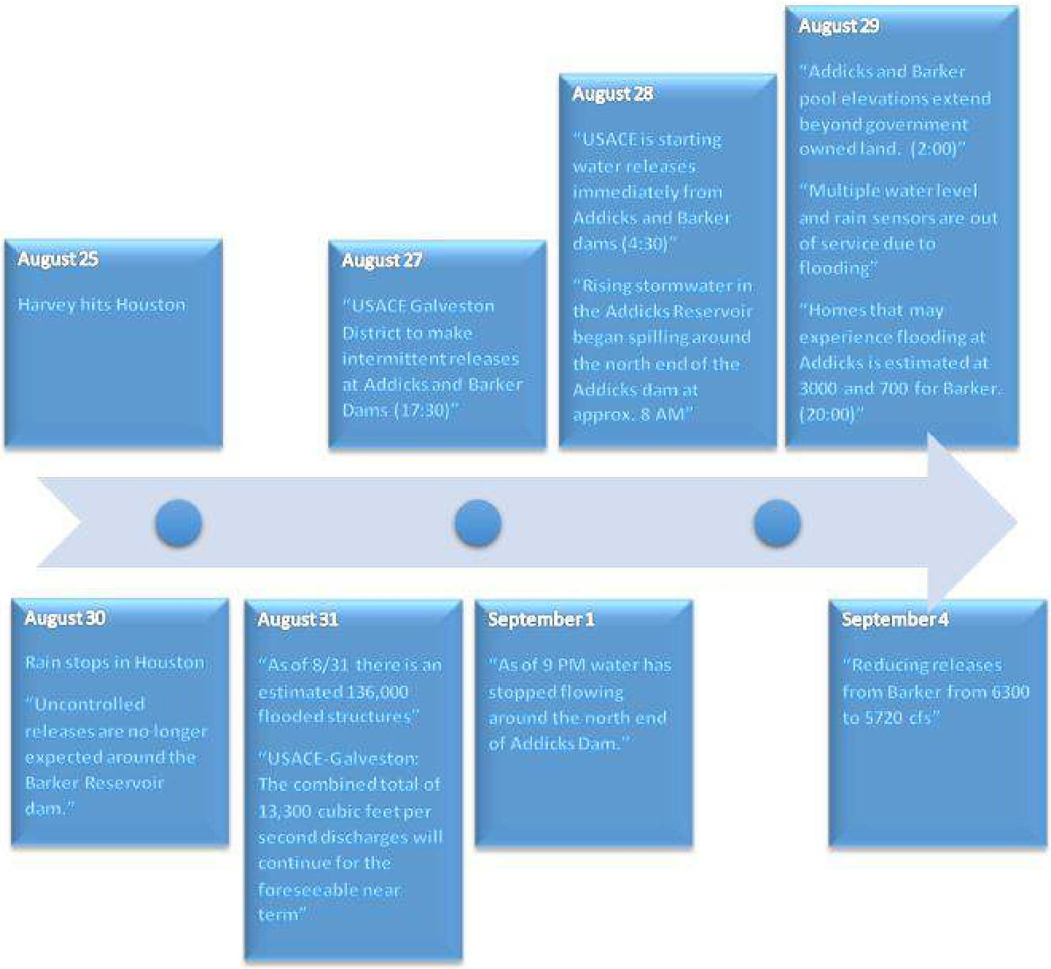
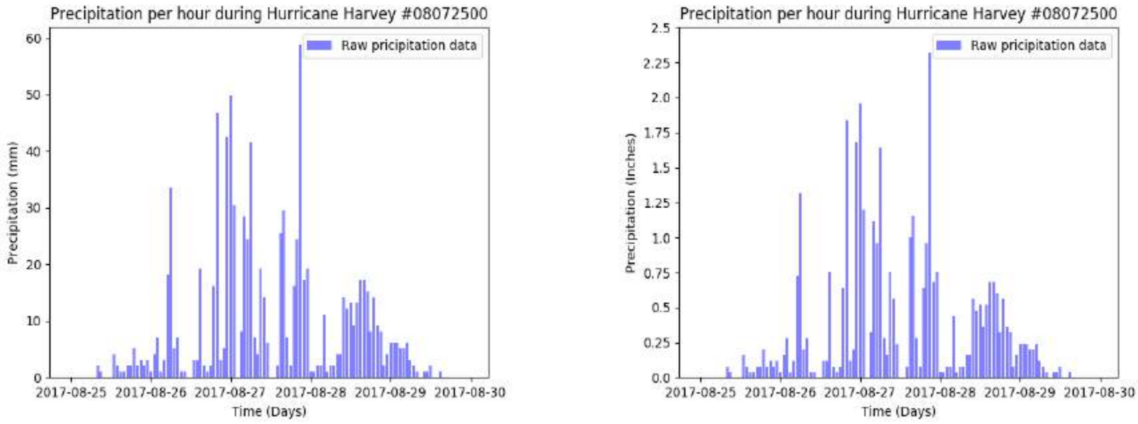


Figure 7.2: Timeline with announced information

7.2.1. Precipitation

The total amount of precipitation during Hurricane Harvey at the Addicks and Barker Reservoirs is 920 mm [36.2 inches] for a duration of 4 days. This precipitation exceeds the 1 in 500 year precipitation of 536 mm [21.1 inches] in 4 days of Harris County estimated Rainfall frequency [17]. The total amount of rainfall during Hurricane Harvey was 927 mm [36.5 inches] in five days. The maximum rainfall for a duration of 1 hour during Harvey is 59 mm [2.32 inches]. In Figure 7.3, this rainfall is plotted.



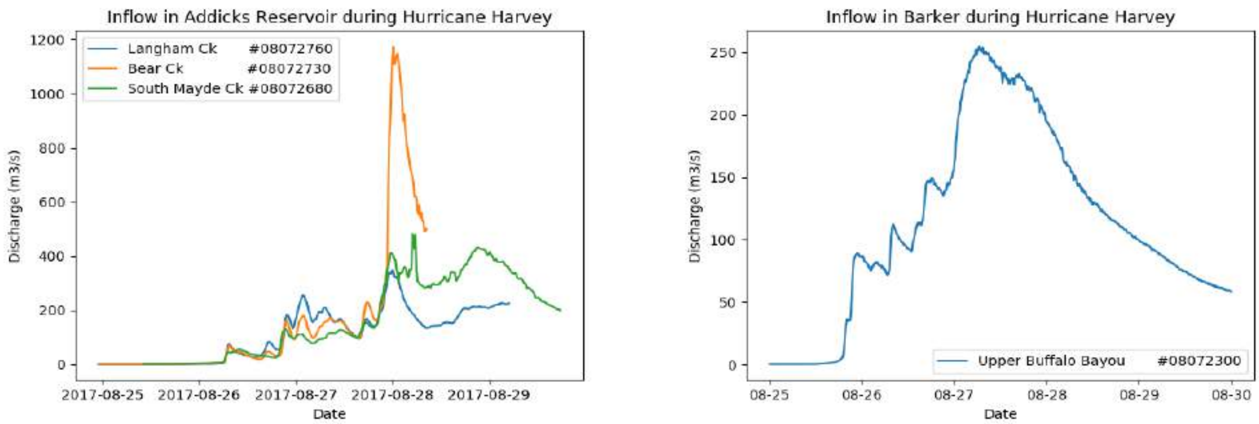
(a) Addicks [27]

(b) Barker [30]

Figure 7.3: Basin and watershed

7.2.2. Inflow

In the Addicks Reservoir, three creeks lead runoff from upstream areas into the reservoir: Langham-, Bear- and South Mayde Creek. The Upper Buffalo Bayou flows into the Barker Reservoir. Horsepen Creek and Mason Creek have an inlet into the reservoirs as well, but these two creeks are excluded in this report as no gage information at these locations is available. In Figure 7.4, the inflow from the creeks into the reservoirs are plotted. All gages in the creeks at Addicks Reservoir, were damaged and therefore stopped performing during Hurricane Harvey. The gage in Langham Creek stopped performing at August 29 in the morning, Bear Creek stopped performing at August 28 in the morning and South Mayde Creek stopped at August 29 in the early evening.



(a) Addicks

(b) Barker

Figure 7.4: Inflow into the reservoirs

7.2.3. Water level of the Reservoirs

The Addicks Reservoir was completely empty before Hurricane Harvey hit and started to fill up directly during this event. According to the data of the gages, the water levels reached the top of the spillway (33.18 m [108.85ft]) on August 29 and started spilling water on the sides. The houses near to the edges of the basin already started flooding in the morning of August 28 when the water level reached an elevation of 31.59 m [103.65 ft]. The reservoir remained full and spilled over the spillway until the 23rd of September. After the 23rd of September, the water level was reduced to a non flooding state in 1 day [33]. See Figure 7.5 for an overview of the water level elevations of the Addicks Reservoir.

According to the announced information in the media, the USACE announced on September 2 that the water levels had already lowered to an elevation of 32.82 m [107.67 ft], which is lower than the spillway elevation[52]. On 30th of August, the Addicks Reservoir reached a record water level of 33.25 m [109.1 ft]. The previous record occurred during the Tax day flood in 2016 in which the water level reached 31.30 m [102.7 ft].

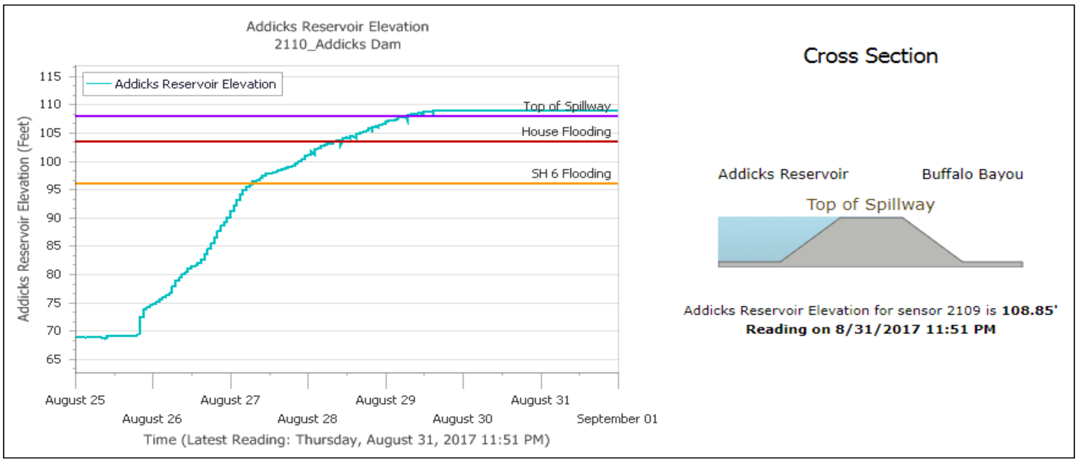


Figure 7.5: Water level of Addicks Reservoir - During Harvey [26]

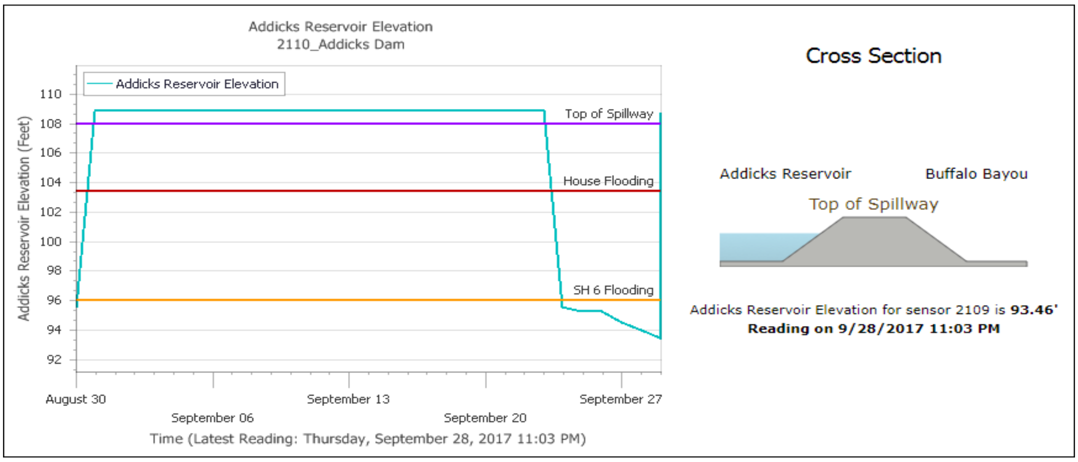


Figure 7.6: Water level of Addicks Reservoir - After Harvey [26]

The Barker Reservoir was also empty before Hurricane Harvey hit and was filled during the event. The top water level of Barker of 30.46 m [99.94 ft] was reached on August 31. The water level in Barker did not reach the spillway height of 31.70 m [104 ft]. The houses upstream of the Barker Reservoir started flooding on the 28th of August. On September 22, one day earlier than in the Addicks Reservoir, the water level dropped below the flood state threshold [33]. The previous record water level at Barker was 29.02 m [95.22 ft]. In the information given to the public, the water level peaked on 30th of August with a level of 30.97 m [101.6 ft] [52]. See Figure 7.7 for an overview of the water level of Barker Reservoir.

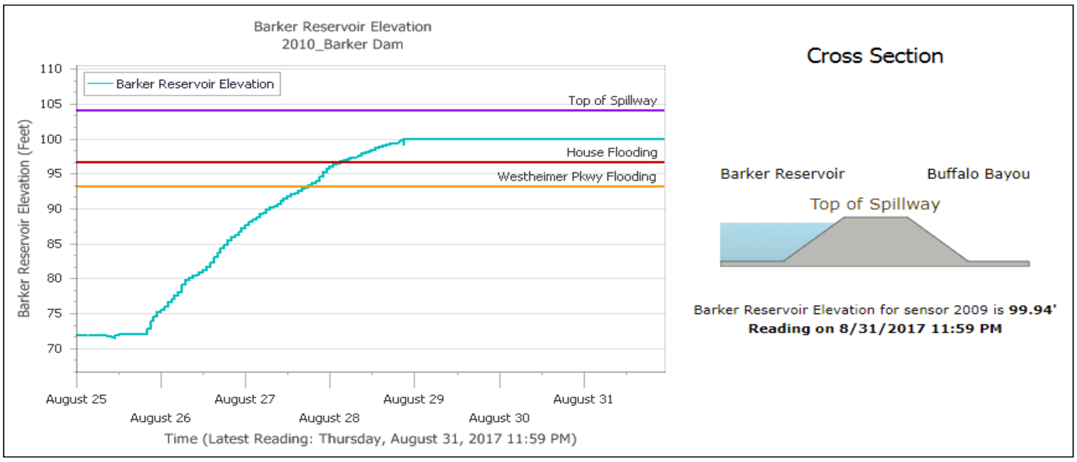


Figure 7.7: Water level of Barker Reservoir - During Harvey [29]

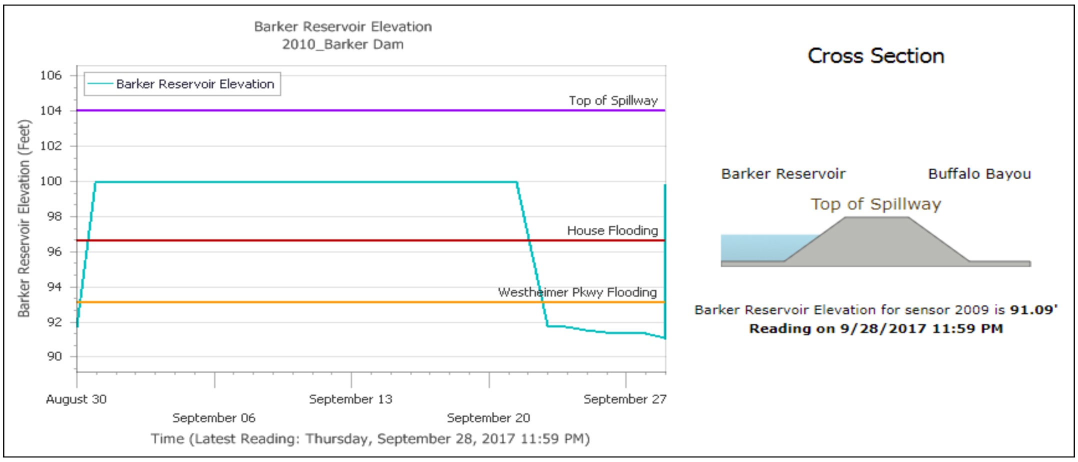


Figure 7.8: Water level of Barker Reservoir - After Harvey [29]

7.2.4. Outflow and releases

The outflow of the reservoirs is measured by a gage located in the Buffalo Bayou downstream of the location where the outlets of the reservoirs come together; the gages which are located at the outlets of both reservoirs did not record the discharges during Hurricane Harvey. In Figure 7.9, the discharge is plotted in the Buffalo Bayou during Harvey and in Figure 7.10 the outflow from the reservoirs is shown just after Harvey.

