Alternative designs for hurricane protection systems and building structures in New Orleans
Interim Report

Project *New Orleans '06*
CT4061 Master Project | Baton Rouge, May 2006
Egon Bijlsma | Anke Rolvink | Diederik Veenendaal | Robert-Jan van de Waal

TUDelft
Delft University of Technology

LSU Hurricane Center

LSU
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hurricane protection systems and
building structures in New Orleans
Interim Report

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Infrastructuur, milieu, gebouwen

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Preface

This report is created by four graduate students in Civil Engineering at the Delft University of Technology. It is part of the MSc-project course, CT4061. The goal of this course is research and a design in one of the sub sectors of civil engineering. In a multidisciplinary team various parts of the design should be specified and integrated. By means of supplied and found information an inventory and analysis of the problem is made. This results in a formulation of the problem, objective and finally a design.

The project team chose the city of New Orleans as a target area. In August 2005 this city was struck by hurricane Katrina and has ever since had a lot of media attention and offers enormous challenges in the discipline of civil engineering. The comparison between the state of Louisiana and the Netherlands is striking, which makes these challenges particularly appealing to Dutch civil engineering students. In co'operation with the Louisiana State University and the LSU Hurricane Center the base of operations for this project is at the LSU in Baton Rouge, about 80 miles north of New Orleans. The entire design process will take eight weeks.

This report is the second of three documents produced for the project. It contains an analysis of the possible solutions for the primary, secondary and tertiary protection levels, that are defined in this report. Also, a choice is made which alternatives to elaborate on in the final stage.

Baton Rouge, May 2006

Egon Bijlsma
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Introduction 1

In the previous report (plan of approach) the goals were given for the project; in this interim report the first steps to achieving these goals are taken. First the demands and the boundary conditions are set for the complete protection system; after that the protection system will be split in the three subsections (primary: peripheral levees, secondary: canal levees and watermanagement and tertiary: structural integrety housing). For each of these protection systems an analysis will be made; this analysis will be used to come up with possible alternatives for solving the problems. These alternatives are elaborated to some extent to be able to make a good comparison and afterwards a choice of which alternative is favorable. The chosen alternative will be worked on in more detail in the last stage of the project; the results will be presented in the final report.

The set-up of this interim report is as follows: the second chapter contains a revised version of the plan of approach, which gives the background information and the goals of the project. Using these goals the program of requirements is formulated in chapter three. In chapter four a concept of how to deal with the primary and secondary protection is chosen; in the two following chapters (five and six) the primary and secondary alternatives are elaborated. Chapter seven deals with the solutions for the tertiary protection system. All is brought back together again in the final chapter, the conclusion. This conclusion will be the starting point for the next phase.

Plan of Approach

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2.1 Introduction

New Orleans, the largest city of the state Louisiana in the USA, is situated in the deltaic area of the Mississippi river (see Figure 2.1). In its existence the Mississippi river often changed its course through the delta. With the development of the city the course of the river became more and more regulated and eventually fixated in its current position. This stopped the natural deposition of sediments in the delta area, which in combination with the extraction of water in the urban area led to subsidence of the land. With the growing population of the city new areas, formerly swamp areas, were pumped dry to make the land suitable for habitation.

Figure 2.1: New Orleans along the Mississippi River. Image from: www.spiegel.de [28]

Now especially these newly claimed areas are situated several meters below sea level, in some places as much as 6 meters (20 feet), and are protected by a levee system. The pumping stations however, are still positioned on the borders of the historic center. Outflow canals are used to transport the pumped water to Lake Portchartrain. These canals also have levees and run through the newer parts of the city.

Hurricane Katrina In the late summer of 2005 the city of New Orleans was struck by a hurricane under the name of Katrina. After crossing the Florida peninsula hurricane Katrina made its second landfall on August 29 just to the east New Orleans and continued its path in northern direction. The hurricane by then had weakened from a category 5 storm in the Gulf of Mexico to category 4 in the shallow Louisiana coastal plain with wind speeds reaching up to 250 km/h (155 mph). The storm was accompanied by a storm surge and high wind waves. The path of Katrina along the Gulf of Mexico coast is shown in Figure 2.2.
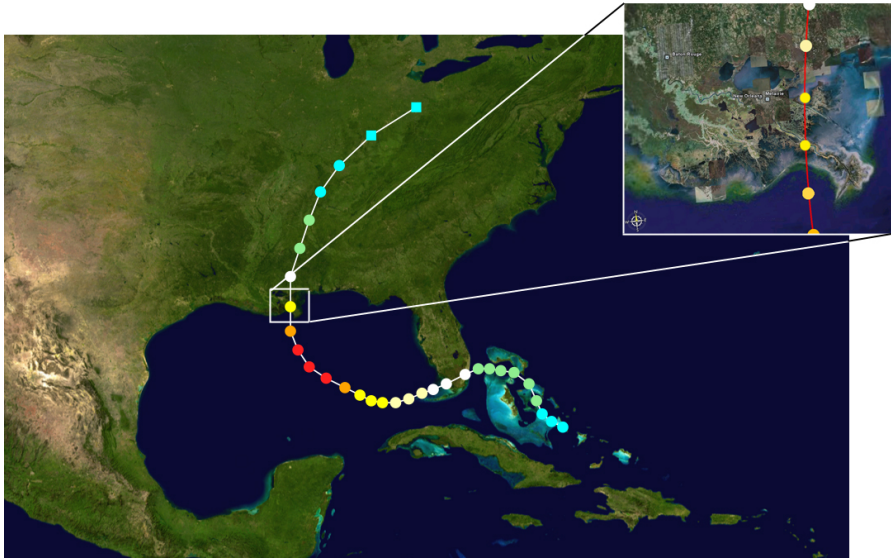
Figure 2.2: Path of hurricane Katrina along the Gulf of Mexico. Image from: http://upload.wikipedia.org [40]

Katrina inflicted massive wind and flood damage to the urban areas in and around New Orleans. High wind speeds caused substantial damage to housing, mainly consisting of wooden timber frame constructions, often leading to instability and collapsing of the structures. High water levels induced by the storm surge and levee breaches caused both flood damage and inundation damage (Figure 2.3). Water that flowed into the basin like areas remained there for a long time, until it was pumped out.

Comparison between Louisiana and the Netherlands Politicians, engineers from the United States Army Corps of Engineers (USACE) and members of the LSU Hurricane Center have contacted institutes and universities in the Netherlands after Hurricane Katrina and have visited some of its water protection systems. Likewise, officials from the Ministry of Transport, Public Works and Water Management (Rijkswaterstaat) and faculty members of the Delft University of Technology (TU Delft) have visited New Orleans and the Louisiana State University (LSU).

The amount of interest is clearly due to the similarities between the Netherlands and the state of Louisiana. Water from the Alps and the Vosges is transported by the rivers Rhine and Meuse respectively to the delta in the south of the Netherlands. The North Sea encloses the
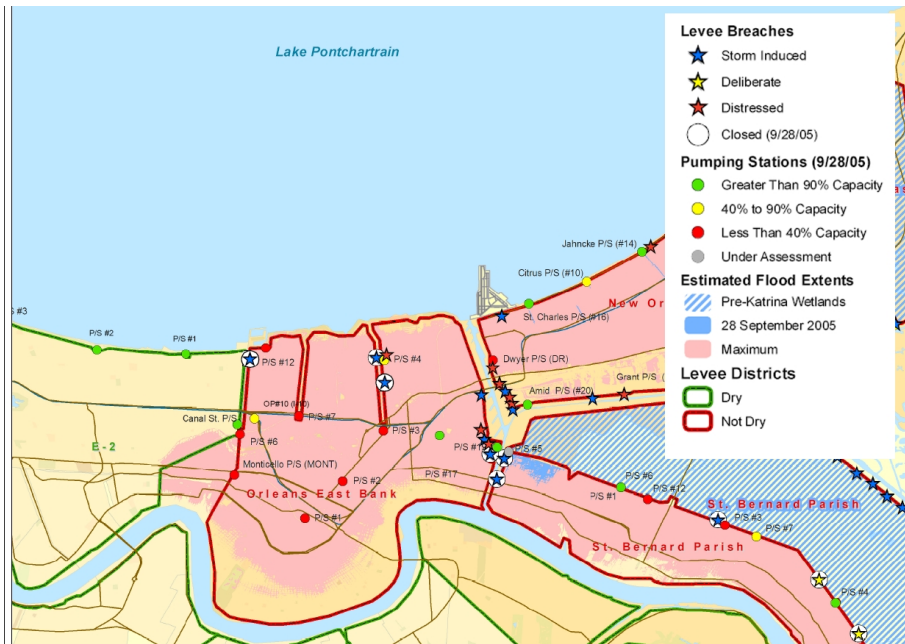
Figure 2.3: Map of levee breaches and inundated areas. Image from: http://soundwaves.usgs.gov [36].

Figure 2.4: Louisiana and the Netherlands to the same scale. Image from: Google Earth [11].

west of the Netherlands. The delta area features some of the most densely populated areas within the country. The port of Rotterdam is the largest port in Europe and forms a highly important economic asset.

Similarly, the Mississippi connects with several large bodies of water and ends at the Gulf of Mexico, about 90 miles below New Orleans. New Orleans consists of large urban areas along the river and boasts one of the countries largest ports (See Figure 2.4).

Both regions lie below sea level and are subject to multiple threats including excessive rainfall, storm wind forces and flooding due to either storm surges or high riverwater levels. To cope with these similar threats, in both regions large areas are protected by levee perimeter systems

or dikes. The history of Louisiana, especially New Orleans and that of the Netherlands is dominated by the struggle to protect its people against the water.

In 1953 a large storm hit the south of the Netherlands, resulting in loss of life and property. The disaster, dubbed the 'Watersnoodramp', paved the way for the 'Delta plan'. The Delta plan consisted of several large hydraulic structures sealing off the water at critical points around the country and was implemented directly, finally to be completed after almost 50 years in 1997. The aftermath of hurricane Katrina mirrors the events after the 'Watersnoodramp'; it has led to many investigations and political turmoil. There are several propositions for large scale flood protection systems, improving on the current situation, though it is yet uncertain what will happen.

2.2   Overview

In this Section the goals of this project are stated. The problems in the struck areas in New Orleans are numerous, and due to the limited time span and manpower of the project team choices have to be made. Therefore a global analysis of the failures during Katrina will first be made, in order to be able to set goals.

The design solutions of this project will have to tackle these failures and prevent them from reoccurring. In addition, an opportunity arises with these designs to contribute to the overall redevelopment of New Orleans. Each design could positively impact the area containing them rather than be seen as an isolated object. This will be an important factor in the design process of this project, and will be elaborated in Chapter 6.

Analysis of the protection system   The occurrence of the disaster and its magnitude was due to many failures, on structural and human domain. The human errors will not be dealt with in this project, the focus lies on the structural errors.

The structural failures can be divided into classes, which can be put into a protection scheme. This is done in Figure 2.5.

Three levels of protection can be distinguished;

- Primary

- Secondary

- Tertiary

With the primary protection the peripheral levees and the outlets of the canals are meant. The secondary protection implies the levees alongside the canals, which cut through the urban areas north of downtown New Orleans. Closely related to this is the system which regulates the removal of excessive water in these urban areas. For example, the excessive water has to be pumped out of the area, into the canals or even directly into Lake Pontchartrain. These two levels of protection are means of prevention. If failure of one of these two protection levels is expected, one has need for the tertiary level of protection, the mitigation of consequences. On this third level of protection one accepts damage to structures; the main concern is to keep people safe. Three options are:

- People stay in their own houses. This means the houses must be able to withstand the loads. Failure of this is indicated in the figure under the subject "Structural integrity of housing".
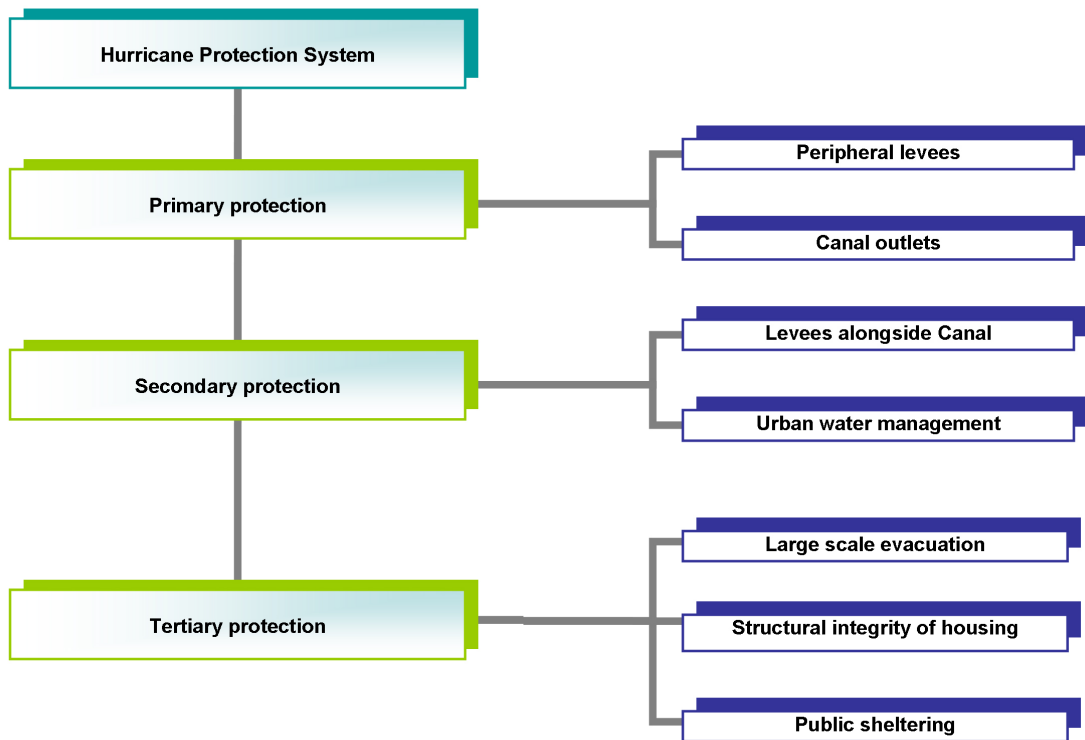
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Figure 2.5: Protection scheme against hurricanes.

- People can go to public buildings serving as shelters if the situation is such that the primary and secondary protections are likely to fail.

- If the primary and secondary protections are likely to fail, the decision can be made to apply large scale evacuation out of the area.

During Katrina several of the above mentioned protections didn't function as they should have done.

- Primary. As there is no way to close the inlets of the canals, during the hurricane the water level in the canals followed that of Lake Pontchartrain. This means these high water levels penetrated into the city to large extend. This can be seen as a failure of the primary protection system.

- Secondary. Because of the above mentioned high water levels, the levees alongside the canals breached and/or were overtopped (See Figure 2.6). So much water inundated the urban areas; the water removal system could not remove all of it. The fact that in the current design the pumps shed the excessive water in the canals made the pumps useless, since all the removed water would flow back through the breaches. In this way the secondary protection failed.