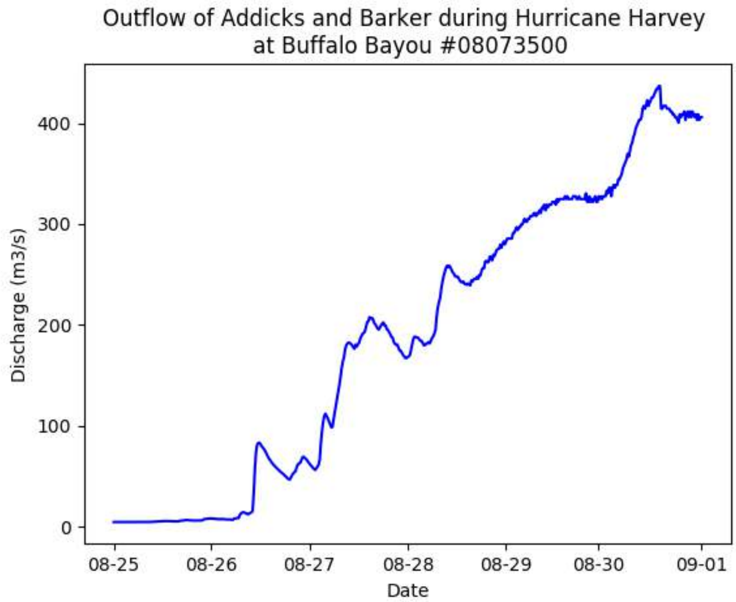


Figure 7.9: Discharge of Buffalo Bayou behind the dams during Harvey

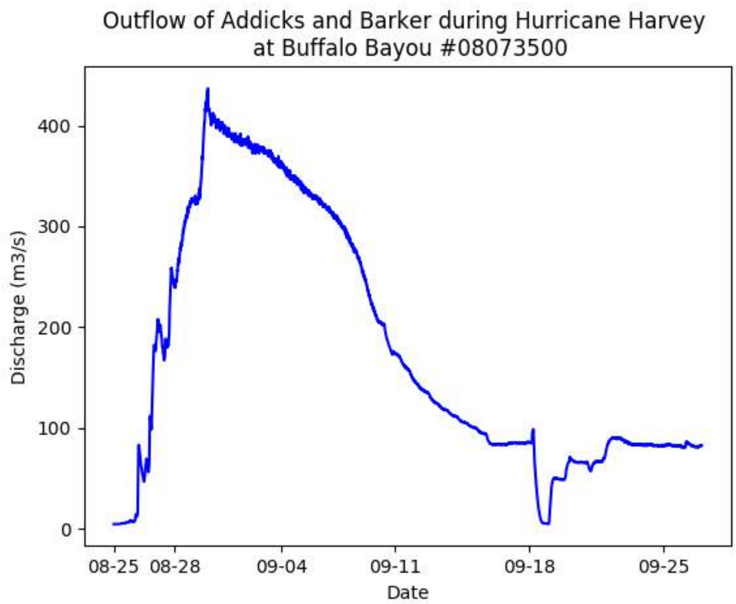


Figure 7.10: Discharge of Buffalo Bayou behind the dams after Harvey

The USACE announced that they released a total of 376.62 m^3/s [13,300 cfs] during Hurricane Harvey, 198.22 m^3/s [7,000 cfs] from the Addicks Reservoir and 178.40 m^3/s[6,300 cfs] from the Barker Reservoir on August 28. They announced that the releases from the Addicks and Barker Reservoirs will gradually be decreased from September 5 until September 17 to a level below the 113.27 m^3/s [4,000 cfs] threshold. In Figure 7.10, it can be seen that the peak outflow is higher than 376.62 m^3/s [13,300 cfs], there is a difference of around 56.63 m^3/s [2000 cfs] between the measurements and the announcements. It could be that this difference is caused by the inflow of a part of the Buffalo Bayou watershed run-off,which is located in between the Addicks and Barker outlets and the used gages. The discharge in the Buffalo Bayou is below the threshold of 113.27 m^3/s [4,000 cfs] on September 17.

7.2.5. Conclusion

It is clear that Hurricane Harvey impacted many people and led to major damage in Harris County alone. The measured precipitation for a duration of 24 hours is 459.23 mm [18.08 inches], this corresponds with a return period in between the 500 and 1,000 years from the analysis in Chapter 4. The measured rain event of 920 mm [36.2 inches] in 96 hours due to Hurricane Harvey exceeds a rain event with a return period of 1/10,000 years for 96 hours. The water level measured by the gages differs a little from the communicated water levels by the USACE. The measured water levels of the Addicks and Barker Reservoirs were 33.18 m [108.85 ft] and 30.46 m [99.94 ft] while the communicated water levels were 33.25 m [109.1 ft] and 30.97 m [101.6 ft].

8 Conclusion

8.0.1. Conclusion Design water level and Dam failure

The maximum water level that the dams can manage without failing as calculated in Chapter 5.3 will be compared with the calculated probable water levels from Chapter 4.3. The original design levels and currently used water levels for managing the dams are discussed as well, see Tables 8.1 and 8.2.

Table 8.1: Addicks Reservoir comparison

Addicks	**Critical water level [m]**	**Return period [y]**	**Duration [h]**
Original design water level	34.74 [113.97 ft]	1,000	96
		10,000	48
Current maximum allowed water level	29.72 [97.51 ft]	25	48
		50	24
		100	24
		250	12
		500	8
		1,000	8
		10,000	4
Overtopping	35.15 [115.32 ft]	10,000	48
Stability	-		
Piping	31.91 [104.69 ft]	250	72
		500	48
		1,000	48
		10,000	12

Table 8.2: Barker Reservoir comparison

Barker	**Critical water level [m]**	**Return period [y]**	**Duration [h]**
Original design water level	32.00 [104.98 ft]	10,000	72
Current maximum allowed water level	28.53 [93.60 ft]	50	96
		100	72
		250	48
		500	48
		1,000	48
		10,000	24
Overtopping	32.80 [107.61 ft]	10,000	72
Stability	-		
Piping	30.99 [101.67 ft]	1,000	96
		10,000	48

The original design water level of the dams is based on the Probable Maximum Flood. However, the current maximal allowed water level, set by USACE, is used for managing the releases and is approximately 5.02 m [16.46 ft] and 3.47 m [11.48] lower. The maximum allowed water level will probably be observed for the Addicks Reservoir once in the 25 years and for the Barker Reservoir once in the 50 years.

The maximum allowed water level is for the Addicks Reservoir 2.19 m [7.19 ft] below the calculated maximum water level before failure caused by piping occurs. If mitigations against piping are taken, the dams can manage an even higher water level than they are originally designed for. The original design water level at an elevation of 34.74 m [113.98 ft] is 2.83 m [9.28 ft] higher than the critical water level according to piping.

Barker Reservoir has a maximum allowed water level of 28.53 m [93.60 ft], which is 2.46 m [8.07 ft] below the maximum water level for which piping is prevented. For Barker Reservoir, the design water level is 1.01 m [3.31 ft] higher than the calculated critical water level.

It is not possible to determine which water level corresponds with the natural water level caused by Hurricane Harvey, because of the releases into the Buffalo Bayou and the broken gages. However, Chapter 7 shows that the measured water levels were significantly higher than the maximum allowed water levels which the USACE maintains. The measured water levels were 33.18 m [108.85 ft] and 30.46 m [99.94 ft] for Addicks and Barker Reservoir which is 3.46 m [11.34 ft] and 1.93 m [6.34 ft] higher than actually allowed.

8.0.2. Conclusion optimization

In Chapter 6, an optimization was conducted to find the minimal flood damage up- and downstream, due to the releases of the Addicks and Barker Reservoirs into the Buffalo Bayou. From this optimization is concluded that the run-off of the Buffalo Bayou watershed has a big influence on downstream flooding. For the duration of 12 hours and 24 hours there is already damage downstream, without any releases from the reservoirs. When the events have a duration of 12 hours and 24 hours and the Addicks and Barker Reservoirs remain closed, they prevent additional flooding downstream without creating damage upstream. For the duration of 12 hours and 24 hours, the Addicks and Barker Reservoirs fulfill there function to prevent Houston from flooding.

For the events with a longer duration, it is advisable to open the outlets of the reservoirs, because this will minimize the total damage. In the events with a duration of 72 hours and 96 hours and a return period of 1/100 years or up, only a part of the excess volume needs to be discharged downstream. For the most extreme events with a return period of 1/10,000 years and a duration of 48 hours and more, the reservoirs should release all there excess volume.
In this study, the piping problem determines the critical water level. When this piping problem is solved, the critical water level will be raised to the critical water level for overtopping. When the Addicks Reservoir is chosen to be dominant and therefore determine the release ratio, the water level will not reach these critical levels, when Barker Reservoir is dominant, Addicks Reservoir could reach these critical levels. For the current situation, it is advised to release water downstream before reaching the critical water level of piping.

Concluding, if the optimization is done to minimize financial damages of residential properties and the calculated ratios of releases are applied, more people living upstream will be effected than people living downstream during big storm events.

9 Discussion and Recommendations

The system of the Addicks and Barker Reservoirs is complex and extensive, and due to limited time, several simplifications and assumptions were made throughout this project. In further research it is advised to include:

- *Mitigation study*
  As mentioned in Chapter 6, several mitigations of changing the boundary conditions are possible. Many people suffered damage during recent floods, especially during Hurricane Harvey. It is strongly advised to conduct a mitigation study in which the damages can be further minimized.

- *Floodplain modeling*
  During Hurricane Harvey, many homes flooded which were not in the known 500-year floodplains. It is important to conduct further research whether this happened because the precipitation was more than a 1/500 years event, the releases caused a too high discharge level in the Buffalo Bayou or the floodplain changed. Besides, more floodplains would increase the accuracy of the optimization of the damages.

- *Special structures*
  The spillways and outlets are not fully considered in this project. Research into the critical water level for which the dams will fail due to failure of the special structures is advised. Furthermore, the area affected by overflowing the spillways are not taken into account, but should be included in further research, because this could cause major flood damages around the reservoirs and will adjust the areas that will be flooded around the Addicks and Barker Reservoirs.

- *More elaborated damage study*
  A more elaborated damage study is required to give a good optimization of managing the Addicks and Barker Reservoirs. In this report, only damages to residential properties are taken into account. For a complete damage study, more aspects should be taken into account: direct damage due to flooding like infrastructure, vehicles, flooding of businesses and fatalities, but also indirect damage like loss of market position. A more elaborated damage study can give a better insight on the real damages.

- *Optimization based on people*
  The choice can be made to optimize based on the amount of people who will be affected by flooding of different areas. In this way a more fair comparison will be made in terms of wealthy and poor neighborhoods.

- *More elaborated release study*
  In this study, it is assumed that the releases by Addicks and Barker in the optimization start at the beginning of the event and remain constant during the event. It is advised to extend the study to different releasing times, for example 12 or 24 hours after the start of the rain event. This could give different, extensive optimization results. Another assumption that has been made is that the Addicks and Barker Reservoirs only release water with the same ratio. Further research in which both reservoirs use different ratios is advisable, so the reservoirs can interact more.

- *Modeling the breach of the dams.*
  In this report, the scenario where breaching of the dams occurs, is prevented, but not calculated. However, this is a heavily impacting and possible scenario and should therefore be considered. This can be best done by modeling the water flow if the dams would fail. This can for example be done with a SOBEK-model (2D flow model). The resulting water flow and their corresponding damages could give a different insight in this research.

- *Investigation of the soil characteristics*
  Detailed soil characteristics will give a more accurate study on the failure mechanisms. In this research, only four different cross-sections for Addicks Reservoir and five different cross-sections for Barker Reservoir were considered. The differences between those cross-sections are only based on shape of the dams, the soil layers are considered to be the same. This assumption was made due to lacking information about the soil composition. Therefore further research to soil composition is advised, preferable by making soil penetration tests.

- *Sediment accumulation inside the reservoirs*
  There should be investigated if sediment is transported into the reservoirs by discharge through the inflowing creeks. Sediment accumulation at the bottom of the reservoirs could decrease the effectiveness of the reservoirs significantly. The impact of this accumulation on the storage height and functioning of the outlets are advisable to be considered as well.

- *Probabilistic strength calculations*
  In this research the strength of the dams is tested in a deterministic way. However, it would be better to use a probabilistic approach for the strength of the dams since this will give a more accurate overview of the strength of the dams and secondly, a failure probability of the dams can be determined.

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A

Pictures of Addicks and Barker

(a) Barker Reservoir

(c) Closed road

(b) Barker Reservoir

(d) House in Houston

Figure A.1: Pictures from field trip

(a) 10-09-2017

(b) 20-09-2017

Figure A.2: Outlet of Barker Reservoir

B
Datum

In the design reports different vertical datums (base measurement points) are used to define elevations. The National Geodetic Vertical Datum of 1929 (NGVD 29) is a base measurement point which has been used during the 20th century as a basis to compare ground and flood elevation. In 1988 this system is replaced by a more accurate system: the North American Vertical Datum of 1988 (NAVD 88). In this datum, the local variations of the sea level and changes of the ground elevation are included. This difference in elevation is called "datum shift" and depends on the observed location. In Figure B.1 an overview of the datum shift is shown for the whole country.

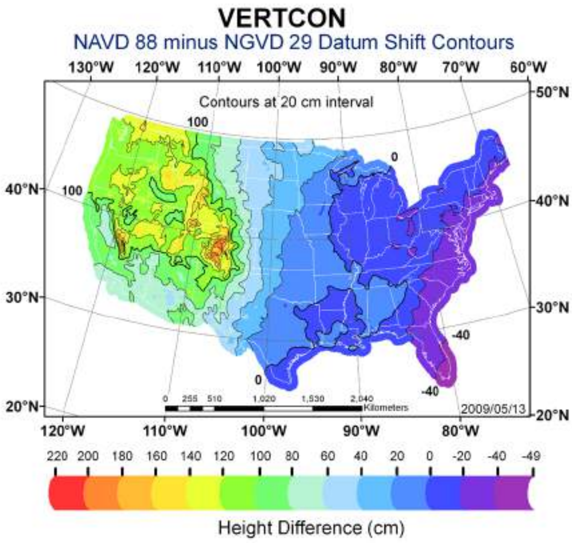


Figure B.1: Datum Shift Contours [46]

By using the VERTCON datum conversion tool, made by the National Geodetic Survey (NGS), the datum shift of NAVD 88 minus NGVD 29 can be given at the millimeter accurate per location. By filling in the North latitude and West longitude of Houston (29°45.797'N , 95°21.796' W) a datum shift of 0.004 m [0.16 inch] is given [47]. This difference in elevation is negligible comparable to the elevations used in this report, so the choice is made to not convert the two datums, using NAVD 88 as the standard.

C

Design water level

C.1. Probable Maximum Precipitation and Flood

The Probable Maximum Precipitation (PMP) gives an estimate of the maximum precipitation depth, based on area and duration. There is a wide range of methods to make the PMP estimate, in this appendix the generalized estimation method will be briefly explained following three steps [45], [48], [42].

- Moisture maximization: The storm precipitation will be increased to a certain value which is consistent with the expected maximum moisture in the atmosphere in the area and the season of the storm event.
- Transposition: The new storm precipitation values are transposed within a homogeneous region that is relative to the terrain and meteorological features related to the storm rainfall.
- Envelopment: Envelopment is used for selecting the largest values from all selected storm data by using a Depth– Area–Duration (DAD) Analysis. By smoothly interpolating between the maxima in the plot with the adjusted and transposed rainfall data, the PMP can be estimated.

The Probable Maximum Flood (PMF) means the theoretical maximum flood and is derived from the PMP by taking the most critical, disadvantageous meteorological and hydrological conditions.

D

Distributions

D.1. Log-Pearson Type III Distribution

The Log-Pearson Type III distribution is used for extrapolation of the water levels of the Addicks and Barker Reservoirs. First the maxima of the data are ranked from largest to smallest value, where n is the total amount of values and m is the rank of a value. The return period of the values is calculated as D.1. The log of all values is calculated and used to calculated the variance in D.2, the standard deviation in D.3 and the skewness Coefficient in D.4. The skewness coefficient determines the Frequency factor K for Gamma, Haan 1977 Table 7.7 [49]. The return periods for which the Log-Pearson can be calculated is limited by the K factor. The used return periods are 2, 5, 10, 25, 50, 100 and 200 years. If the calculated skewness coefficient lies in between two skewness coefficients in the table, the K needs to be linear extrapolated. Each K factor is linked to a certain return period, so by using formula D.5 the values for the elevation are found for there corresponding return period.

$$Return\,period = \frac{n+1}{m} \tag{D.1}$$

$$var = \frac{\sum_i^n \left(\log Q_i - \overline{\log Q}\right)^2}{n-1} \tag{D.2}$$

$$\sigma = \sqrt{var} \tag{D.3}$$

$$skew = \frac{n\sum_i^n \left(\log Q_i - \overline{\log Q}\right)^3}{(n-1)(n-2)(\sigma \log Q)^3} \tag{D.4}$$

$$\log Q_T = \overline{\log Q} + K\sigma \log Q \Rightarrow Q_T = 10^{\overline{\log Q} + K\sigma \log Q} \tag{D.5}$$

D.2. General Extreme Value distribution

The general extreme value distribution consists of the Fréchet, Weibull and Gumbel distribution, these distributions are further elaborated in the next paragraphs.