- Tertiary. Because of structural flaws many buildings weren't able to cope with the water and wind. In the direct vicinity of the breaches high flow velocities occurred and literally

swept away houses. No public shelters were available so it was not possible for people to find refuge nearby their houses.
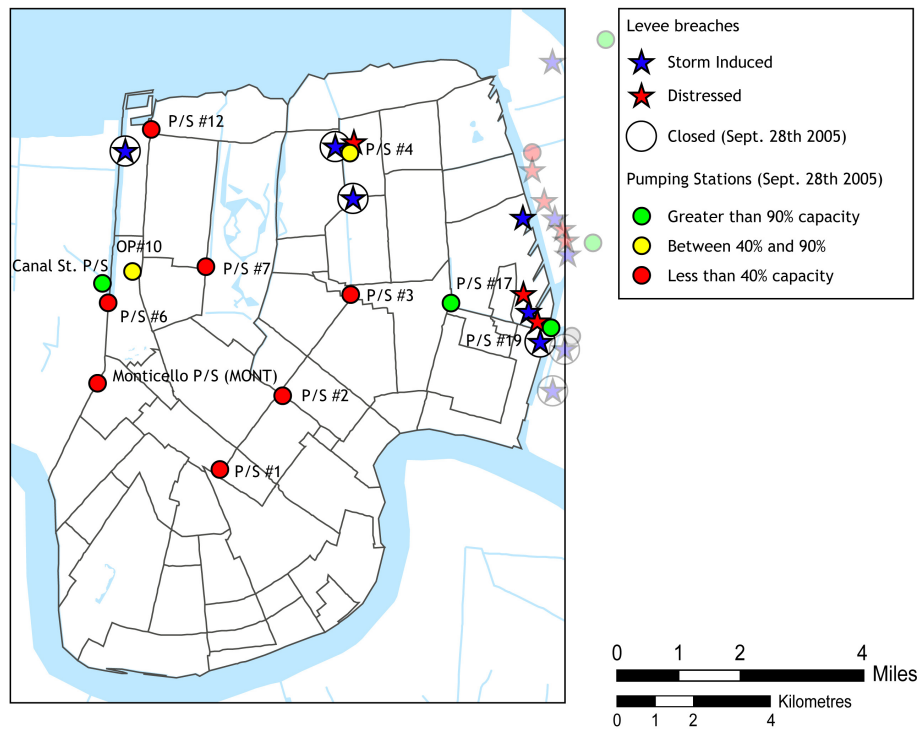

Figure 2.6: Location of failures of the secondary protection system in central New Orleans.

2.3 Determination of scope area

In this chapter a specific scope or focus area is chosen. An area is defined to limit the project in a way such that a detailed design can be made within the seven weeks of this course. The area is chosen based on the disciplines within the team, relevance to the situation in the Netherlands and its relative importance.

Qualitative analysis of possible scope areas Because the state of Louisiana and the Netherlands face the same problems with regards to water protection, the city of New Orleans is the focus of this project.

Within the city of New Orleans several outfall canals can be found in the north of the New Orleans Parish. Originally these canals were constructed to move water from the north side of the city to Lake Pontchartrain. Since then the city has expanded towards the lake and has enveloped these canals. During the hurricane several breaches occurred within these canals; one on each side of the London Avenue Outfall Canal and one on the 17th Street Outfall Canal (or Metairie Outfall Canal). Most of the pumping stations at these canals performed well below

Figure 2.7: Several outfall canals in the north of the New Orleans Parish. Image from: Google Earth [11].

capacity.

This project is a part of the course CT4061, or Master Project, and aims to teach students from the faculty of Civil Engineering problem-solving with a multi-disciplinary approach. For the purposes of this project, the areas of these canals form ideal research subjects. To combine all disciplines of the project team one of these canals will be chosen as a subject. The presence of large residential areas, levees, pumping systems and current plans for flood gates offers possibilities for both hydraulic and building engineering to make a design for a single area.

There are four interior outfall canals fitted with pumping stations that serve to de-water the New Orleans Parish. From west to east these are; the 17th Street Outfall Canal, the Orleans Avenue Outfall Canal, Bayou St. John and the London Avenue Outfall Canal.

For the purposes of this project a quantitative analysis would cost too much time though the GNOCDC website offers lots of data on specific neighborhoods. Nevertheless, a qualitative comparison is made (see Table 2.1). For hydraulic engineering the canals differ only slightly, so the comparison is done from a building engineering point of view. A densely populated urban area of historical, social and economic importance is preferred to adequately apply lessons learned here in the Netherlands.

An important factor for comparison is the population density of the surrounding area. A demographic [2005 Louisiana Hurricane Atlas] map of New Orleans shows that the area along the London Avenue Outfall Canal was the most densely populated. The number of houses per square mile also coincides with the population density. The City Park between the Orleans Avenue Outfall Canal and Bayou St. John obviously had no residents and thus is of less immediate importance for rebuilding.

Figure 2.8: Population per square mile. Image from: www.gnocdc.org [3].

The cultural and historical importance can also be taken into account by comparing the percentage of historic buildings (built before 1950). [GNOCDC] The neighbourhoods west of the Orleans Avenue Outfall Canal and surrounding the London Avenue Outfall Canal consist of up to 60% of such buildings.

Though the findings of this project are equally important for all four canals, the areas where the levees breached offer more opportunities for implementation. As mentioned, levees along the 17th Street Outfall Canal and the London Avenue Outfall Canal were breached. Also, the housing units that were hit near these areas boast the most information on direct damage incurred during flooding. These areas will be investigated in any case. A final criterion is the number of institutions within each area [11]. By far the largest number of schools and no less than three universities can be found east of the London Avenue Canal. All other areas are purely residential in nature, with the exception of the City Park and a small marina north at the 17th Street Canal.

	17th Street Canal	Orleans Avenue Canal	Bayou St. John	London Avenua Outfall Canal
Population /housing density	0	-	-	+
Historical /cultural importance	0	+	0	+
Implementation opportunities	+	0	0	+
Institutions	0	0	0	+

Table 2.1: quantative analysis of specific neighborhoods

The London Avenue Outfall Canal and the adjacent residential areas are selected as the scope
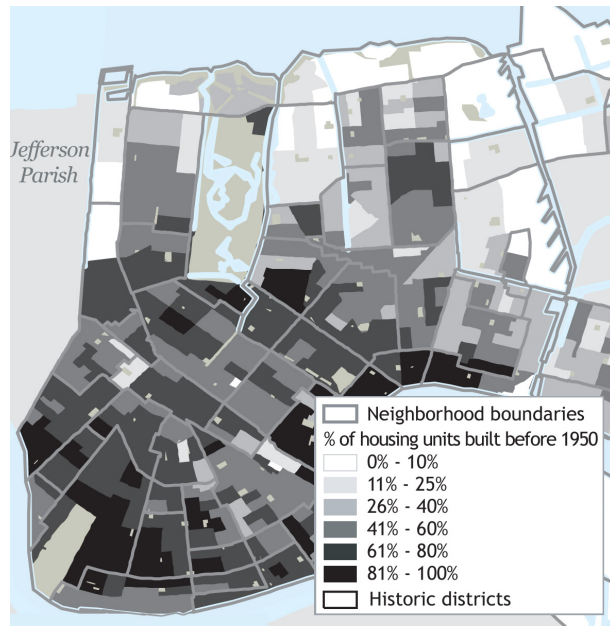
Figure 2.9: Percentage of housing units built before 1950. Image from: www.gnocdc.org [3].

area of this project. The area is also of some importance for providing a connection between Downtown and the Lakefront area and airport.
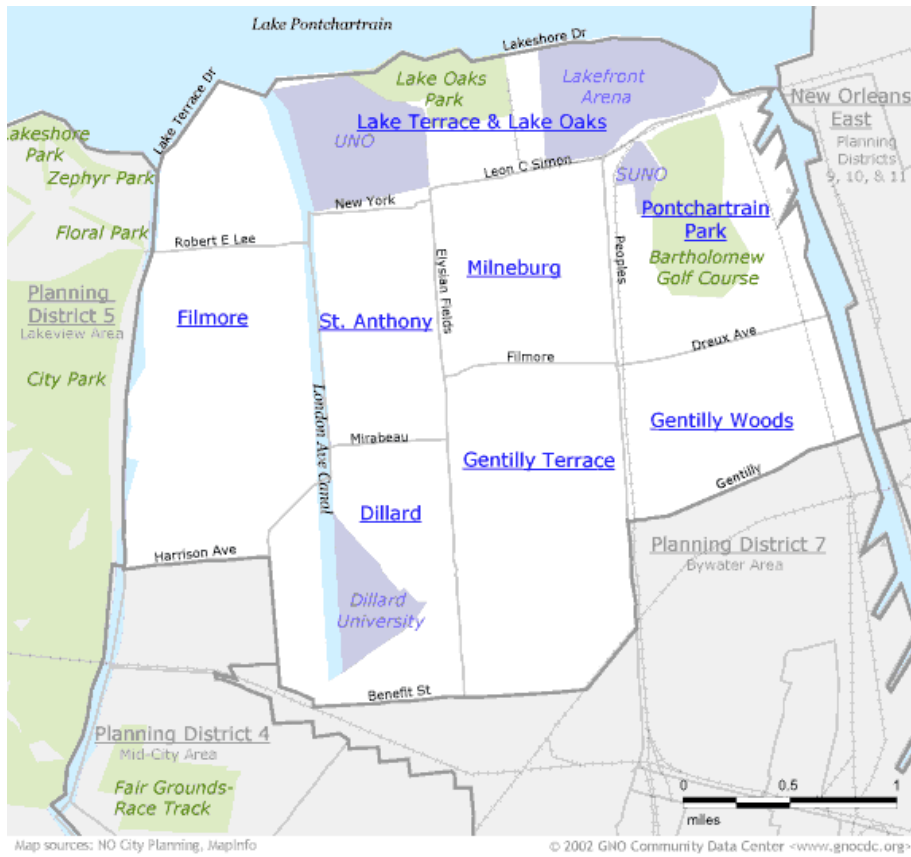
Figure 2.10: The scope area surrounding the London Avenue Outfall Canal. Image from: GNOCDC.org [3]

2.4 Project goals

The overlying thought behind this project is the (re)design of one of the Katrina struck areas of New Orleans, in such a way that it is able to better cope with future similar events and will increase potential for redevelopment. The main objective of the project is defined as:

To design alternative solutions for the water protection system and building structures in the Gentilly district, considering their potential impact on prospects for redevelopment.

The main objective consists of two issues; safety and redevelopment. Both issues are strongly related in their effects on the prospects of Gentilly. Providing a higher degree of safety will not necessarily mean that Gentilly will return to its former state. Likewise, stimulating development efforts is moot without better guarantees that an event such as Katrina won't reoccur.

This project will deal with both safety and redevelopment, by defining specific design objects that protect the area and by later introducing factors that affect redevelopment as important criteria for choosing the optimal designs. In this way, safety is the primary focus but the designs will not be indifferent to the surroundings. First an inventory is made of the protection gainst threats induced by hurricanes. This will be done on the three protection levels.

- Primary. The main object of interest here is to investigate the possibility of a way to close off the canals from Lake Pontchartrain. This may imply a fixed barrier or a movable one.

- Secondary. The levees showed to be insufficiently strong. A new design can be made for them, holding into account all the failure mechanisms of a levee. Also improvements can be made for the excessive water removal system, e.g. relocating pumping stations and improving the sewer system.

- Tertiary. The structural design of the timber framed housing can be improved as to make it more resilient against flooding and inundation. Investigation whether to implement public shelters should be conducted, together with the design of these shelters. The large scale evacuation out of the area proved to be adequate and needs not to be investigated.

2.5 Overview of Gentilly District

The scope area surrounding the London Avenue Outfall Canal lies within Planning District 6, or Gentilly area, and is therefore defined as such. Four neighborhoods border the canal; Fillmore to the west and Lake Terrace & Lake Oaks, St. Anthony and Dillard to the east. See Figure 2.10. There are several assets in the area. There is a racetrack, a ball park and Dillard University on the southend of the London Avenue Outfall Canal. At lakeside, to the north, lies the University of New Orleans as well as a large golf course. The Gentilly district also links several areas within New Orleans; the French Quarter (and Business District), the City Park, Lakefront and Lake Pontchartrain. This chapter states specific objectives both on the topic of the levee system and building structures. First, some observations are made on how these specific design objects are related and how they could effect redevelopment.

In most urban, deltaic areas the presence of water offers quality spaces, which in turn attracts investments in residential housing and commercial projects. Though Gentilly itself borders to Lake Pontchartrain and has a connection canal running through it, it offers no connection to these waters. At Lake Pontchartrain one can find an artificial beach and some sports facilities, but for the most part a road with a few parking lots run along the shoreline.

The concrete T-walls along the canal obstruct views and prohibit access.

This project concerns both housing and water protection systems. This provides an opportunity to connect water, levee and city in ways that the quality of space improves. Property along or near the water could gain from an esthetic point of view, potentially attracting residents and businesses to the benefit of Gentilly and New Orleans. The new design solutions could be sensitive to this concept and would contribute to overall redevelopment by:

- increasing connectivity between isolated elements

- increasing connectivity between parts of the city

- improving quality of space and quality of living
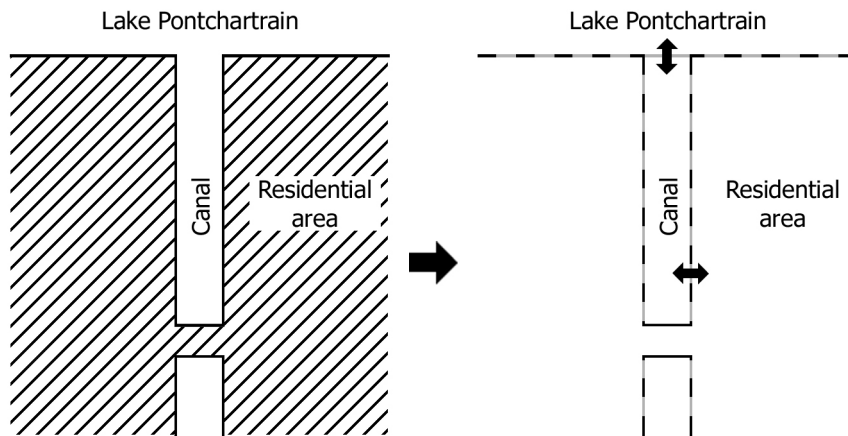
- increasing economic and commercial potential

Figure 2.11: Improved communication between isolated elements after redesign.

When viewing the urban planning within Gentilly, it can be said that several infrastructural arteries and the internal T-wall levees divide and separate the areas on both sides. The major roads running through the district cut up the residential area into smaller compartments, while the concrete levees keep them all walled in together. The safety of this area needs to be increased, but this doesn't imply that access between water and land will be more restricted. In fact, the opposite might be possible. As access is improved, water and land will communicate more which is beneficial for the area as a whole. Realizing that the primary, secondary and tertiary protections, as well as the inhabitants of the area they protect are related, a total design could not only achieve a cumulative effect but become more than just the sum of its parts.

As Gentilly becomes increasingly attractive to development, it will no longer function as a traffic hub for the surrounding areas. Instead the district could form a strong link in the chain as New Orleans' recovery will start and spread from the French Quarter and Business District outwards.
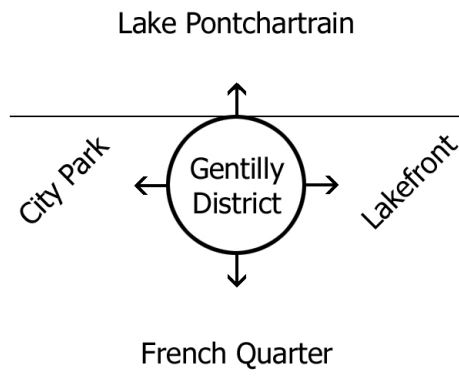
Lake Pontchartrain

City Park
Gentilly
District
Lakefront

French Quarter

Figure 2.12: Orientation of Gentilly District within New Orleans.

2.5.1 Levee protection system

The greater part of the Gentilly district lies below sea level and is protected by a levee system along the perimeter. This protection consists of a primary and secondary system.