Fréchet

The Fréchet distribution within the GEV-distribution is used to fit the Langham Creek and Upper Buffalo Bayou inflow volumes. See D.6 and D.7 for the PDF and CDF.

PDF:

$$f(x) = \frac{\alpha_f}{\beta_f}(\frac{\beta_f}{x-\eta_f})^{\alpha_f+1}\exp(-(\frac{\beta_f}{x-\eta_f}))_f^{\alpha} \tag{D.6}$$

CDF:

$$F(x) = \exp(-(\frac{\beta_f}{x-\eta_f})^{\alpha_f}) \tag{D.7}$$

Where:

$$\alpha_f = shape\,parameter \tag{D.8}$$

$$\beta_f = scale\,parameter \tag{D.9}$$

$$\eta_f = location\,parameter \tag{D.10}$$

$$F(x) = 1 - \frac{1}{T} \tag{D.11}$$

Weibull Distribution

Weibull is plotted for every data set within the GEV-distribution, but did not fit the data best. The PDF and CDF are given in D.12 and D.13.

PDF:

$$f(x) = \frac{\alpha_w}{\beta_W}(\frac{x-\eta_w}{\beta_w})^{\alpha_w-1}\exp(-(\frac{x-\eta_w}{\beta_w})_w^{\alpha}) \tag{D.12}$$

CDF:

$$F(x) = 1 - \exp\left(-(\frac{x-\eta_w}{\beta_w})^{\alpha_w}\right) \tag{D.13}$$

Where:

$$\alpha_w = shape\,parameter \tag{D.14}$$

$$\beta_w = scale\,parameter \tag{D.15}$$

$$\eta_w = location\,parameter \tag{D.16}$$

$$F(x) = 1 - \frac{1}{T} \tag{D.17}$$

Gumbel Distribution

Gumbel is used for the discharge data of Bear Creek and South Mayde Creek. To fit the Gumbel distribution, maximum likelihood estimators for the Gumbel parameters X_0 and β are used which iteratively calculate a value for the parameters.

$$X_0 = -\beta * \ln\left(\frac{1}{n}\sum_{i=1}^{n} exp(-\frac{x_i}{\beta})\right) \tag{D.18}$$

$$\beta = \frac{1}{n}\sum_{i=1}^{n} x_i - \frac{\sum_{i=1}^{n} x_i * \exp(-\frac{x_i}{\beta})}{\sum_{i=1}^{n}\exp(-\frac{x_i}{\beta})} \tag{D.19}$$

The Gumbel distribution:

$$F(x) = \exp\left(-\exp\left(-(\frac{x-x_0}{\beta})\right)\right) \tag{D.20}$$

Where:

$$F(x) = 1 - \frac{1}{T} \tag{D.21}$$

E

Data analysis

E.1. Gages

The data from gages used for the design water level analysis are derived from the the databases of USGS [53] and HCFCD [31]. Table E.1 and Figure E.1 contain information about the used gages. The precipitation measured during Harvey is still 'Provisional data' and subject to revision.

Table E.1: Gages

Gage number	Location	Used for	Starting year
08072760	Langham Ck at W Little Yord Rd nr Addicks	Inflow in Addicks	2001
08072730	Bear Ck nr Barker	Inflow in Addicks	1993
08072680	S Mayde Ck at Heathergold Dr nr Addicks	Inflow in Addicks	2015
08072300	Buffalo Bayou nr Katy	Inflow in Barker	1990
08073100	Langham Ck at Addicks Res Outflow nr Addicks	Outflow of Addicks	2013
08072600	Buffalo Baryou at State Gw 6 nr Addicks	Outflow of Barker	2010
08072500	Barker Res nr Addicks	Precipitation	1987
08073500	Buffalo Bayou nr Addicks	Outflow of Addicks and Barker	1990

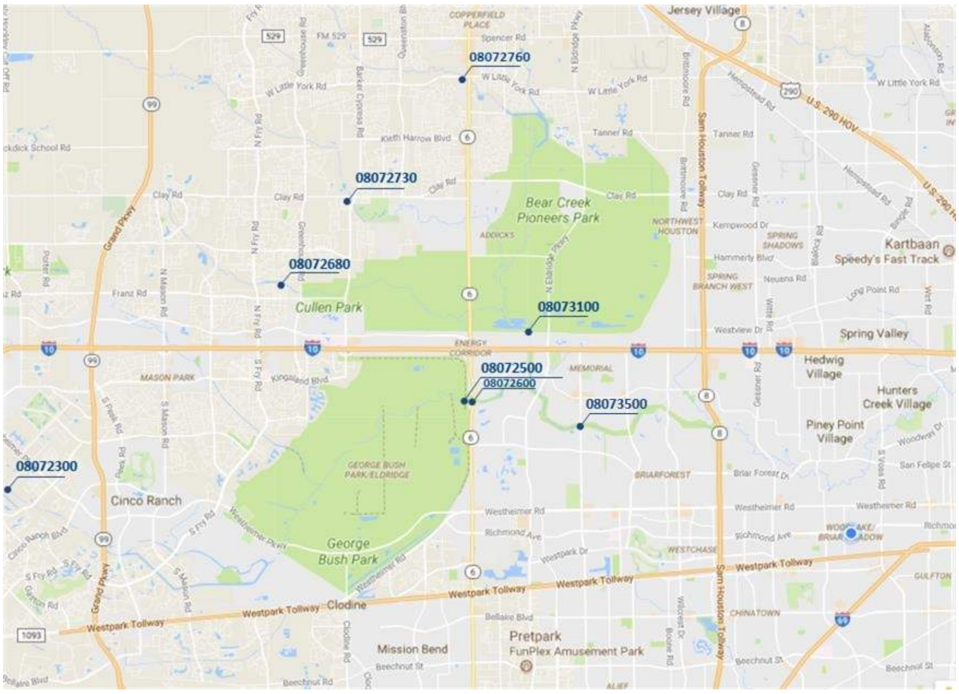


Figure E.1: Gages on Map

E.2. Data Analysis

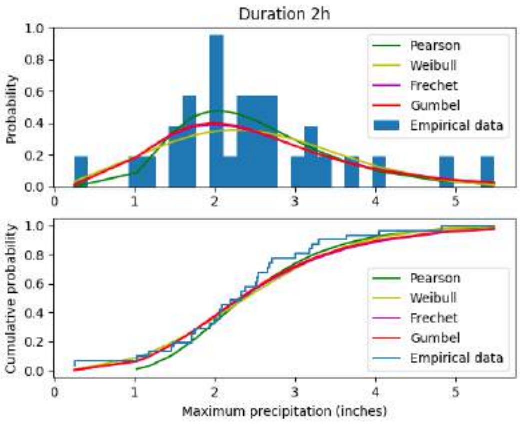
To fit the data onto a distribution (either a Log-Pearson type III or a GEV-distribution), first the empirical data is plotted as a Probability Density Function (PDF) and as a Cumulative Distribution Function (CDF). For each distribution, the location, scale and shape parameter are estimated using a Maximum Likelihood Estimator (MLE) of the Python extension from the scipy.stat package. Within this function, the best fit for the data is calculated following a particular distribution.
To compare the four fits, the root mean square error method is applied. The RMSE gives the vertical difference between the empirical and fitted data by taking the difference between the points, taking the square of it, followed by taking the mean and the root of the mean, see E.1. All errors of the used distributions for the data is given in Table E.2 to E.13.

$$RMSE = \sqrt{\frac{1}{n}\sum_{t=1}^{n} e_t^2} \tag{E.1}$$

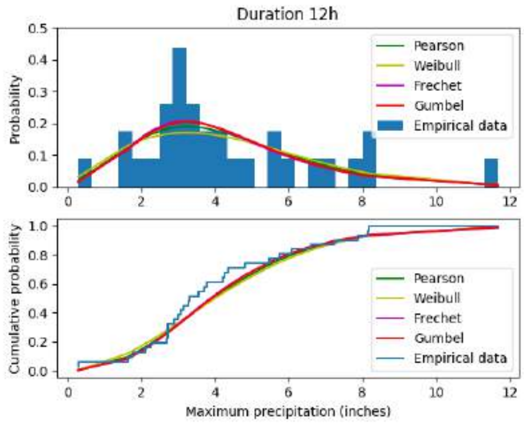
According to the RMSE method, the best fit is chosen and this chosen distribution is used for extrapolation up to a return period of 10,000 years. After extrapolation, common sense is used to decide if the results are credible or if the results need to be extrapolated following another distribution. The upper tail of the distributions is considered most important as the interest is in maximum values.

E.3. Precipitation

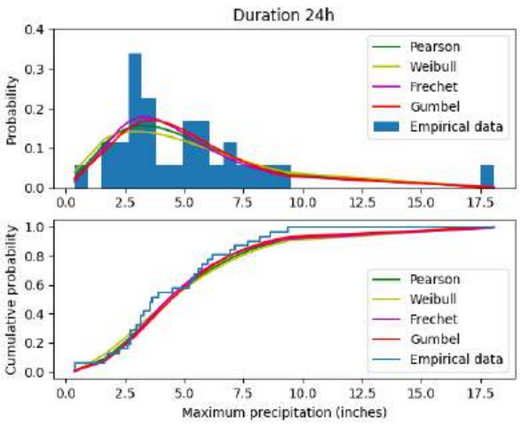
In Figure E.2 the plots of the PDF and the CDF is shown for the durations 2 hours, 12 hours, 24 hours and 96 hours. In Table E.2, the RMSE of all durations can be seen. Overall, the Weibull distribution performed best according to the RMSE. However, some durations cross each other in the plot, which is physically not possible. Therefore the Gumbel distribution is fitted, the upper tail of the Gumbel distribution performs best. When extrapolating the Fréchet distribution the values became unrealistically high, so this fit is dismissed as possible fit.
The result of the precipitation analysis is shown in Table E.3.



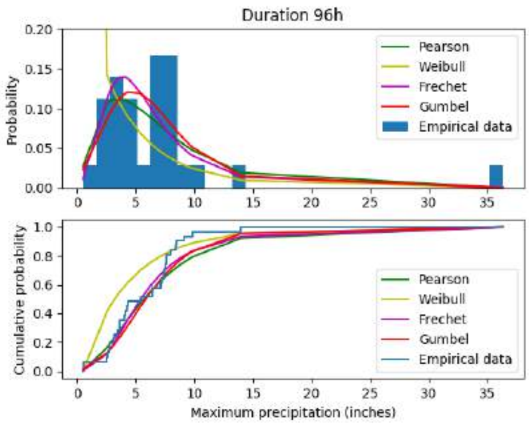
(a) Duration 2h



(c) Duration 12h



(b) Duration 24h



(d) Duration 96h

Figure E.2: Precipitation at Addicks and Barker Reservoirs

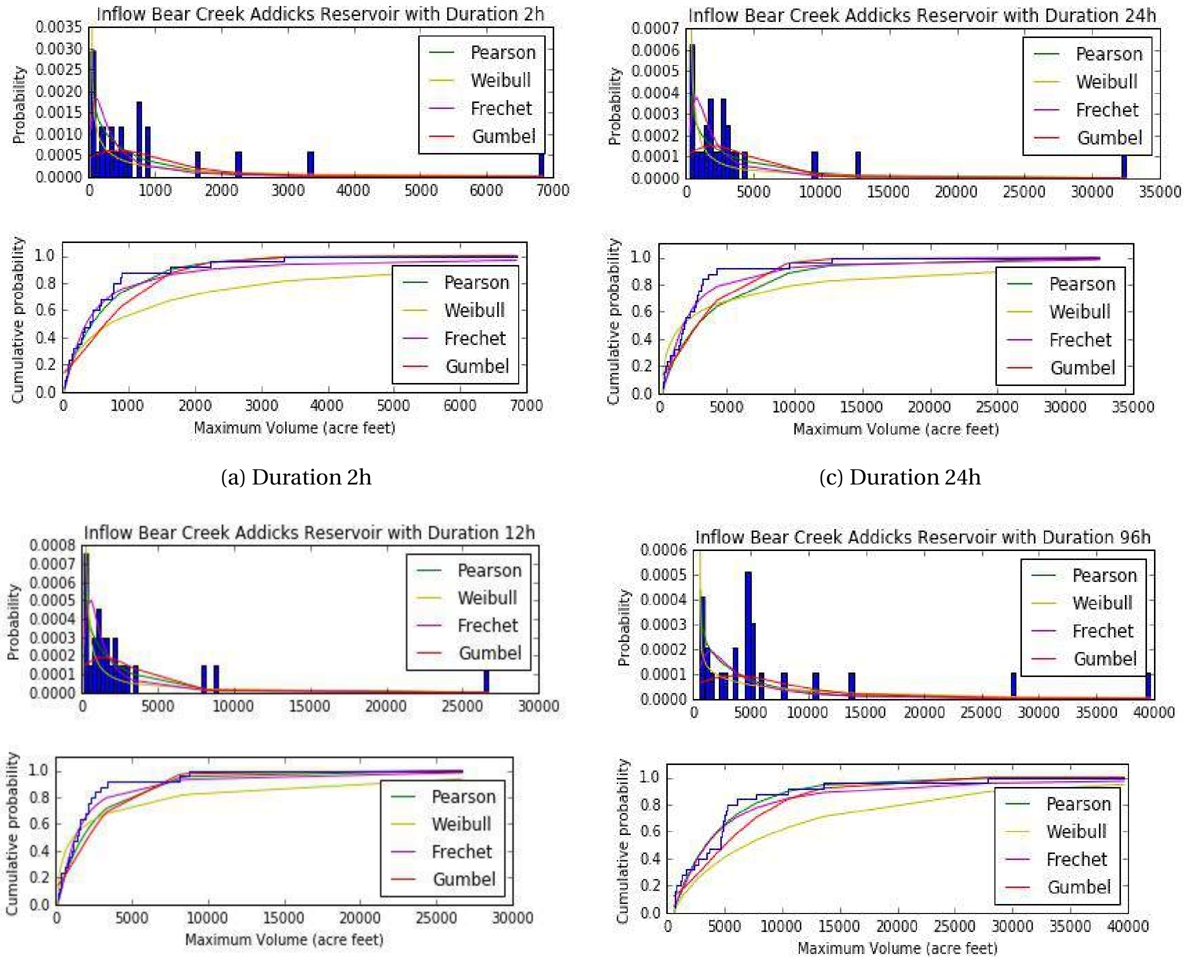
Table E.2: RMSE of the precipitation in Addicks and Barker Reservoirs

	1h	**2h**	**4h**	**8h**	**12h**	**24h**	**48h**	**72h**	**96h**
Log-Pearson	0.39066	0.31633	0.32603	0.30695	0.31006	0.31802	0.31232	0.30032	0.30271
Weibull	0.36925	0.29692	0.32011	0.30196	0.30357	0.30713	0.36422	0.29064	0.32077
Frechet	0.36365	0.29827	0.32188	0.30340	0.31222	0.32617	0.32765	0.32032	0.32321
Gumbel	0.36421	0.29987	0.32201	0.30341	0.31393	0.32907	0.32783	0.31979	0.32206
Best fit	Frechet	Weibull	Weibull	Weibull	Weibull	Weibull	Log-Pearson	Weibull	Log-Pearson

Table E.3: Precipitation Results with a Gumbel fit

Durations	Ret.Period	10	25	50	100	250	500	1,000	10,000
1	inches	2.77731	3.3728	3.81457	4.25308	4.83044	5.2664	5.70204	7.14845
	mm	70.54367	85.66912	96.89008	108.0282	122.6932	133.7666	144.8318	181.5706
2	inches	4.04369	4.91551	5.56227	6.20426	7.04954	7.6878	8.32559	10.44319
	mm	102.7097	124.854	141.2817	157.5882	179.0583	195.2701	211.47	265.257
4	inches	5.30223	6.48235	7.35783	8.22685	9.37105	10.23502	11.09836	13.9648
	mm	134.6766	164.6517	186.8889	208.962	238.0247	259.9695	281.8983	354.7059
8	inches	6.41776	7.86976	8.94694	10.01616	11.42397	12.48697	13.54921	17.07603
	mm	163.0111	199.8919	227.2523	254.4105	290.1688	317.169	344.1499	433.7312
12	inches	7.29268	8.99784	10.26283	11.51847	13.17173	14.42008	15.66752	19.80924
	mm	185.2341	228.5451	260.6759	292.5691	334.5619	366.27	397.955	503.1547
24	inches	8.52989	10.5849	12.10941	13.62267	15.61513	17.11959	18.62296	23.61443
	mm	216.6592	268.8565	307.579	346.0158	396.6243	434.8376	473.0232	599.8065
48	inches	10.32878	12.89081	14.79148	16.6781	19.16216	21.03781	22.91211	29.13512
	mm	262.351	327.4266	375.7036	423.6237	486.7189	534.3604	581.9676	740.032
72	inches	10.96814	13.67196	15.67782	17.66886	20.29039	22.26985	24.24788	30.8153
	mm	278.5908	347.2678	398.2166	448.789	515.3759	565.6542	615.8962	782.7086
96	inches	11.56024	14.42736	16.55435	18.66564	21.4455	23.54451	25.64201	32.60606
	mm	293.6301	366.4549	420.4805	474.1073	544.7157	598.0306	651.3071	828.1939

The inflow data is resampled by taking the maximum discharge per hour. The maximum hourly data is converted to volumes per duration 1, 2, 4, 8, 12, 24, 48, 72 and 96 hours. The GEV-distribution and the Log-Pearson type III are fitted onto the volume data. The PDF and CDF per duration are plotted the same way as with the precipitation data, see Figures E.3 till E.5 for the PDF and CDF plots for duration 2, 12, 24 and 96 hours and Table E.4till E.6 for the corresponding RMSE.



(a) Duration 2h

(c) Duration 24h

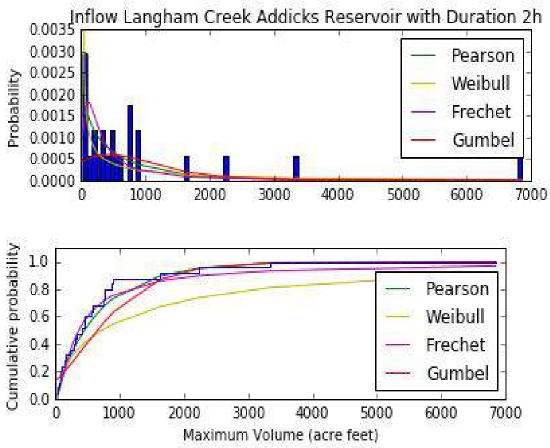
(b) Duration 12h

(d) Duration 96h

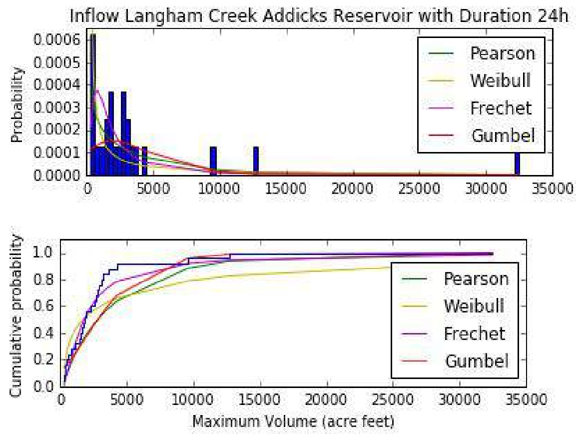
Figure E.3: Bear Creek, Addicks Reservoir

Table E.4: RMSE of the volumes of Bear Creek, Addicks Reservoir

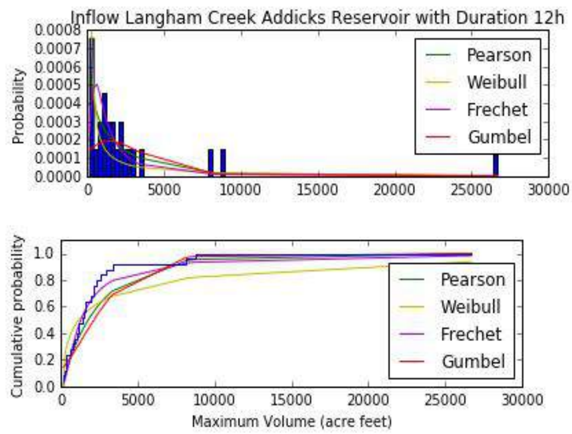
	1h	2h	4h	8h	12h	24h	48h	72h	96h
Log-Pearson	0.20961	0.20658	0.21024	0.22148	0.22002	0.26403	0.26020	0.28616	0.29646
Weibull	0.27037	0.21659	0.19058	0.20082	0.19482	0.22162	0.24887	0.25652	0.30286
Frechet	0.19749	0.19538	0.19705	0.21632	0.20911	0.25440	0.26555	0.27767	0.28713
Gumbel	0.23243	0.22945	0.23284	0.24531	0.23440	0.26056	0.26085	0.26865	0.27641
Best fit	Frechet	Frechet	Weibull	Weibull	Weibull	Weibull	Weibull	Weibull	Gumbel



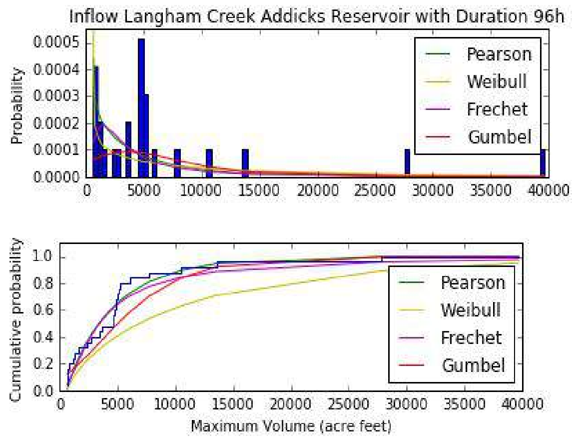
(a) Duration 2h



(c) Duration 24h



(b) Duration 12h

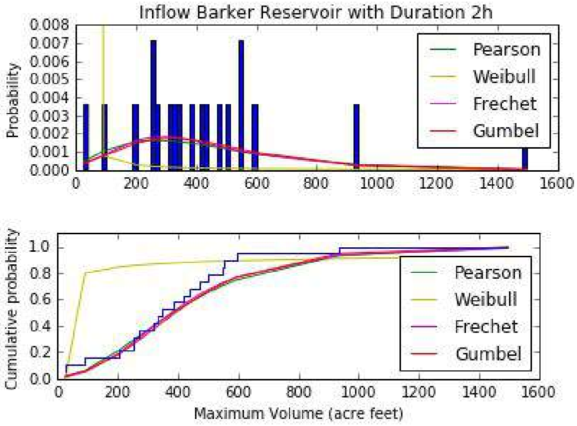


(d) Duration 96h

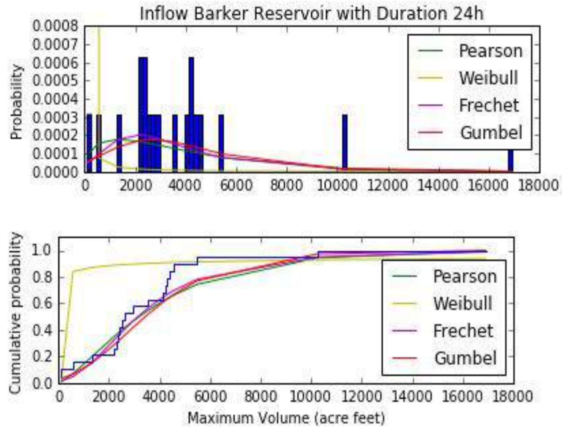
Figure E.4: Langham Creek, Addicks Reservoir

Table E.5: RMSE of the volumes of Langham Creek, Addicks Reservoir

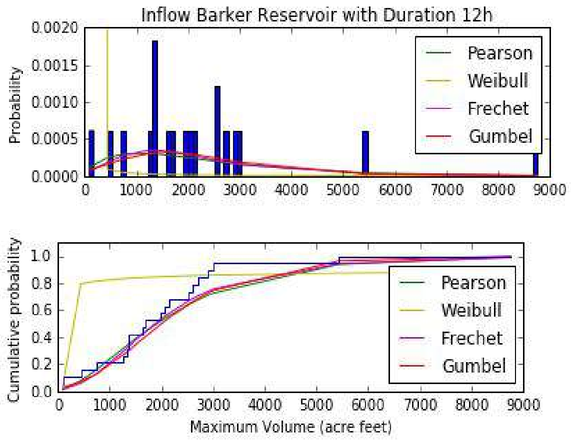
	1h	**2h**	**4h**	**8h**	**12h**	**24h**	**48h**	**72h**	**96h**
Log-Pearson	0.31678	0.31883	0.32012	0.31488	0.30453	0.29180	0.27968	0.28117	0.29388
Weibull	0.42324	0.40471	0.41976	0.42849	0.41917	0.39047	0.28198	0.26987	0.29389
Frechet	0.32770	0.33088	0.33569	0.34030	0.33026	0.31464	0.30296	0.29427	0.30229
Gumbel	0.33213	0.33495	0.33998	0.34512	0.33648	0.31908	0.30413	0.29490	0.29927
Best fit	Log-Pearson	Log-Pearson	Log-Pearson	Log-Pearson	Log-Pearson	Log-Pearson	Log-Pearson	Weibull	Log-Pearson



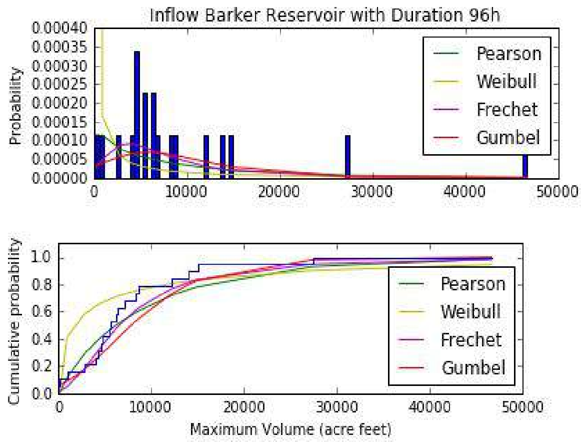
(a) Duration 2h



(c) Duration 24h



(b) Duration 12h



(d) Duration 96h

Figure E.5: Upper Buffalo Bayou, Barker Reservoir

Table E.6: RMSE of the volumes of Upper Buffalo Bayou, Barker Reservoir

	1h	**2h**	**4h**	**8h**	**12h**	**24h**	**48h**	**72h**	**96h**
Log-Pearson	0.24309	0.23700	0.24284	0.24161	0.24650	0.25913	0.23884	0.24208	0.24223
Weibull	0.41253	0.40224	0.42566	0.43238	0.38254	0.42284	0.23725	0.25837	0.28125
Frechet	0.25045	0.24422	0.25093	0.25108	0.25743	0.27505	0.26462	0.26631	0.27322
Gumbel	0.25337	0.24720	0.25406	0.25446	0.26148	0.28044	0.27016	0.27215	0.27913
Best fit	Log-Pearson	Log-Pearson	Log-Pearson	Log-Pearson	Log-Pearson	Log-Pearson	Weibull	Log-Pearson	Log-Pearson

E.4. From Volume to Elevation

Volumes in the Addicks and Barker Reservoirs

In Tables E.7 and E.10 the volumes which enter the reservoirs by the creeks is shown per duration and return period. These volumes where derived from the extrapolation of the different distributions. In Tables E.8 and E.11, the precipitation directly into the reservoirs can be seen. The precipitation is in the lower durations and return periods a bigger percentage of the total volume than in the higher durations. The volume from the creeks and the precipitation are added. The results can be found in Tables E.9 and E.12.

Table E.7: Total Volume caused by Creeks in Addicks Reservoir

	1h	**2h**	**4h**	**8h**	**12h**	**24h**	**48h**	**72h**	**96h**
10	2515.27	4877.78	8795.19	13905.57	17121.59	22832.17	24628.49	29628.50	38622.57
25	3461.74	6730.43	12338.75	19637.02	24341.15	33099.99	47271.54	55233.78	58527.72
50	4257.38	8294.16	15417.51	24684.24	30783.65	42596.63	62733.11	74245.69	78385.87
100	5140.28	10035.84	18937.98	30525.16	38328.15	54080.01	82394.61	98927.61	104101.06
250	6469.95	12670.90	24437.62	39781.22	50454.92	73260.95	117279.87	143815.35	150748.12
500	7622.55	14965.85	29386.19	48229.38	61681.15	91707.91	152888.12	190750.04	199417.35
1000	8924.86	17569.92	35166.44	58221.10	75124.29	114547.68	199332.40	253265.72	264136.44
10000	14665.58	29164.47	62707.94	107212.30	142958.47	239102.10	485611.67	658224.37	681982.49

Table E.8: Total Volume caused by precipitation in Addicks Reservoir

	1h	**2h**	**4h**	**8h**	**12h**	**24h**	**48h**	**72h**	**96h**
10	4997.77	7276.61	9541.35	11548.75	13123.17	15349.54	18586.63	19737.16	20802.64
25	6069.36	8845.45	11664.98	14161.62	16191.61	19047.51	23197.01	24602.69	25962.02
50	6864.32	10009.30	13240.41	16100.01	18467.95	21790.88	26617.25	28212.22	29789.55
100	7653.41	11164.56	14804.21	18024.07	20727.48	24513.99	30012.24	31795.10	33588.81
250	8692.38	12685.65	16863.21	20557.42	23702.52	28099.42	34482.29	36512.55	38591.17
500	9476.88	13834.19	18417.91	22470.30	25948.92	30806.69	37857.53	40074.59	42368.34
1000	10260.82	14981.90	19971.49	24381.80	28193.68	33512.01	41230.33	43634.05	46142.77
10000	12863.63	18792.51	25129.64	30728.30	35646.71	42494.15	52428.63	55452.11	58674.58

Table E.9: Total Volume in Addicks Reservoir

	1h	**2h**	**4h**	**8h**	**12h**	**24h**	**48h**	**72h**	**96h**
10	7513.04	12154.39	18336.55	25454.31	30244.76	38181.70	43215.12	49365.66	59425.21
25	9531.09	15575.88	24003.73	33798.65	40532.75	52147.50	70468.55	79836.47	84489.74
50	11121.70	18303.46	28657.92	40784.25	49251.60	64387.51	89350.36	102457.91	108175.42
100	12793.69	21200.40	33742.20	48549.23	59055.63	78594.01	112406.84	130722.71	137689.88
250	15162.33	25356.55	41300.83	60338.64	74157.45	101360.36	151762.16	180327.89	189339.28
500	17099.43	28800.04	47804.10	70699.69	87630.06	122514.60	190745.65	230824.63	241785.68
1000	19185.68	32551.82	55137.93	82602.90	103317.97	148059.69	240562.73	296899.77	310279.21
10000	27529.20	47956.97	87837.58	137940.61	178605.19	281596.25	538040.30	713676.48	740657.07

Table E.10: Total Volume caused by Upper Buffalo Bayou in Barker Reservoir

	1h	2h	4h	8h	12h	24h	48h	72h	96h
10	411.29	816.59	1611.11	3085.52	4414.29	7866.87	13071.88	16977.99	19769.22
25	537.77	1070.05	2125.83	4152.01	6034.32	11094.85	19266.04	25711.37	30630.29
50	639.41	1274.42	2544.57	5043.50	7417.27	13963.29	25076.83	34166.16	41445.14
100	747.39	1492.20	2994.15	6023.04	8964.28	17284.16	32124.07	44700.99	55251.07
250	901.18	1803.43	3642.29	7473.66	11303.49	22510.07	43830.87	62762.47	79600.47
500	1026.74	2058.38	4177.85	8704.49	13329.45	27217.32	54949.66	80452.87	104115.45
1000	1160.93	2331.68	4756.25	10064.61	15608.31	32695.34	68498.35	102594.33	135543.65
10000	1677.12	3389.79	7033.08	15699.80	25435.02	58214.50	138676.06	224677.51	318905.06