2.5.1.1 Primary levee protection

The London Avenue Outfall Canal is situated in the northeastern section of the New Orleans East Bank quarter, perpendicular to Lake Pontchartrain (Figure 2.13). Here the primary protection system embodies two parts,

- The peripheral levees adjacent to Lake Pontchartrain

- The London Avenue Outfall Canal outlet

Figure 2.13: London Avenue Outfall Canal outlet in the New Orleans East Bank quarter. Image from: Google Earth [11]

The peripheral levees adjacent to the lake proved to be sufficient to cope with the storm surge water levels. No breaches occurred.

The outlet directly connects the London Avenue Outfall Canal with the lake. This is an open connection, which means that the water surplus of the canal is shed into the lake, but also the water level in the canal follows that of the lake.

During hurricane Katrina this open connection proved to be one of the weaknesses of the city's primary protection. The storm surge water levels could enter the city directly through the canal, thus penetrating into the urban areas for several kilometers. This way the canals levees were loaded with extremely high water levels and finally breached.

Objectives   The following improvements of the primary protection of the London Avenue Outfall Canal will be investigated:

- A flood barrier in the London Avenue Outfall Canal outlet

- Protection works around this barrier

The purpose of the flood barrier is to physically separate the water bodies in Lake Pontchartrain and the London Avenue Outfall Canal. This barrier can either be a fixed or a movable structure. The protection works should prevent the barrier to be damaged by for example ships.

2.5.1.2   Secondary levee protection

The secondary protection of the New Orleans East Bank quarter consists of the levees along the London Avenue Outfall Canal and the pumping facilities at most southern point of it.

With the high water levels during hurricane Katrina the canal was breached in the northern part and also in the southern part. Investigations shortly after the disaster could not indicate the true reasons of failure, but signs of massive earth displacement behind the levees were seen [12]. Presently the levees are being repaired and brought back to their original pre-Katrina conditions. This means that if another similar event would occur, the levees are likely to fail again, unless improvements are made.

Objectives   The objective is to analyze and upgrade the secondary protection. The following improvements will be investigated:

- Strengthening of the levee system in such a way, that the design water levels can be withstood. In the process of determining the design the long term effects will be taken into account, i.e. the life span is aimed to be several decades, structural as well as esthetical.

- Revision of the water management system, in order to enable it to cope with excessive amounts of water if for example overtopping occurs, or heavy rainfall.

2.5.2   Building performance

More than 14.000 Gentilly homes suffered some form of damage, some 81% of all homes in the area. Of these, 11.355 were either severely damaged or completely destroyed mainly due to flooding and inundation. There were 230 houses damaged to some degree by wind only [4].
Flood-prone areas are divided in different zones of varying risk to flooding by the National Flood Insurance Program (NFIP). The Gentilly area is an A zone. Flood forces in A zones are not as severe as in coastal zones, but are still capable of damaging or destroying buildings on shallow foundations.

2.5.2.1   Flood damage

The areas surrounding the levee breaches were directly hit by hydrodynamic forces and flood borne debris and suffered the most severe damage seen.

- Failure of load-bearing walls

- Poor soil conditions leading to structural damage, cracking of load-bearing walls and sagging floors

- Slab-on-grade foundations or foundation piers with insufficient reinforcement

Though the devastation around the London Avenue Outfall Canal was not as vast as in the Lower Ninth Ward, several houses did collapse and other modes of failure were observed as well, all due to high hydrodynamic loading. A number of buildings near the southern breach were stripped off their outer leaf. In general housing with a brick outer leaf seems to have held up better than those with strictly wooden facades.

2.5.2.2 Inundation damage

After the hurricane the buildings around the London Avenue Outfall Canal generally had to cope with long-term flooding, rather than wind or hydrodynamic induced damage. LIDAR data from the LSU suggest that flood depths exceeded as much as 12 ft, but generally ranged from less than 1 to 8 ft. The effects of this continued exposure to water resulted in mold growth, loss of property and irreversible damage to building materials and components. Some home-owners have returned to empty their houses and allow the wood-framing and other parts to dry by natural ventilation. In other areas of New Orleans some houses floated off their foundation due to hydrostatic forces and poor connections.
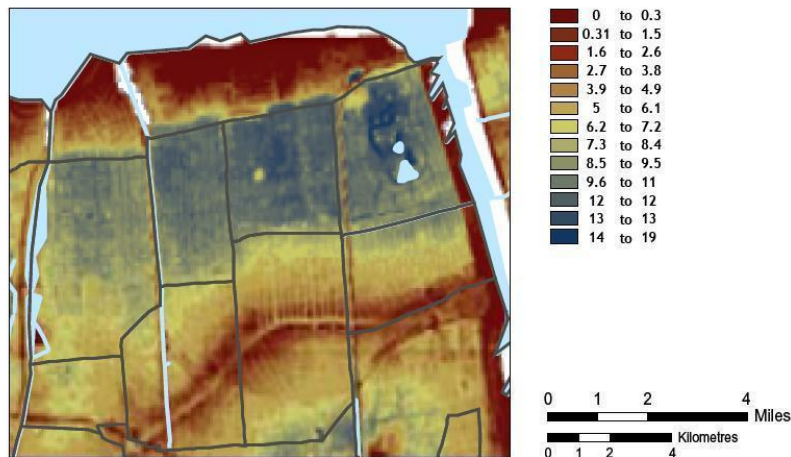

Figure 2.14: Flood depths in Gentilly District (ft). Image from: 2005 Louisiana Hurricane Impact Atlas [5]

2.5.2.3 Wind damage

Throughout Louisiana several types of wind-induced damage were observed. Several types of structural damage were common and sometimes led to successive failure of other building component or total collapse:

- Insufficient attachment of roof sheathing panels

- Gable end wall failures

- Collapse of unreinforced load-bearing masonry walls

Windborne debris and in some cases poor construction led to damage to the building envelope to varying degrees. Loss of roof coverings, failure of wall coverings and broken glazing has been widely reported.

Damage that occurred as a result of hurricane winds in the scope area was most visible at rooftops, where roof-decking was blown off. The loss of roof shingles, especially hip/ridge trim shingles was common. Other cases of damage to the building envelope were seen as well. At this point it is hard to determine if wind was cause of structural failure for some homes and in this particular area clearly most of the damage is due to flooding.

2.5.2.4 Raising rules based on flood elevation

Recently the FEMA released new recommendations for the Base Flood Elevation (BFE), which were last adjusted in 1984 and determine the design height of structures in flood-prone areas. BFE's were based on the assumption that levee-protected areas perform adequately and only account for accumulation of rainfall within the area. The BFE's are currently documented in the pre-Katrina Flood Insurance Rate Maps (FIRM's). The temporary Katrina Flood Recovery Maps show newer Advisory BFE's or ABFE's [16].

Because the BFE's weren't mandatory, most houses in the New Orleans Parish were built on slab-on-grade foundations. For substantially damaged or newly built housing it is now recommended that the top of the lowest floor is elevated to at least 3 feet above the ground or BFE requirement, whichever is higher. 'Substantial damage' is defined as repairs costing more than 50 percent of the cost to completely rebuild the house. In the Gentilly area the ground level is at 6 ft below sea level, while the BFE is determined at -2,5 ft. A badly damaged or new home will have to be raised to 3,5 ft above grade. Because insurance companies determine flood insurance premiums based on compliance with these recommendations, slab-on-grade foundations will effectively become obsolete.

Objectives   Before Katrina most structures in the Gentilly district were timber frame houses. A significant number was built on grade. Clearly the disaster showed that previous designs were lacking with respect to possible flooding. With current recommendations it is expected that all future structures will be elevated as a solution. Elevating homes has been a historically proven method of coping with floods. However, alternative solutions exist as well and aren't being considered as real options, though the disaster has left more room for innovation as opposed to using time-proven methods. This project will focus on these other possibilities. The objectives are:

- Designing innovative, alternative solutions to current recommendations for housing structures.