Table E.11: Total Volume caused by precipitation in Barker Reservoir

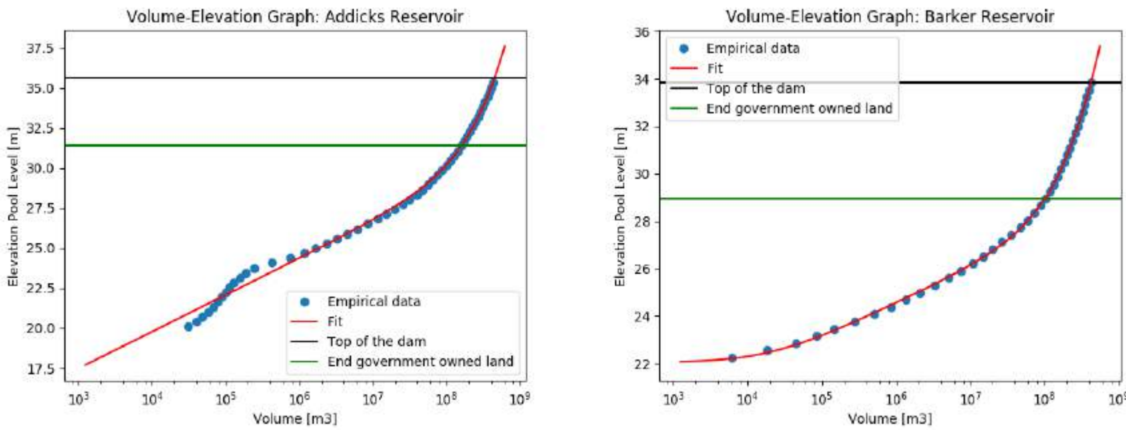
	1h	2h	4h	8h	12h	24h	48h	72h	96h
10	4735.09	6894.14	9039.85	10941.73	12433.40	14542.75	17609.70	18699.75	19709.24
25	5750.35	8380.52	11051.86	13417.28	15340.56	18046.36	21977.75	23309.55	24597.43
50	6503.52	9483.20	12544.49	15253.77	17497.26	20645.53	25218.22	26729.36	28223.78
100	7251.14	10577.74	14026.09	17076.71	19638.03	23225.52	28434.77	30123.92	31823.35
250	8235.50	12018.88	15976.86	19476.91	22456.70	26622.49	32669.87	34593.41	36562.78
500	8978.77	13107.05	17449.85	21289.24	24585.02	29187.46	35867.71	37968.23	40141.42
1000	9721.50	14194.44	18921.77	23100.27	26711.80	31750.59	39063.23	41340.61	43717.47
10000	12187.50	17804.76	23808.81	29113.20	33773.09	40260.62	49672.94	52537.50	55590.59

Table E.12: Total Volume in Barker Reservoir

	1h	2h	4h	8h	12h	24h	48h	72h	96h
10	5146.37	7710.73	10650.97	14027.26	16847.69	22409.62	30681.59	35677.74	39478.46
25	6288.12	9450.57	13177.69	17569.29	21374.89	29141.21	41243.80	49020.92	55227.72
50	7142.93	10757.63	15089.06	20297.28	24914.52	34608.83	50295.06	60895.52	69668.92
100	7998.53	12069.94	17020.23	23099.75	28602.31	40509.67	60558.84	74824.91	87074.43
250	9136.69	13822.31	19619.15	26950.57	33760.19	49132.55	76500.74	97355.88	116163.25
500	10005.51	15165.44	21627.70	29993.74	37914.47	56404.78	90817.36	118421.10	144256.87
1000	10882.43	16526.12	23678.02	33164.87	42320.11	64445.92	107561.57	143934.93	179261.12
10000	13864.62	21194.54	30841.89	44812.99	59208.11	98475.11	188349.00	277215.01	374495.66

Volume-Elevation graphs

The volumes are transformed into elevation via the Volume- Elevation graphs shown in Figure E.6. These Empirical data was measured by USACE [5].From these data points a fit was found and used to calculate the volumes.



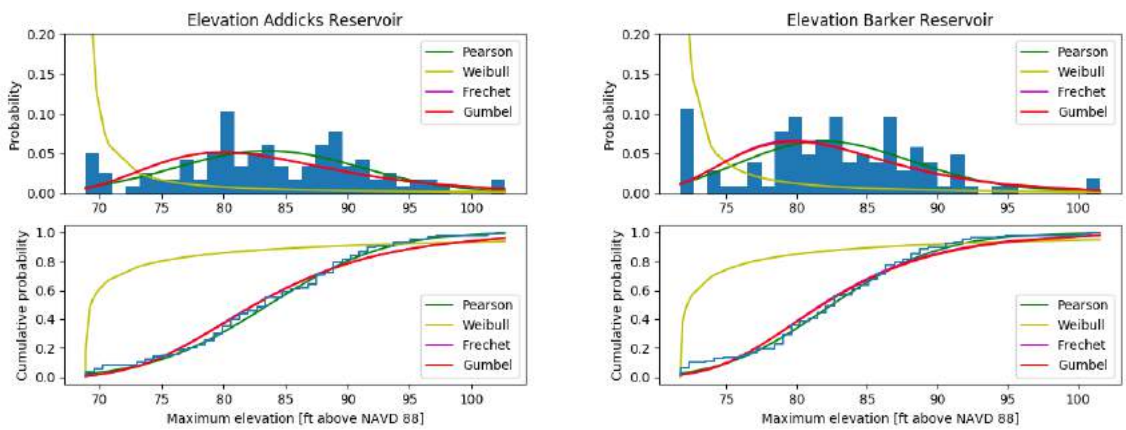
(a) Addicks

(b) Barker

Figure E.6: Volume Elevation graph

E.5. Method 2: Water levels

The best fit according to the RMSE would be Gumbel for both elevations as can be seen in Table E.13. When looking at the CDF of both the Addicks and Barker Reservoirs the Gumbel underestimates the upper tail of the data, the same for Fréchet as depicted in Figure E.7. Therefore Log-Pearson is chosen to be the best fit.



(a) Addicks

(b) Barker

Figure E.7: Elevation at Addicks and Barker Reservoirs

Table E.13: RMSE of the elevations of the Addicks and Barker Reservoirs

	Elevation Addicks	**Elevation Barker**
Log-Pearson	0.34586	0.34325
Weibull	0.46371	0.45139
Frechet	0.34305	0.34411
Gumbel	0.34268	0.34193
Best fit	Gumbel	Gumbel

F

Geometry

The geometry of the different dam section can be found in Figure F.2 to F.9. Dam section A1 is not included, because this section only consists out of the spillway. The legend of the drawings can be found in Figure F.1. The soil properties corresponding with the different soil types can be found in Table 5.1.

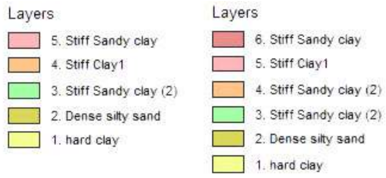


Figure F.1: left: Legend A4, right: Legend A2-A3 and B1-B5

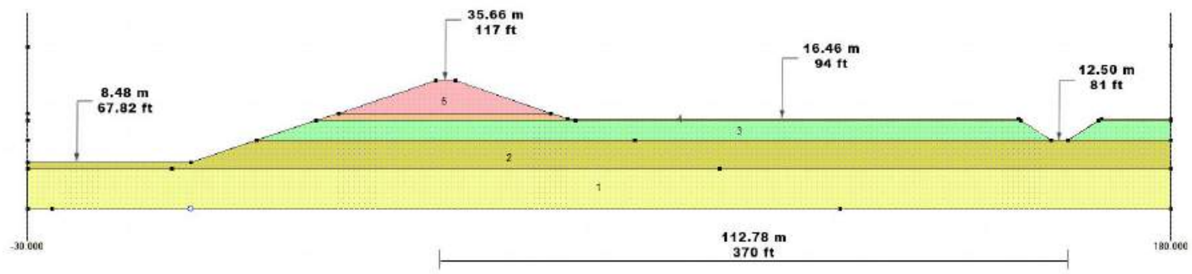


Figure F.2: Addicks section A2

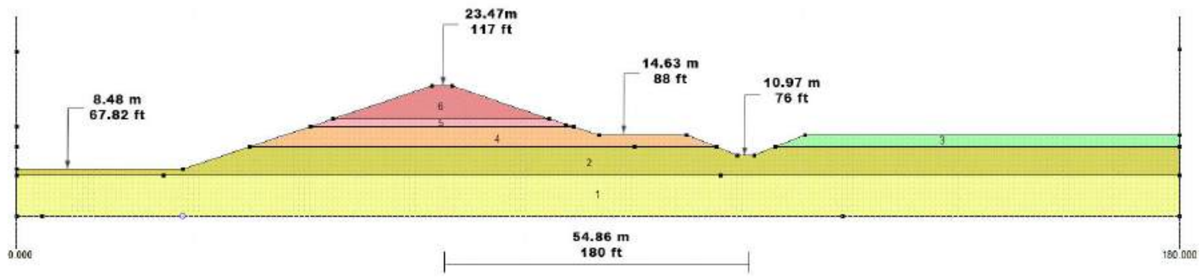


Figure F.3: Addicks section A3

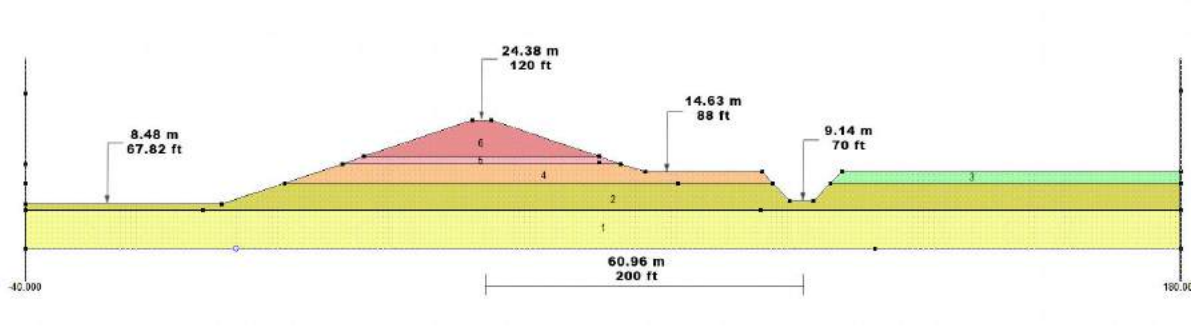


Figure F.4: Addicks section A4

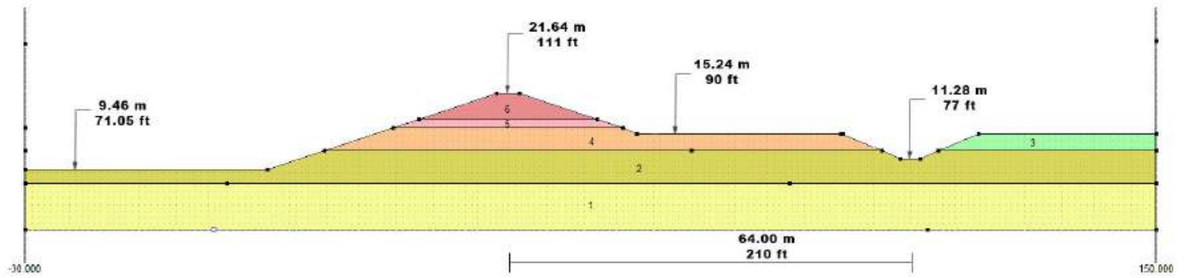


Figure F.5: Barker section B1

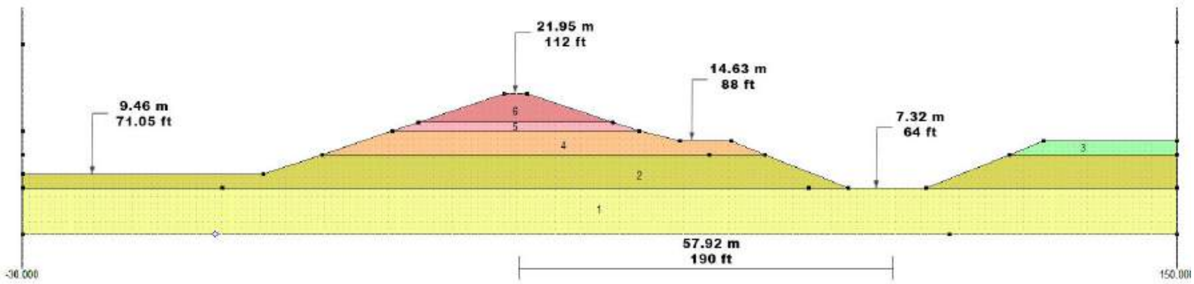


Figure F.6: Barker section B2

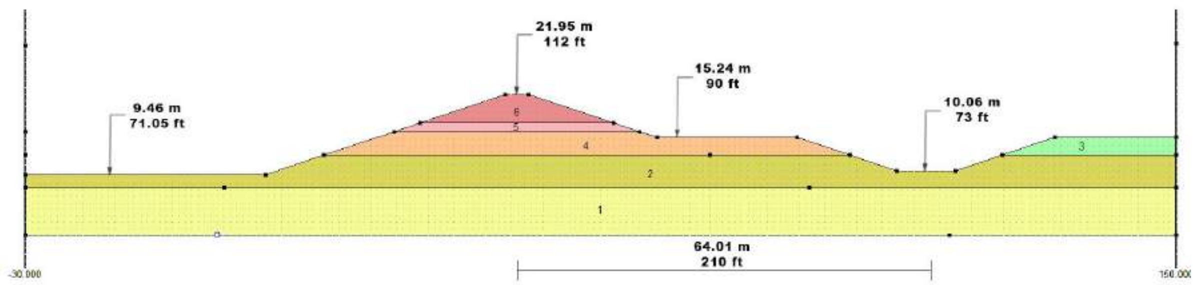


Figure F.7: Barker section B3

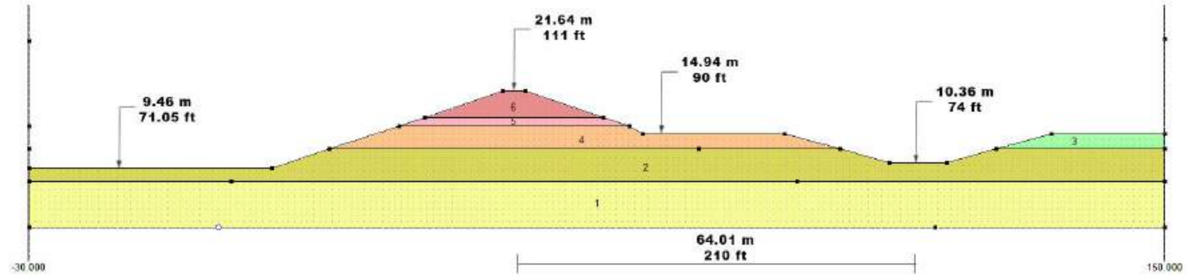


Figure F.8: Barker section B4

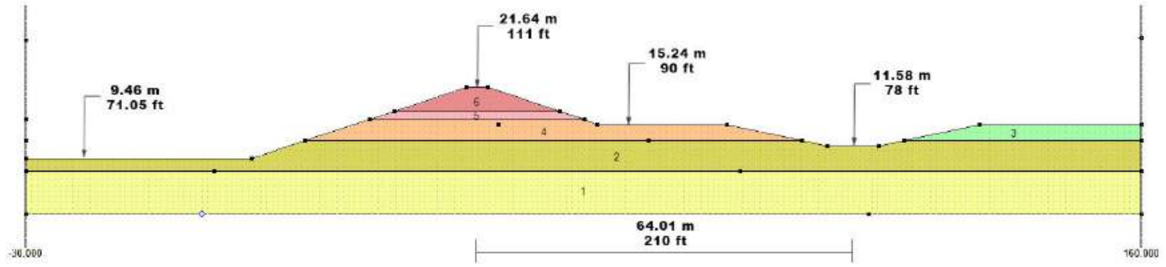


Figure F.9: Barker section B5

G

Failure Mechanisms

Figure G.1 gives an schematic overview of the most relevant failure mechanisms of earthen embankments functioning as flood deferences. The failure mechanisms that are considered in this report are briefly described below:

- *Overflow*
  If the still water level is higher than the crest level of the dam, the term "overflow" is used. The water can flow over the crest into the protected area, where the discharge itself can cause flooding. Even worse, the overflow can cause infiltration and erosion at the inner slope which can eventually lead to a breach.
- *Wave overtopping*
  If the still water level is lower than the crest level of the dam, but the waves runs over the crest, the term "overtopping" is used. This is normally expressed in a critical discharge to prevent the inner slope from erosion. The term "overtopping" can be subsided into two terms: overtopping and wave run-up. During overtopping the waves break on the crest and during run-up the waves break on the slope of the dam.
- *Stability*
  Macro stability includes both inner as outer slope failure and horizontal shearing. Due to changes in hydrological condition, the embankment can become unstable and sliding planes can occur which lead to failure of the dam. Shearing of the dam means that the whole dam can be pushed from it's place due to the water pressure when the shear capacity of the dam is to small. Micro stability is not considered in this report.
- *Piping*
  Piping, or Backward Internal Erosion, is the process of forming channels in the subsoil of the dam due to high hydraulic gradients. If these 'pipes' grow underneath the whole cross section, the dam can collapse or slide.

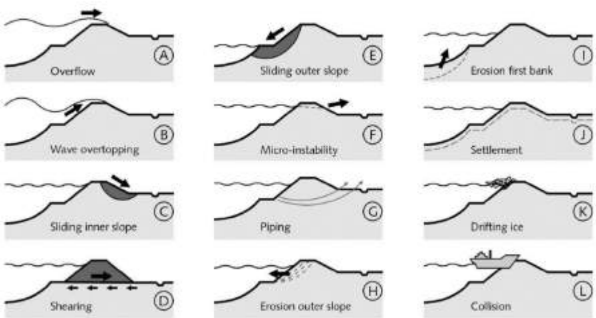


Figure G.1: Failure mechanisms

G.1. Wind Set-up and Waves

Wind Set-up

Due to friction between the air and the water, the water will be pushed up. By choosing a factor on the conservative side, the friction coefficient following CUR-CIRIA [1991] is 2.7*10^{-3}.

$$\kappa = c_w \frac{\rho_{air}}{\rho_{water}} = 2.7 * 10^{-3} \frac{1.21}{1030} = 3.2 * 10^{-6} \tag{G.1}$$

The actual water level difference depends on the wind speed, fetch, water depth and the approach angle of the wind. In the Netherlands the *potential wind speed (u_{10})* is taken, which is derived from hourly averaged wind speed at a height of 10 m [32.81 ft][39]. Since Houston is located in a hurricane prone-area and an extreme event is considered, this parameter is comparable with the *basic wind speed (ASCE 7-93)*. This is the annual, extreme, fastest-mile speeds having an annual probability of exceedance of 0.02 for 10 m [32.81 ft] above ground level in flat, open country terrain, also called 'fastest mile' [15]. At the Addicks and Barker Reservoirs, this wind speed is 38.9 m/s [87 mph] [13].

A hurricane causes a circular wind around the eye. The eye-width of the hurricane is typically 32.2 till 64.4 km [20-40 miles] [22], which is significantly larger than the size of the reservoirs. This is why the assumption is made to consider the fetch straight instead of curved. The fetch depends on the angle of the wind approaching the dam and is calculated per section of the dam. With Formula G.2 the rise of the water level is calculated and included in overtopping and the run-up calculations.

$$\delta h = 0.5\kappa \frac{u_{10}^2}{gh} F cos(\phi) \tag{G.2}$$

Wind Waves

In order to calculate the wave height and wave period the formulas of Breugem and Holthuijsen (2006) are used[7]. The fetch that is used in the formulas is determined by measuring the longest possible fetch in Figure 3.3 and 3.4 for each dam section. As elaborated in Paragraph 5.1.2 a reduction factor is used to take into account the effect of vegetation on the wave height.

$$H_s = \gamma_{veg} \frac{0.240 * U^2 c_1^{0.572}}{g} tanh\left(\frac{4.410 * 10^{-4}}{c_1}\left(\frac{gF}{U^2}\right)^{0.790}\right)^{0.572} \tag{G.3}$$

$$T_p = \frac{7.690 * U^2 c_2^{0.187}}{g} tanh\left(\frac{2.770 * 10^{-7}}{c_2}\left(\frac{gF}{U^2}\right)^{1.450}\right)^{0.187} \tag{G.4}$$

Where:

$$c_1 = tanh\left(0.343\left(\frac{gd}{U^2}\right)^{1.14}\right) \tag{G.5}$$

$$c_2 = tanh\left(0.100\left(\frac{gd}{U^2}\right)^{2.01}\right) \tag{G.6}$$

$$\gamma_{veg} = (1 - 0.0074)^{(0.8F/100)} \tag{G.7}$$

G.2. Overtopping

The maximum allowed overtopping discharge for the dams are set on 10 l/s/m. This discharge can lead to severe erosion on the land side of the dams which can lead to possible failure of the dam. Smaller overtopping discharges may lead to hindrance due to the water, but not to failure of the dams [37]. The formulas that are used to calculate the wave run-up and wave overtopping can be found below. The wave height and wave period in the formulas are calculated using formulas G.3 and G.4 in Appendix G.1.
Since there is no vertical wall and no berm on the Addicks and Barker Dams, γ_b and γ_v are equal to 1.0. Since the dam has a grass cover, γ_f is equal to 1.0 as well, because this smooth type of cover does not reduce the wave height.

Run-up formula [37]:

$$\frac{R_{u2\%}}{H_{m0}} = 1.07 * \gamma_f * \gamma_\beta * \left[0.578\left(4.0 - \frac{1.5}{\sqrt{\gamma_b * \xi_{m-1,0}}}\right)\right] \tag{G.8}$$

Where:

$$\gamma_\beta = 1.0 - 0.0022 * \beta \qquad 0 < \beta < 80 \qquad Degrees \tag{G.9}$$

$$\gamma_\beta = 0.824 \qquad \beta > 80 \qquad Degrees \tag{G.10}$$

Overtopping formula[37]:

$$\frac{q}{\sqrt{g * H_{m0}^3}} = 0.09 * exp\left[-\left(1.5 * \frac{R_c}{H_{m0}}\right)^{1.3}\right] \tag{G.11}$$

Where:

$$\gamma_\beta = 1.0 - 0.0033 * \beta \qquad 0 < \beta < 80 \qquad Degrees \tag{G.12}$$

$$\gamma_\beta = 0.736 \qquad \beta > 80 \qquad Degrees \tag{G.13}$$

In the above formulas, the unknowns are the wave height (because it is depended on the water depth) and the free board (which is dependent on the wave height). These formulas can only be solved by using an iteration on the water depth, which will lead to a certain water depth matching with a free board. The lowest water level per dam section that is encountered will be considered the critical water level for the over topping mechanism.

G.3. Stability

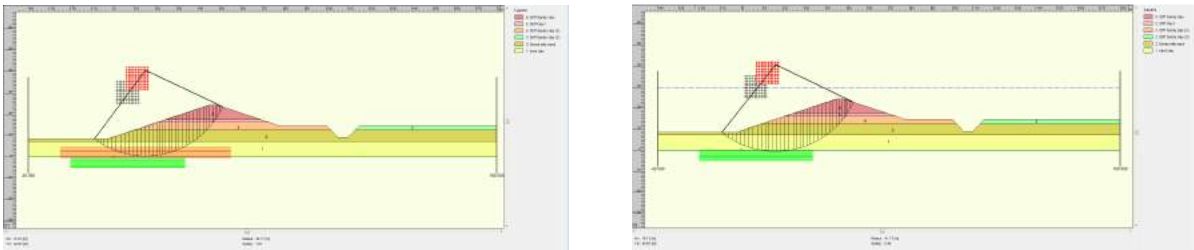
Macro stability includes both inner- as outer slope failure and horizontal shearing. Due to changes in hydrological condition, the embankment can become unstable and sliding planes can occur which lead to failure of the dam.

Inner- and outer slope failure

The safety factors of both the inner- and outer slopes for all the dams are calculated using D-Geostability from Deltares . For the modeling of the slip planes the method of Bishop is used, which is based on a circular slip plane. This plane is devided in multiple slices with vertical interfaces as depicted in Figure G.2 and G.3. Using the method of Bishop the cohesion and friction between the different slices are taken into account. The safety factor is the ratio between the driving moment and the resisting moment, the formula according to Bishop can be found in Formula G.14. As can be seen from this formula, retrieving the safety factor for a certain sliding plane is an iterative process. When the safety factor is smaller than one, this means that the dam is likely to fail[6][16].

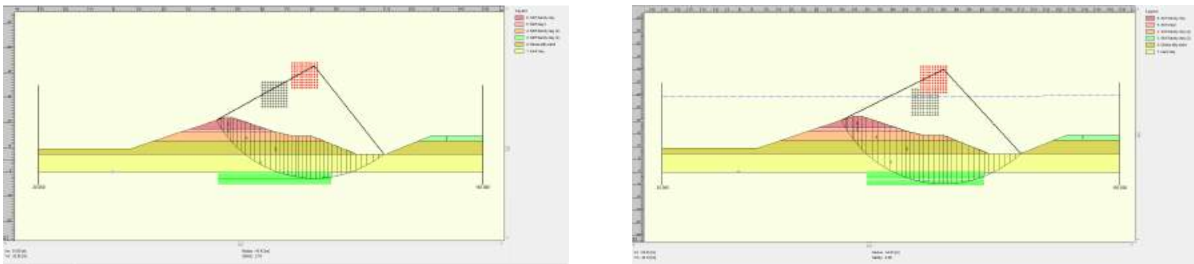
$$F = \frac{\sum \frac{c+(\gamma h-p)tan(\phi)}{cos\alpha(1+tan(\alpha)tan(\phi)/F}}{\sum \gamma hsin(\alpha)} \tag{G.14}$$

D-Geostability calculates the sliding planes using a starting grid. From each point of the grid it calculates the different sliding planes with their according safety factor. If the lowest safety factor is detected at one of the edges of the grid, the grid will move and start over with the calculations until it finds the most critical sliding plane with the lowest safety factor[16]. The most critical slip planes for each situation are depicted in Figure G.2 and G.3.



(a) Dry A4 (b) Saturated A4

Figure G.2: Critical sliding planes inner slope



(a) Dry B2 (b) Saturated B2

Figure G.3: Critical sliding planes outer slope

Shearing
In order to check if the dams are able to resist shearing Formula G.15 is used.

$$W \geq T \qquad \rightarrow \qquad B_i * \tau_i \geq 0.5 * \gamma_w * h_i^2 \tag{G.15}$$

Where:

$$i = V, IV, III \tag{G.16}$$

$$B_i = 2 * 3 * h_i + B_c \tag{G.17}$$

$$\tau_i = c_i + \sigma'_i tan(\phi_i) \begin{cases} \sigma'_V = (\gamma_s - \gamma_w) * (h - h_V) \\ \sigma'_{IV} = \sigma'_V + (\gamma_s - \gamma_w) * (h - h_{IV}) \\ \sigma'_{III} = \sigma'_{IV} + (\gamma_s - \gamma_w) * (h - h_{III}) \end{cases} \tag{G.18}$$

Dividing the hydraulic horizontal force by the shear force gives a factor of maximum 0.19 when the water level is as high as the crest of the dam. Only if this value exceeds 1 there is a chance that shearing can exist. So the dams are safe against horizontal shearing.

G.4. Piping

To check the dams on piping, first some characteristics of the soil composition and the layers itself need to be determined. The different components of piping (uplift, heave and backward internal erosion) are used to calculate the critical water level. The aquitard consists of Very Stiff Sandy Clay (CL) and the aquifer of Dense Silty Sand (SM). The saturated weight of CL and the moist weight of SM is both 20.89 kN/m^3. Since no additional properties of the existing soils are available, a standardized hydraulic conductivity is calculated for these materials by Formula G.19 (Hazen, 1982). A 10-percent particle size of silty sand of 0.02 mm, gives a hydraulic conductivity of 4*10^{-6} m/s. As the ratio between CL and water is between the range of silty and sandy clay, the 10-percent particle size of 0.003 mm is taken, which gives a hydraulic conductivity of 9*10^{-9} m/s [10]. The thickness of the layers and hydraulic conductivity give a leakage factor:

$$k = C_H * (D_{10})^2 \tag{G.19}$$

$$\lambda_h = \sqrt{\frac{kDd}{k_h}} \tag{G.20}$$

$$\lambda = \frac{\lambda_h}{L_f + B + \lambda_h} e^{\frac{\frac{B2}{-} x_{exit}}{\lambda_h}} \tag{G.21}$$

Since uplift and heave are checked at the must vulnerable location, the toe of the dam, the second part of the Equation G.21 becomes 1, since the exponent becomes 0.

a Uplift

To prevent the blanket from *lifting up* and rupture, the weight of the aquitard needs to be higher than the pore pressure underneath.

$$\Delta\phi \geq \Delta\phi_{c,u} \tag{G.22}$$

The formulas for uplift are rewritten to determine the critical height for uplifting of the blanket. The maximum elevation of the ground water during Harvey is used to define the elevation of the outer water in the ditch at a level of 1.52 m [3 ft] below the land elevation of 28.65 m [94 ft], so at an elevation of 27.74 m [91 ft] [11].

$$\left.\begin{array}{r}\Delta\phi_{c,u} = d\frac{\gamma_{sat}-\gamma_w}{\gamma_w}\\ \Delta\phi = \phi_{exit} - h_p\\ \phi_{exit} = h_p + \lambda(h - h_p)\end{array}\right\} h_{uplift} \geq \frac{d(\gamma_{sat} - \gamma_w)}{\lambda\gamma_w} + h_p \tag{G.23}$$

b Seepage

When the aquitard is ruptured, water can *seep* from the aquifer *through* the blanket and form water boils at the downstream side of the dam.

c Heave

If the water starts to take aquifer material with the seepage, erosion occurs which is called *heave*. The heave gradient is required to be larger than the critical gradient to prevent erosion of the silty sand layer. The higher the porosity of the aquifer, the easier a piping problem occurs, so the upper limit of the void ratio for Dense Silty Sand (SM) of 0.49 is chosen. Formula G.25 gives a porosity of 0.329 [24]. Rewritten the formulas for heave, gives the critical water elevation per dike section, see Formula G.26.

$$i \geq i_{ch} \tag{G.24}$$

$$n = max\left(\frac{emin}{1+emin}, \frac{emax}{1+emax}]\right) \tag{G.25}$$

$$\left.\begin{array}{r}i_{ch} = \frac{(1-n)(\gamma_s-\gamma_w)}{\gamma_w}\\ i = \frac{\Delta\phi}{d} = \frac{\phi_{exit}-h_p}{d}\\ \phi_{exit} = h_p + \lambda(h - h_p)\end{array}\right\} h_{heave} \geq \frac{i_{ch}d}{\lambda} + h_p \tag{G.26}$$

d Backward erosion

For piping resistance, the critical head difference has to be compared to the driving force of internal erosion (head difference between waterside and land side water level) and the erosion resistance of the sand grains in a partially developed piping channel. The critical head difference can be determined with the help of the seepage length, which is the combination of the length of the foreshore, the width of the levee and the length of the hinterland. For the lane of creep an combination of the foreshore, the width of the levee and hinterland is taken. Since a flood in considered, there is no foreshore and in case of looking to the toe, there is also no hinterland. Since the water will flow through to the Dense Silty Sand layer at an elevation of 24.96 m [81 ft], the width of the levee is taken at this elevation.

$$L = L_f + B + L_h \tag{G.27}$$

Bligh and Lane

For Lane the depth of the aquifer is taken for the vertical distance the water needs to bridge. The creep factor for silty sand is 18 for Bligh and 8.5 for Lane.

$$L \geq H * C_{Bligh} \qquad \rightarrow \qquad h_{Bligh} \leq L/C_{Bligh} + h_p \tag{G.28}$$

$$L_v + 1/3 * L_h \geq H * C_{Lane} \qquad \rightarrow \qquad h_{Lane} \leq \frac{L_v + 1/3 * L_h}{C_{Lane}} + h_p \tag{G.29}$$

Sellmeijer

$$H_{c,p} = F_R F_S F_G L \tag{G.30}$$

Where:

$$F_R = \eta(\frac{\gamma_s}{\gamma_w} - 1) tan(\theta) \tag{G.31}$$

$$F_S = \frac{d_{70m}}{\sqrt[3]{\frac{\nu k l}{g}}} \left(\frac{d_{70}}{d_{70m}}\right)^{0.4} \tag{G.32}$$

$$F_G = 0.91(\frac{D}{L})^{\frac{0.28}{(\frac{D}{L})^{2/8}-1}+0.04} \tag{G.33}$$

$$H_{c,p} \leq h - h_p - 0.3d \qquad \rightarrow \qquad h_{BIE} \geq H_{c,p} + h_p + 0.3d \tag{G.34}$$

e Continuous pipe

If backward internal erosion will continue, the pipe will grow till a continuous pipe, which could eventually lead to collapsing of the aquifer and so the levee.

f Collapse

The results of these calculations can be found in Tables 5.5 and 5.6 in the metric system and and in the Tables G.1 and G.2in the American system . The total outcome of the failure mechanisms are shown in Table 5.7 in the metric system and in Table G.3 in the American system.