- Determining the structural and economical feasibility of these designs compared to current solutions.

Program of requirements

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In this chapter the demands, the boundary conditions and the assumptions are stated. In Appendix B and C the topics are further elaborated.

3.1 Demands

- The Gentilly District levee-protection system must be adequately designed against a hurricane for a period of 100 years (hurricane category 5).

- The Gentilly District building structures system must be adequately designed against a hurricane for a period of 50 years (hurricane category 3).

- The designs must be economically feasible and competitive.

- The drainage capacity of the water management system should be at least equal to the current capacity of 221.4 m^3/s.

3.2 Boundary conditions

- The designs are made with respect to all applicable minimum design loads and building codes.

- The surge induced by a hurricane of category 5 is at least 5.49 m above normal, the surge of a category 3 hurricane is maximally 3.66 m above normal.

- The wind speeds that are induced by a category 5 and category 3 hurricane are > 249 km/hr (69.2 m/s) and ≤ 209 km/hr (58 m/s or 130 mph)

- The Highest High Water (HHW) of the tide in Lake Pontchartrain is Mean Sea Level (MSL) + 0.40 m

- During a category 5 hurricane the wave's $H_s = 4,46$ m and $T_p = 7,32$ s

- The design floodwater level within the levee-protected area is the Base Flood Elevation (BFE) or at least 0,9m (3 ft) above grade, whichever is higher.

- The ground layer that can be considered load bearing is at MSL - 25 m, it consists of sand.

- In the situation the levee heights are:

    - MSL + 5.60 m for the peripheral levees
    - MSL + 4.38 m and MSL + 4.51 m for the levees alongside the canal

- In the present situation the pump stations are located at the southern point of the London Avenue Outfall Canal and 3.14 km north of that on the east side of the canal.

3.3   Assumptions

- Maximum precipitation of 500 mm in a period of 12 hours.

- The sea level rise is 0.66 m/century.

- The ground elevation of the Gentilly District ranges from -4m to +1m compared to Mean Sea Level.

- The design elevation of new buildings is 2 m above grade.

- Inundation within the levee-protected area occurs due to only overtopping and excessive rainfall.

- The demographic properties of the Gentilly District remain unaltered.

- The general subsidence in the Gentilly District is 0.9 m/century (3 ft).

- The subsidence of newly placed clay structures is 10% of their height.

- The soil properties:

	Sand	Clay and peat	Silt
Density ρ $[kg/m3]$	2700	1300	3000
Cohesion c $[N/m2]$	0	25-50	2-5
Angle of internal friction ϕ [-]	30-45	15-30	27.5-32.5
Permeability k $[m/s]$	10^{-6} - 10^{-3}	10^{-10} - 10^{-8}	10^{-8} - 10^{-6}
Coefficient of compression C [-]	50-500	2-100	25-125
Coefficient of compression C_{10} [-]	20-200	1-40	10-50

Alternative concepts for primary and secondary protection

4

Two different concepts are mentioned in this chapter. They will be elaborated to a certain extent, in order to be able to make a good comparison between them. The focus on this chapter is to determine which part of the first defense line will be enhanced; the primary or the secondary protection. As stated before it is not necessary to improve both to the state in which they can cope with a category 5 hurricane.

4.1 Concept I Enhancement of the primary protection

The purpose of the primary protection is to keep the surge out of the urban areas. This implies that the canals must be separated from Lake Pontchartrain, with either a fixed or a movable barrier. The peripheral levees must be able to withstand the loads induced by a hurricane.

4.2 Concept II Enhancement of the secondary protection

The secondary protection serves to withstand the loads coming from the canal. If the secondary protection is enhanced to the state that it can withstand a category 5 hurricane, it is implied that the primary protection is not able to cope with such a hurricane. The levees alongside the canal must be high enough to prevent too much overtopping. The levels in the canal will follow that of Lake Pontchartrain.

4.3 Choice of concept

Concept I is chosen for further study. In the decision making the following criteria have played a role:

- Building costs. Present structures need to be updated and new ones must be constructed. For Concept I a whole new structure must be made (the floodgate); for concept II many kilometers of levees need to be raised and strengthened. Concept II is probably the least expensive; it is more an adjustment to the present situation, as Concept I means a new structure and possible revision of the water management.

- Length of protection. The longer the line of defense, the greater the chance of a weak spot. Concept I shortens the line of the defense with many kilometers. Also, Concept I will be a smaller structure than Concept II and thus need less maintenance.

- Public sense of safety. In general people won't feel safe living close to a large levee that must hold back large quantities of water. With Concept I the high water tables are kept out of the urban areas, concepts II allows them. People will feel safer with concept I, especially after the breaches that occurred with Katrina.

Considering the last two criteria Concepts I clearly predominates, but Concept II is probably cheaper. Still, also keeping the disaster of hurricane Katrina in mind, the (sense of) safety of

the people is considered more important. That's the reason the enhancement of the primary protection is chosen over that of the secondary protection.

Primary protection

5

The concept of the design has been determined in Chapter 4. In this chapter a number of alternatives will be examined. In order to keep everything understandable the concept will be divided in three parts; the peripheral levees, the floodgate and the urban water management system. The first part (peripheral levees) is mostly independent from the other two, save for the connection to the floodgate; the floodgate and the urban water management system are linked tightly. This should be kept in mind in the determination and evaluation of the designs.

5.1 Peripheral Levees

In the present situation the peripheral levees are at a height of MSL + 5.60 m (see Appendix B.5). The levees are earthen; on top of it a road is placed. In 100 years, this height will be insufficient to withstand the surges induced by a category 5 hurricane. However, if a category 5 hurricane makes landfall it will never reach Lake Pontchartrain that strong. It will at the most have the strength of a severe hurricane 3. The future surge height that must be withstood is dependant on the following factors.

- Sea level rise. This is assumed to be 0.66 m in a century.

- Tidal elevation. The HHW that will be used is 0.4 m.

- Surge induced by the hurricane. According to the Saffir -Simpson Hurricane Scale a category 3 hurricane causes a surge of 3.66 m. This is the maximal height. Still, to be on the safe side, this value is used.

So during a hurricane the MSL will rise to an elevation of 4.72 m (15.5 ft) above the present MSL. What should be kept in mind is that it isn't enough to design a levee to this height. One should not forget the extra height that is needed to cope with the wave height and the wave run-up. The size of the wave run-up depends on the structure in three ways; the slope, the roughness of the surface and the presence of a berm.

In the present situation there is a berm at some locations (the west side of Gentilly District), which is approximately 50 m wide and is at a level of about MSL + 2m. More to the east there is no berm present. The revetment on the berm and the parts above this berm is of grass; underneath is a concrete revetment. The problem with a grass revetment is that its resistance against wave attack is limited. See also figure 5.1. In the figure one can also see some trees growing on top of the berm, this is not preferable for the stability of the levee. Trees will die and when they do, the roots will deteriorate leaving weak spots in the levee.

Below three different alternatives will be elaborated, each with a different principal behind it.

Figure 5.1: The present peripheral levee (picture taken on top of the berm).