Table G.1: Piping at the toe per dam section

Critical elevations [ft]	**A1**	**A2**	**A3**	**A4**	**B1**	**B2**	**B3**	**B4**	**B5**
Crest level	117	117	117	120	111	112	112	111	111
Critical water level Uplift	115.86	105.69	98.92	98.92	105.69	98.92	101.18	100.05	101.18
Critical water level Heave	107.69	100.86	96.32	96.32	100.86	96.32	97.83	97.07	97.83
Critical water level Sellmeijer	-	-	-	-	-	-	-	-	-
Critical water level Bligh	103.68	103.68	103.68	104.68	101.68	102.01	102.02	101.68	101.68
Critical water level Lane	107.93	107.93	107.93	108.64	106.90	106.90	107.13	107.13	106.90

Table G.2: Piping at the ditch per dam section

Critical elevations [m]	**A1**	**A2**	**A3**	**A4**	**B1**	**B2**	**B3**	**B4**	**B5**
Crest level	35.66	35.66	35.66	36.58	33.83	34.14	34.14	33.83	33.83
Critical water level Sellmeijer	-	-	-	-	-	-	-	-	-
Critical water level Bligh	-	36.60	33.07	33.57	33.38	32.99	33.43	33.33	33.38
Critical water level Lane	-	36.42	33.94	34.28	34.27	33.99	34.30	34.23	34.27

Table G.3: Critical water level per failure mechanism per dike section [ft]

	A1	**A2**	**A3**	**A4**	**B1**	**B2**	**B3**	**B4**	**B5**
Overtopping	**115.32**	115.42	115.41	118.86	**107.11**	95.11	108.96	107.94	106.68
Stability	-	-	-	-	-	-	-	-	
Piping at the toe	115.68	103.68	-	**104.69**	105.71	101.68	102.01	**101.68**	**101.68**
Piping at the ditch	-	**120.07**	-	-	109.51	**108.23**	109.68	109.34	109.51

H
Optimization

H.1. Floodplains from HEC-RAS

The HEC-RAS model the cross-sections are defined as the orange lines shown in figure H.2. For each of these cross-sections the model returns the water elevation in the cross section as can be seen in figure H.1. This model was made to model the 1/500 year floodplain with the 1/500 year discharge in the Buffalo Bayou, the cross-sections are already preprogrammed into the model. This cross-sections are not wide enough to model the extra amount of discharge released from the Addicks and Barker Reservoirs. The cross-section in Figure H.1 is one of the cross-sections which shows a "water wall". The first blue line above the water shows the water elevation for the two times the 500 year discharge, this is not the real water elevation. The water piled up at the sides, which shows the limitations of the cross section.
This can be fixed by extending the ground elevation profiles for each section, however this is not possible done is this report. In future projects this needs to be considered.

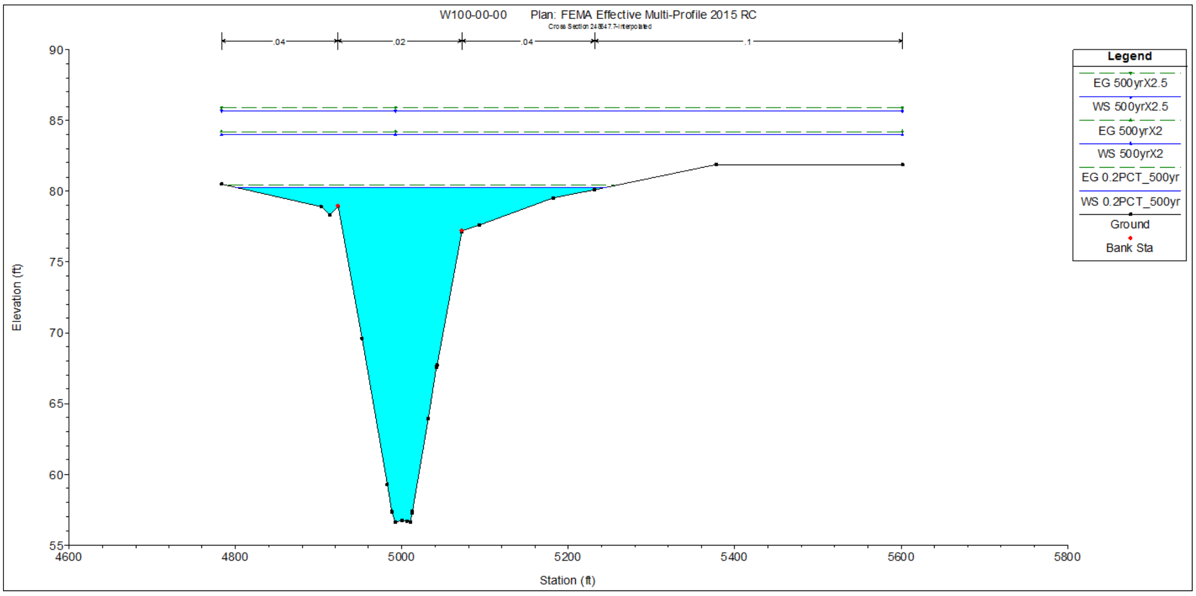


Figure H.1: One of the cross sections of HEC-RAS model

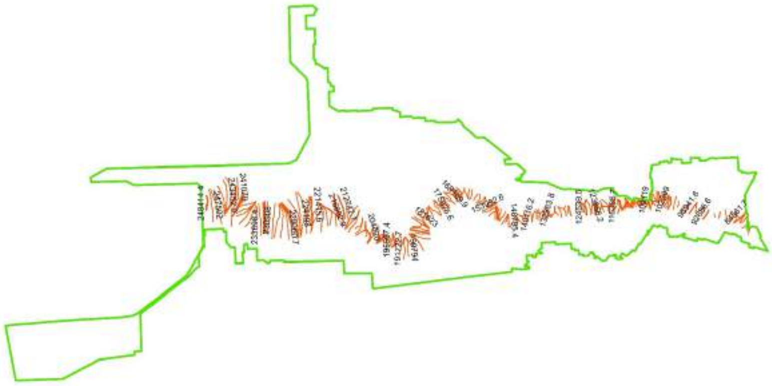


Figure H.2: All input cross section

The 1/500 year floodplain can be seen in figure H.3 and in figure H.4 the 2 times 1/500 year discharge floodplain is shown. In the 500 year floodplain more detailed limits are visible and in the two times 500 year there are less detail limits especially in the downstream end. This is the result of the max out of some of the cross sections (resulting in a non excising "water wall"), however it still gives an good estimate of the areas prone to flooding. Therefor the floodplains for the two times the 500 year discharge are still used to make cost estimates, despited their inaccuracy.

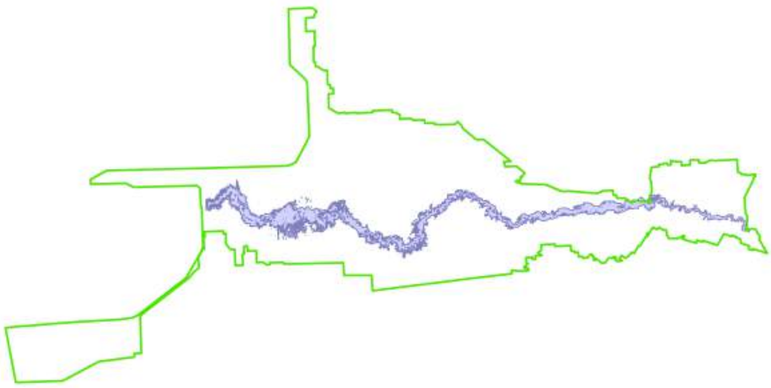


Figure H.3: Floodplain of the 1/500 year discharge

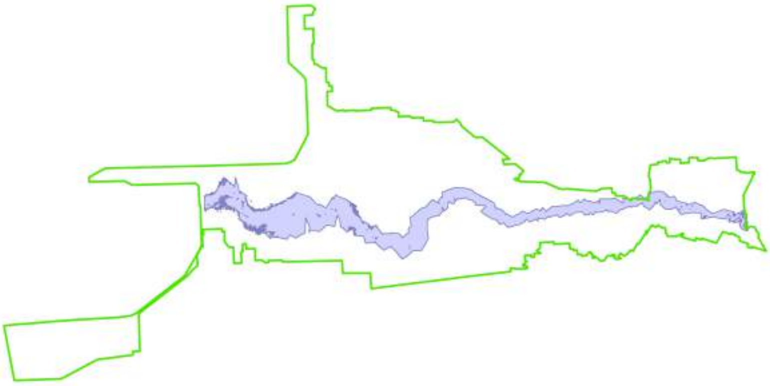
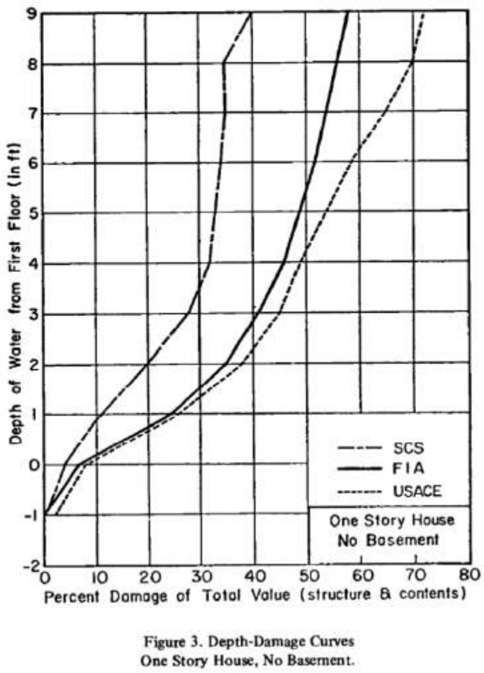


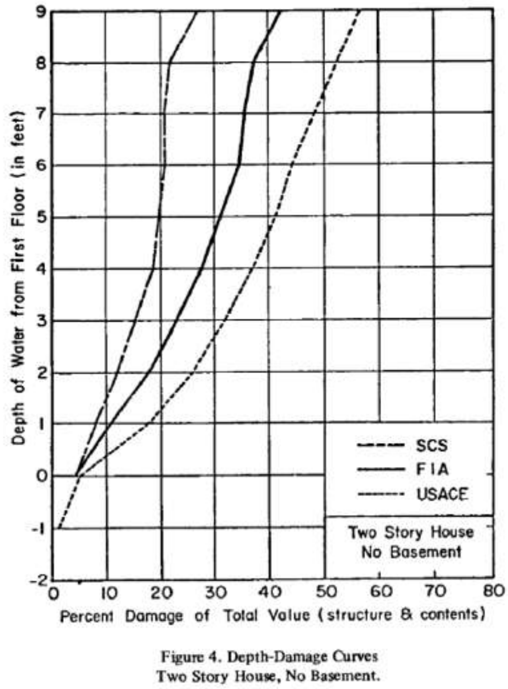
Figure H.4: Floodplain of two times the 1/500 year discharge

H.2. Inundation-damage graphs

In Figure H.5 the Inundation-damage graphs for both a one-story house and two-story house can be found. In Figure H.6 the comparison between the inundation-damage graphs for a house with and without basement can be found.



(a) One story house



(b) Two story house

Figure H.5: Inundation-damage curve [41]

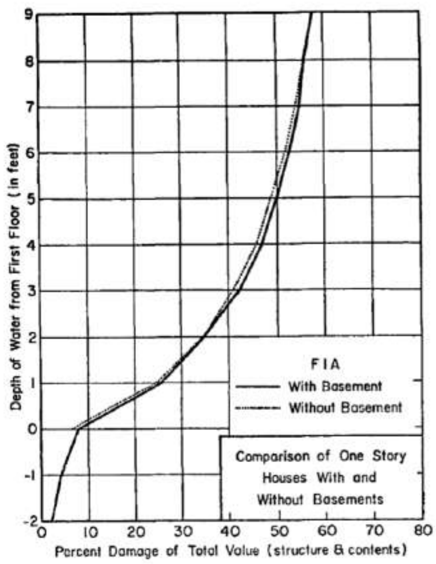


Figure H.6: House with or without basement inundation-damage curve [41]

H.3. Sub areas

The sub areas are depicted in Figures H.7, H.8 and H.9.

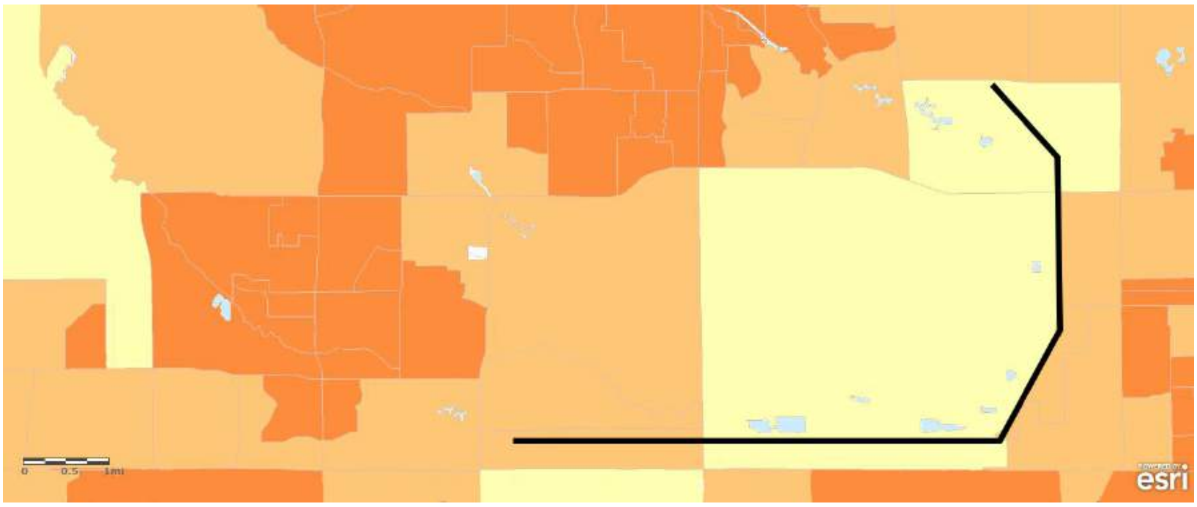


Figure H.7: Sub areas Addicks (Adjusted illustration from Esri[18])

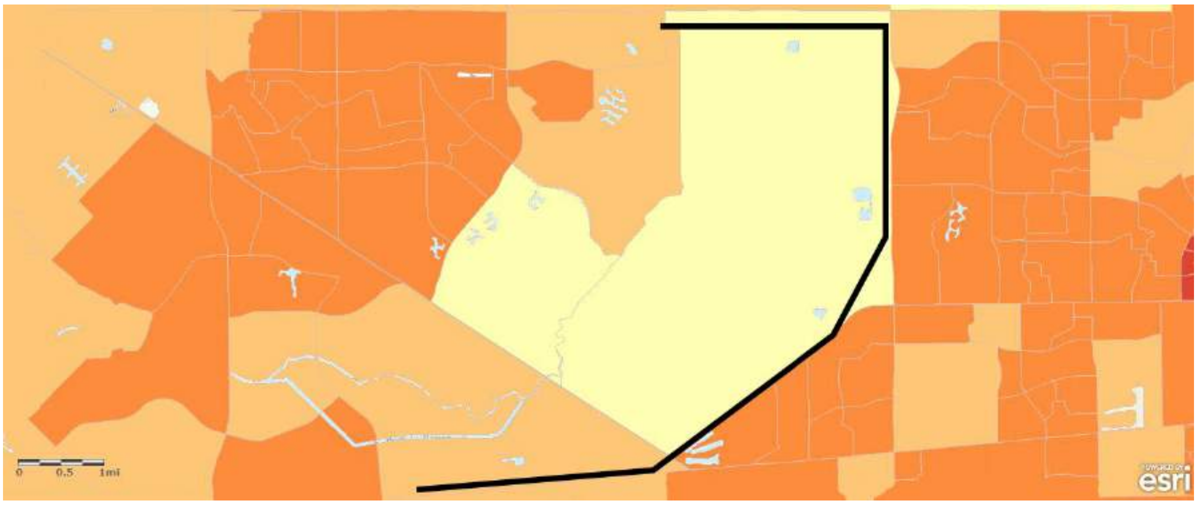


Figure H.8: Sub areas Barker (Adjusted illustration from Esri[18])

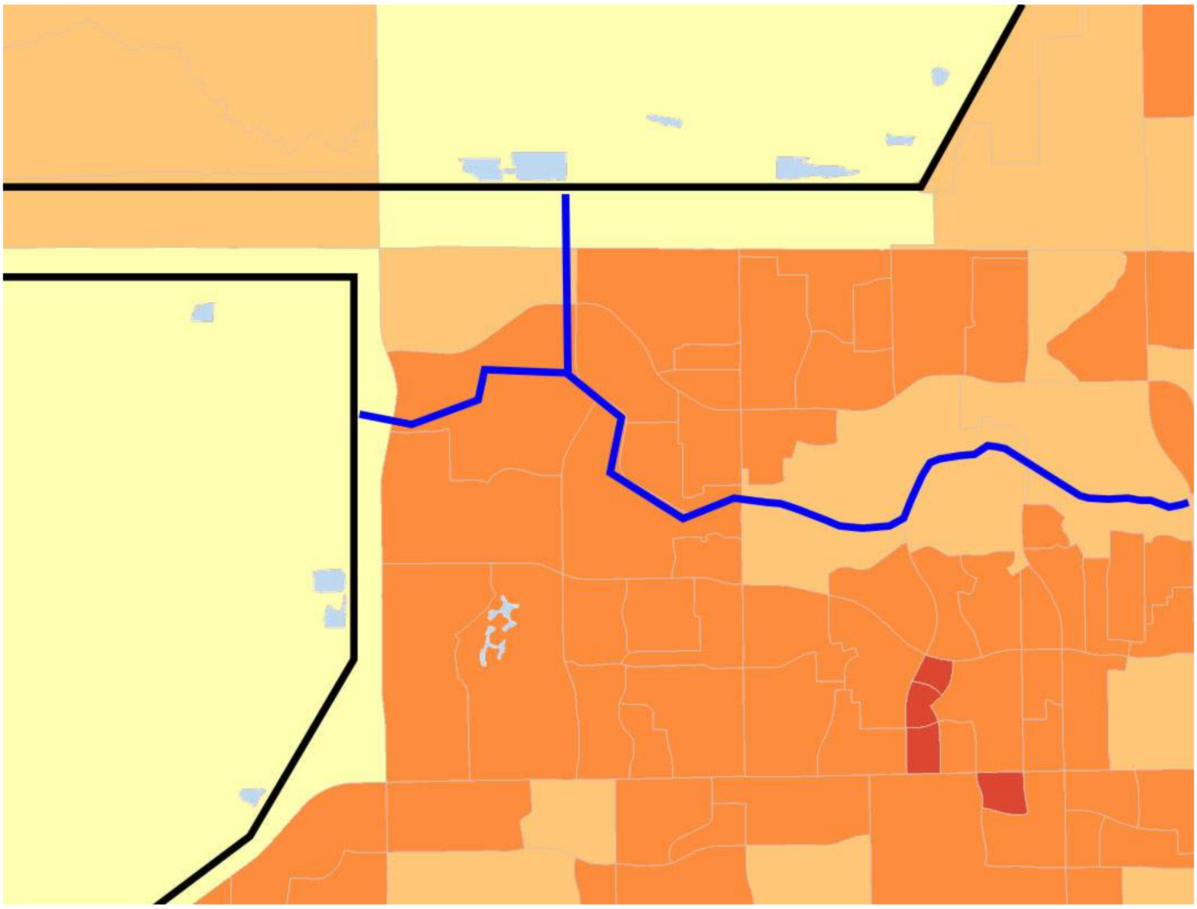


Figure H.9: (First part of) Sub areas Buffalo Bayou (Adjusted illustration from Esri[18])

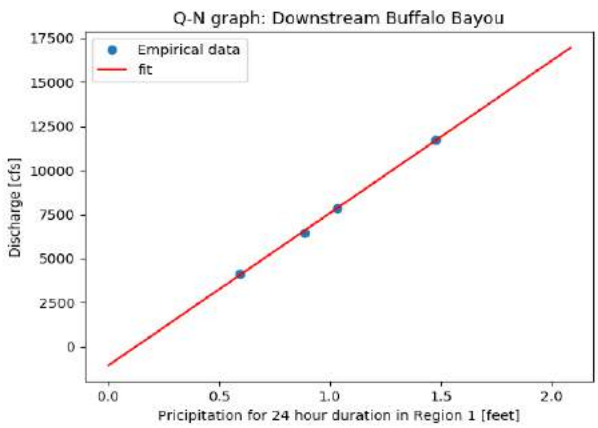
H.4. Discharge

Run-off

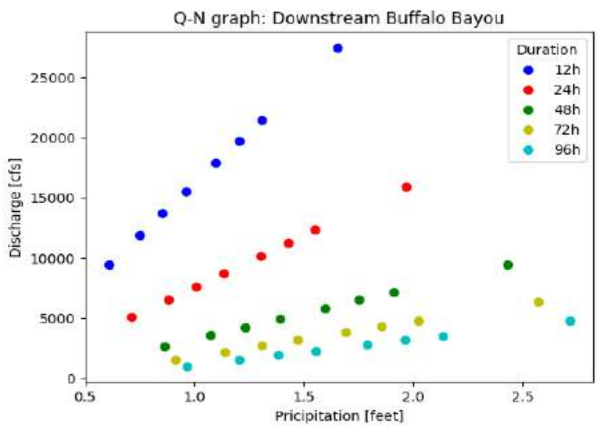
The initial discharge in the Buffalo Bayou is caused by the precipitation which falls directly onto the Buffalo Bayou watershed. This precipitation is assumed to be constant in the whole watershed. The discharge in the Buffalo Bayou therefore initially consist of only run-off. When the Addicks and Barker Reservoirs start releasing during an storm event, additional discharge will be added into the Buffalo Bayou. To calculate the amount of run-off caused by the precipitation determined in Chapter 4.3, the values of the discharge caused by run-off and the associated precipitation for 24 hour duration of the HCFCD are used[20]. This is done for return periods of 10, 50, 100, 500 years, which lead to a precipitation-discharge relation shown in figure H.10a.

$$\left.\begin{array}{c} V = A * N \\ Q = \frac{V}{t} \end{array}\right\} \quad Q = \frac{A*N}{t} - Q_{Infiltrate} \tag{H.1}$$

The Q~N relation is assumed to be linear and a part of the volume, $Q_{Infiltrate}$, which is assumed to be constant for all durations, see formula H.1. This formula is fitted onto the HCFCD data and then transformed for the different durations by changing the time step. The slope of the Q~N relation decreases with longer durations, because the same volume can be discharged during a longer period. Therefor a lower discharge over time can discharge the whole volume, which needs to be drained due to precipitation. The discharge in the Buffalo Bayou caused by run-off for all different duration and each return period is shown in Figure H.10b.



(a) HCFCD values used to make fit



(b) Results from chapter4.3 insert in fit

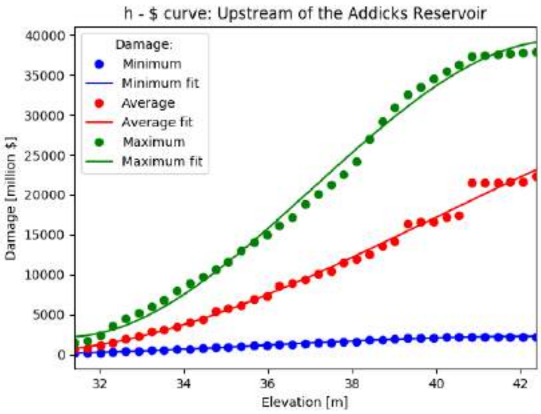
Figure H.10: Q~ N relation per duration

Q ~ h_d relation

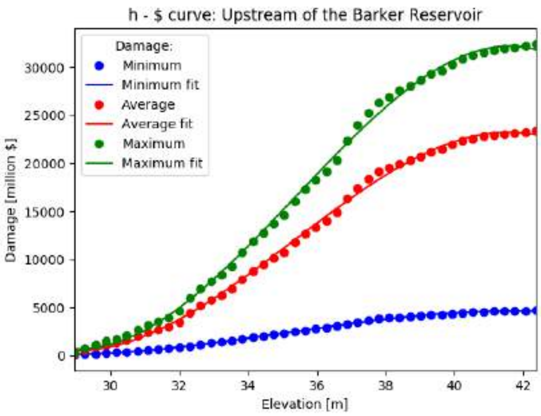
The Q~h_d is constructed from the results of the HEC-RAS model, where the discharge of the 1/10, 1/50, 1/100, 1/500 and the two times 1/500 has been used as input [32]. The HEC-RAS model returns the elevations which correspond to these discharges, which are comparable with the discharges near to this STA of the Flood Insurance Study of the FEMA [20]. The corresponding water levels are comparable with the ones in the Letter Of Map Revision of the FEMA as well [21]. The data of the discharge and water level of the HEC-RAS model are used to fit and determine the relation between those two variables.

h ~ $ relation

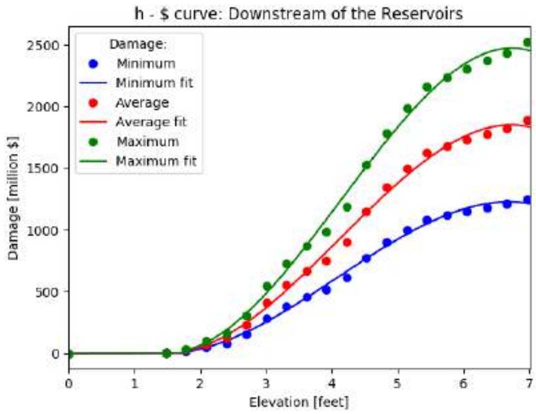
The data from Paragraph 6.2 for the flood damage per elevation is used to fit a relation between those two variables. This is done for the minimum, average and maximum damage due to flooding.



(a) Upstream of Addicks Reservoir



(b) Upstream of Barker Reservoir



(c) Downstream of the Reservoirs

Figure H.11: Q~ $ relations

H.5. Results

Table H.1: Totale damage (million $): Barker dominant

	12	24	48	72	96
10	231.29	1.12	0.00	0.00	0.00
	1,002.09	1.68	0.00	0.00	0.00
	2,182.12	2.24	0.00	0.00	0.00
25	496.52	1.99	0.00	0.00	0.00
	2,181.83	2.98	0.00	0.00	0.00
	4,768.44	3.97	0.00	0.00	0.00
50	693.28	73.65	0.50	0.00	0.00
	3,064.46	317.33	0.75	0.00	0.00
	6,708.31	691.79	1.00	0.00	0.00
100	878.10	163.23	1.03	154.30	175.06
	3,896.63	703.27	1.54	775.97	879.54
	8,539.34	1,529.65	2.05	2,221.57	2,293.50
250	1,101.00	309.13	220.60	444.83	514.72
	4,903.04	1,346.46	1,114.28	2,198.02	2,484.64
	10,755.60	2,935.85	2,541.57	4,090.69	4,527.03
500	1,253.41	427.17	475.93	713.07	848.77
	5,592.52	1,871.91	2,413.91	3,604.36	4,237.77
	12,274.90	4,088.04	4485.37	6,622.60	7,657.31
1000	1,392.60	753.00	767.36	1,112.54	1,343.91
	6,223.07	3,450.93	3,963.91	5,938.02	7,106.68
	13,664.80	7,716.44	7,482.19	11,205.30	13,102.90
10000	2,084.26	1,780.97	2,531.95	3,341.30	4,082.03
	9,580.01	8,903.43	15,893.60	23,293.00	27,616.00
	20,785.70	18,343.20	31,169.20	41,836.80	47,735.30

Table H.2: Elevation of Addicks [feet]: Addicks dominant

	12	24	48	72	96
10	93.35	94.51	95.18	95.92	97.04
25	94.83	96.24	98.15	99.03	99.44
50	95.91	97.55	99.87	100.96	101.42
100	97.00	98.91	101.75	102.90	102.93
250	98.50	100.87	104.63	105.71	105.87
500	99.72	102.53	107.22	109.12	109.30
1000	101.03	104.38	110.26	113.07	113.37
10000	106.44	112.60	102.90	102.90	102.90

Table H.3: Elevation of Addicks [feet]: Barker dominant

	12	24	48	72	96
10	93.35	94.51	95.18	95.92	97.04
25	94.83	96.24	98.15	99.03	99.44
50	95.91	97.55	99.87	100.96	101.42
100	97.00	98.91	101.75	102.90	102.93
250	98.50	100.87	104.63	105.71	105.87
500	99.72	102.53	107.22	109.12	109.30
1000	101.03	104.38	110.26	113.07	113.37
10000	106.44	112.60	125.01	131.87	132.61

Table H.4: Elevation of Barker [feet]: Addicks dominant

	12h	24h	48h	72h	96h
10	87.89	88.82	89.98	90.60	91.04
25	88.66	89.78	91.24	92.07	92.70
50	89.19	90.47	92.20	93.24	94.06
100	89.71	91.16	93.22	94.53	95.03
250	90.37	92.08	94.67	95.91	96.87
500	90.86	92.82	95.66	97.61	99.08
1000	91.36	93.58	97.02	99.55	101.59
10000	93.09	96.29	94.98	94.98	94.98

Table H.5: Elevation of Barker [feet]: Barker dominant

	12h	24h	48h	72h	96h
10	87.89	88.82	89.98	90.60	91.04
25	88.66	89.78	91.24	92.07	92.70
50	89.19	90.47	92.20	93.25	94.06
100	89.71	91.16	93.22	94.53	95.00
250	90.37	92.08	94.67	95.91	96.87
500	90.86	92.82	95.66	97.61	99.08
1000	91.36	93.58	97.02	99.55	101.59
10000	93.09	96.29	102.68	107.80	112.50

Table H.6: Total Discharge [cfs]: Addicks dominant

	12h	24h	48h	72h	96h
10	9440.33	5077.24	2652.32	1565.52	1013.61
25	11897.50	6557.91	3575.32	2214.91	1530.07
50	13720.50	7656.36	4260.05	2696.67	1913.21
100	15529.90	8746.70	4939.73	3701.16	3872.15
250	17912.30	10182.30	6573.20	6541.11	6543.30
500	19711.20	11266.30	6510.37	6399.44	6524.49
1000	21508.80	12349.50	7185.60	6318.89	6392.12
10000	27477.30	15946.00	139472.00	137479.00	118827.00

Table H.7: Total Discharge [cfs]: Barker dominant

	12h	24h	48h	72h	96h
10	9440.33	5077.24	2652.32	1565.52	1013.61
25	11897.5	6557.91	3575.32	2214.91	1530.07
50	13720.5	7656.36	4260.05	2696.67	1913.21
100	15529.9	8746.7	4939.73	3701.16	4017.32
250	17912.3	10182.3	5834.64	6541.11	6543.3
500	19711.2	11266.3	6510.37	6399.44	6524.49
1000	21508.8	12349.5	7185.6	6318.89	6392.12
10000	27477.3	15946	9427.51	6332.3	5956.35

Table H.8: Run off Discharge [cfs]

	12	24	48	72	96
10	9440.33	5077.24	2652.32	1565.52	1013.61
25	11897.50	6557.91	3575.32	2214.91	1530.07
50	13720.50	7656.36	4260.05	2696.67	1913.21
100	15529.90	8746.70	4939.73	3174.86	2293.51
250	17912.30	10182.30	5834.64	3804.49	2794.25
500	19711.20	11266.30	6510.37	4279.90	3172.35
1000	21508.80	12349.50	7185.60	4754.97	3550.17
10000	27477.30	15946.00	9427.51	6332.30	4804.61

Table H.9: Elevation of downstream [feet]: Addicks dominant

	12	24	48	72	96
10	7.64	2.70	0.00	0.00	0.00
25	9.39	4.79	0.00	0.00	0.00
50	10.46	6.01	1.20	0.00	0.00
100	11.38	7.05	2.47	0.00	0.36
250	12.43	8.22	4.81	4.77	4.77
500	13.13	8.98	4.73	4.59	4.75
1000	13.77	9.67	5.51	4.49	4.58
10000	15.54	11.57	27.08	26.98	25.95

Table H.10: Elevation of downstream [feet]: Barker dominant

	12	24	48	72	96
10	7.64	2.70	0.00	0.00	0.00
25	9.39	4.79	0.00	0.00	0.00
50	10.46	6.01	1.20	0.00	0.00
100	11.38	7.05	2.47	0.00	0.69
250	12.43	8.22	3.84	4.77	4.77
500	13.13	8.98	4.73	4.59	4.75
1000	13.77	9.67	5.51	4.49	4.58
10000	15.54	11.57	7.63	4.51	4.01

Table H.11: Downstream people affected by the flood: Barker dominant

	12	**24**	**48**	**72**	**96**
10	1,956	0	0	0	0
	5,877	0	0	0	0
	0	0	0	0	0
25	8,388	0	0	0	0
	26,121	0	0	0	0
	0	0	0	0	0
50	8,388	1,956	0	0	0
	26,121	5,877	0	0	0
	0	0	0	36,903	36,903
100	8,388	1,956	0	0	0
	26,121	5,877	0	0	0
	0	0	36,903	36,903	36,903
250	8388	8,388	0	0	0
	26,121	26,121	0	0	0
	0	0	36,903	36,903	36,903
500	17,562	8,388	0	0	0
	54,846	26,121	0	0	0
	0	0	36,903	36,903	74,975
1000	17,562	8,388	1,956	0	0
	54,846	26,121	5,877	0	0
	0	36,903	36,903	74,975	74,975
10000	17,562	8,388	1,956	0	0
	54,846	26,121	54,846	54,846	54,846
	36,903	36,903	36,903	36,903	36,903