5.1.1 Alternative I Earthen levee

An earthen levee structure has got a number of failure modes. These are shown in Figure 5.2. The levee has to be designed in such a way that all these modes will not appear. Of great importance with levees is the height; this should be sufficient to prevent too much overtopping. This height will be determined first and will serve as the starting point for the rest of the design.
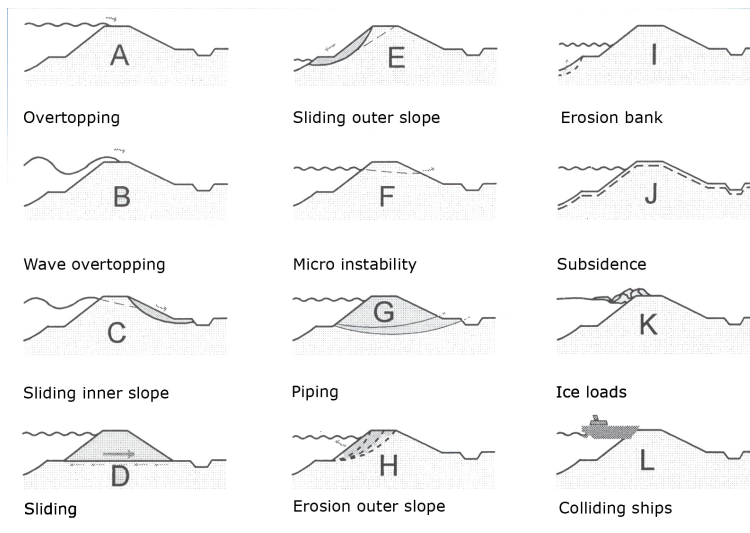

Figure 5.2: Failure modes earthen structures. Image from: TAW [30].

5.1.1.1 Levee Height

In the determination of the design height three levels are of importance:

- Design level. This is the height that the levee must have at least. This level is determined by the tidal elevation, wind set up and the hurricane surge. Wave action is not taken into account yet. So the design level will be MSL + 4.06 m.

- Wave overtopping resistant level. The wave overtopping resistant level consists of the design level and the retaining height. The retaining height is the extra height because of wave run-up and sea level rise, and has a minimal value of 0.50 m.

- Construction level. With this level the height of the levee directly after the construction is meant. The construction level is the wave overtopping resistant level and the freeboard. The freeboard consists of the subsidence of the subsoil and of the structure itself.

Retaining height The first part of the retaining height is determined with the sea level rise, which is 0.66 m/century. The other part, the wave run-up, is dependant of three factors:

- The presence of a berm. If a berm is placed at the design level it has a reducing effect on the wave height and thus on the run-up. Another effect of the smaller waves is that the wave attack on the revetment will be less severe if a berm is present. Both effects have a positive effect on the structure and therefore a berm will be placed in front of the levee.

- The slope of the levee. In conditions such as those that appear in The Netherlands it is possible to optimize the slope in order to get the smallest needed volume of sand. In Louisiana the conditions are a lot rougher. Here it is more important to take a good look at the revetment to make sure that the levee remains undamaged. Most of the time a certain type of revetment implies a maximum slope.

- The type of the revetment. The revetment's primary function is to make sure that the incoming waves will not erode the levee. This means that the revetment itself must be strong enough to withstand the wave attacks. Not all the types of revetments are able to withstand the conditions that will appear during a category 5 hurricane (e.g. grass would not survive long). A choice will be made out three types of revetment: asphalt, rock or concrete.

5.1.1.2 Asphalt revetment

The big advantage of a revetment made of asphalt is that it's fairly cheap. Downside of the method is that asphalt is rather smooth, and that will result in a relatively high run-up. Maintenance is of great importance too. The layer of asphalt needs to be intact; if there is a crack or a hole the sand body underneath will be eroded away during a storm. If the underlying sand is gone, uneven settlements will appear and the cracks will grow bigger. This causes more erosion and a vicious circle has started.

The soil in New Orleans consists of a lot of peat and clay, so uneven settlements are not unlikely. Therefore the material asphalt will not be used to construct the revetment.

5.1.1.3 Revetment made of rock

The armour layer (outside layer) should be made of rocks that are heavy enough so that they won't be washed away during a storm. Hudson (1953, 1959 and 1961) proposed the following equation:

$$W \geq \frac{\rho_r g H^3}{\Delta^3 K_D cot_\alpha} \tag{5.1}$$

Where:

W	=	mass of the stone
ρ_r	=	density of the stone
H	=	wave height
Δ	=	relative density of the stone
α	=	slope (the formula is applicable to slopes between 1:1 and 1:4)
K_D	=	coefficient for many influences

Later Anon (1984) improved the formula by replacing H with H_{10}, which is the average of the 10% highest waves. This H_{10} is equal to $1.27 * H_s = 5.66$ m.

Type of block	number of layers (N)	structure trunk K_D		structure head K_D	
		breaking wave	non breaking wave	breaking wave	non breaking wave
rough angular squarry stone	1	**	2,9	**	2,2
rough angular squarry stone	2	2,0	4,0	1,6	2,8
rough angular squarry stone	3	2,2	4,5	2,1	4,2
tetrapod	2	7,0	8,0	4,5	5,5
dolos	2	15,8	31,8	8,0	16,0
cube	2	6,5	7,5		5,0
akmon	2	8	9	n.a.	n.a.
Accropod ®(1:1,33)		12	15		

Table 5.1: Values recommended given in SPM 1984 [8].

If two layers are assumed, breaking waves and an angle of 1:3, then the mass of the rock should be minimally:

$$W \geq \frac{2600 * 9,81 * 5,66^3}{1,6 * 2 * 3} = 482 * 10^3 kg$$

The relation between the mass and the nominal diameter D_n is $W = \rho_r g D_n^3$ So the nominal diameter of the rock will lie in the order of 2.66 m. This is a very conservative estimate, mostly because of the use of H_{10}, but it gives an indication of what is needed. Rocks this side must be made in a quarry, and not every quarry is capable of producing this size of rock. New Orleans lies in the delta of a large river, the Mississippi River, and there are little quarries to be found in the surroundings. The nearest quarries that can possibly produce these stones can be found in Arkansas, but that is far away, about 1000 km. This makes it unattractive to use stones for the armour layer.

5.1.1.4 Revetment made with concrete armour units

There is a wide arrange of types of concrete blocks. In Figure 5.3 an overview is given of the possible concrete rocks.
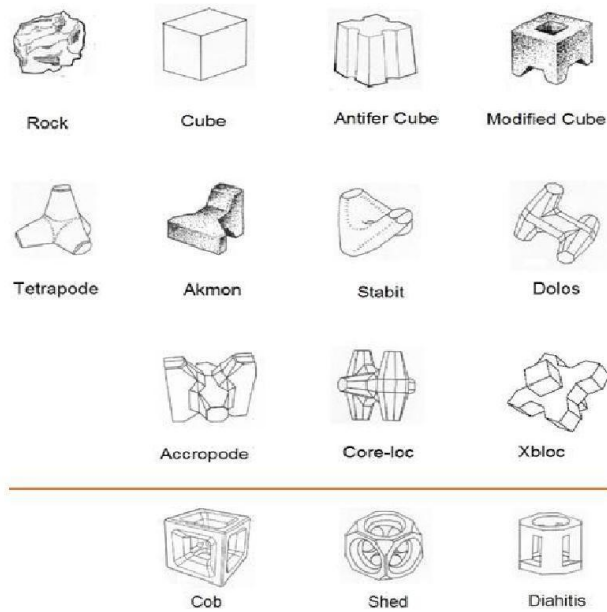
Figure 5.3: Overview of concrete armour units.

Although there are many different types of concrete armour units, the slope they need is relatively equal. For the Cube, Tetrapod and Dolos type a slope of 1:1.5 is required, for the Accropod® type a slope of 1:1.33 is sufficient [8]. For the determination of the wave run-up a slope of 1:1.5 is used. In a later stage the desirable type of armour unit and its dimensions will be determined.

Wave run-up   The wave run up can be determined using the theory of Ibarren. For this the breaker parameter ξ_m must be introduced.

$$\xi_m = \frac{tan\alpha}{\sqrt{H/L}} = \frac{1/1,5}{\sqrt{4,46/83,7}} = 2,89 \tag{5.2}$$

Where

ξ_m	=	breaker parameter for deep water, mean period
H = wave height	=	4.46m
L = wave length	=	$\frac{gT^2}{2\pi}$ =83.7 m

Van der Meer and Stam (1992) proposed for $\xi_m > 1.5$ the relation (see also Table 5.2)

$$\frac{z_{u\%}}{H_s} = b\xi_m^c \tag{5.3}$$

Where

$z_{u\%}$	=	run-up level exceeded by u% of the incoming waves
b,c	=	parameters according to Table 6

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As there will be a berm included with the levee, at a height of MSL + 2 m, the wave heights that reach the levee will be diminished with a factor γ_b. Figure 5.4 shows the relationship between the reduction factor γ_b and the wave attack.
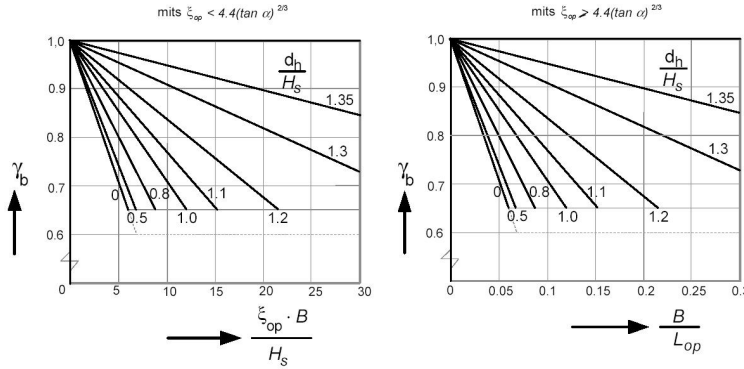
Figure 5.4: Factors of reduction induced by a berm. Image from: Inleiding waterbouwkunde [9].

$$\xi_{0p} = 2,89 \geq 4,4(tan\alpha)^{2/3} = 0,65 \tag{5.4}$$

$$\frac{d_h}{H_s} = \frac{4,72-2,0}{4,46} = 0,61 \tag{5.5}$$

$$\frac{B}{L_{0p}} = \frac{50}{83,7} = 0,6 \tag{5.6}$$

From these values it turns out that $\gamma_b = 0.65$, resulting in $H_s = 2.90$m.
This relation only applies to structures with an impermeable or almost impermeable core. This will be the case for the levees adjacent to Lake Pontchartrain, because the existing levees are made of impermeable soil. The adjustment will most probably mean that the present levees will be raised, not that they will be demolished and after that rebuilt. Since a category 3 hurricane will probably not occur more than one or two times in the coming century in Lake Pontchartrain, some overtopping is allowed (though the levee must not be eroded because of it). So for u a value of 10 is taken.

This results in a $z_{10\%} = 3.50$ m.
The total retaining height will be 4.16 m, resulting in a wave overtopping resistant level of MSL + 8.90 m (29.1 ft).

run-up level u(%)	a	b	c	d
0,1	1,12	1,34	0,55	2,58
1	1,01	1,24	0,48	2,15
2	0,96	1,17	0,46	1,97
5	0,86	1,05	0,44	1,68
10	0,77	0,94	0,42	1,45
Sign.	0,72	0,88	0,41	1,35
Mean	0,47	0,6	0,34	0,82

Table 5.2: Run-up parameters for rubble covered and permeable slopes [8]

Construction level As stated above the construction level consists of the wave overtopping resistant level added with the subsidence of the subsoil and that of the newly placed soil. In 100 years the subsidence of the subsoil is assumed to be 0.9m. The subsidence of the newly placed soil is estimated 10% of its original height. The old levee will be raised; so that means the additional needed height is 3.30m. This implies that the freeboard needs to be 0.9m + 10% * 3.30 m = 1.23m to make sure that the levee will have its required height in 100 years to come. The construction level will need to be MSL + 10.1 m (33 ft).

5.1.1.5 Levee crest and inner slope

In this paragraph the minimal crest width and inner slope will be determined. For the inner slope the stability is the most important factor; for the crest width functions of the levee other than keeping water out of urban areas have to be considered too.

Levee crest In the determining of the levee height some overtopping during a hurricane is taken into account. The crest must be able to withstand this overtopping. Another function of the levee in the present state is transport; a road is built on top of the levee. In the new design this will also be the case, so the levee crest should be wide enough for that road. A width of 8m (26 ft) will be used; two driving lanes 3m wide with on each side a shoulder of 1m.

Inner slope Two failure modes apply to the inner slope; the erosion of the inner slope due to overtopping and/or sliding of it.

To protect the inner slope against erosion a good revetment has to be placed. As the overtopping during a category 3 hurricane is considerable the revetment has to be a strong one. Revetments like the ones made of grass are not strong enough. A good example of a possible revetment is the placed block revetment; if placed meticulously the blocks are well able to protect the levee from eroding. Another advantage this solution it has over e.g. a loose rocks revetment is that it has some esthetical value. This can be considered to be important, as there are people living close to this levee and have to look at it each day. A slope that will go well with this type of revetment is in the order of 1:3. In most cases this slope will be sufficient to prevent the inner slope from sliding. In further studies this should be examined extensively.
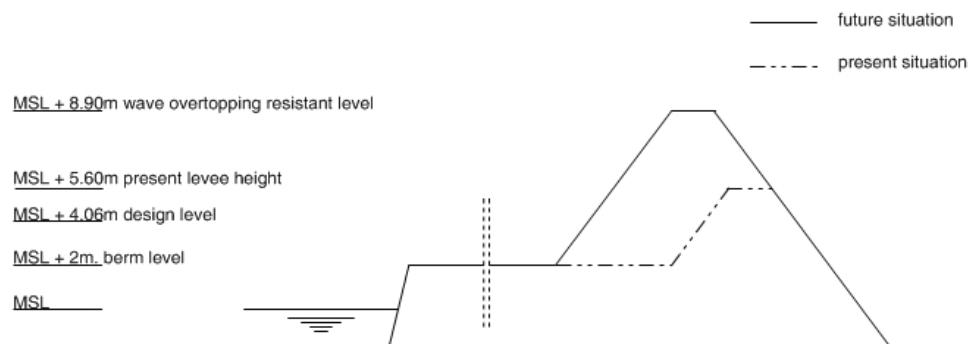

Figure 5.5: Cross section of an earthen levee design

5.1.2 Alternative II T-Wall Levee

In this alternative the levee will have the same principle as that of some of the present levees alongside the canals in New Orleans; an earthen levee with on top a t- shaped floodwall.

Figure 5.6: Floodwall in New Orleans. Image from: NOLA [23].

This type of structure has the same failure modes as a completely earthen structure (see Figure 5.2), but how it copes with these failure modes differs. The most threatening failure modes will be mentioned below, and a general solution will be given for it.

Overtopping   In order to make sure not too much overtopping during a hurricane will occur, the levee must be high enough. The design level is the same as that belonging to an earthen levee, MSL + 4.06m. The extra height needed to cope with the sea level rise and the waves (retaining height) is determined in a different way.

5.1.2.1 Retaining height

The sea level rise remains the same, 0.66 m/century. With a floodwall there will be no wave run-up, the wave will act as a standing wave against the wall. So if we take H_{10}= 5.66 m for the design wave, the extra height needed for the waves is 5.66m. So the retaining height will be 6.32m, resulting in a wave overtopping resistant level of MSL + 10.38 m (34 ft)

Piping   The floodwall is a relatively slender structure, so at the base piping might occur in the case of a big hurricane. This can have detrimental effects, because if the floodwall loses its support it can collapse and the Gentilly District will flood. The only way to counter piping is to extend the piping length. This can be done horizontally and/or vertically. Looking at Figure 5.7 the horizontal piping length is determined by the horizontal part of the T-Wall, the sheet pile determines the vertical length.

If the base of the floodwall is placed on a level that's sufficiently high enough (at about the design level) piping won't be a great threat. This is because piping will not take place instantaneously, it needs some time. This means waves will have little or no influence on piping.

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The base of the T-wall will be at the level of the current crest height (MSL + 5.60m). This is because now the least soil has to be placed or excavated. This means the T-wall will need to have a height of 5.7m; this includes the mitigation of the subsidence of the subsoil. At some places this height will be somewhat too large because of the berm present in front of the levee, but at most places this is not the case, so there this height is needed.
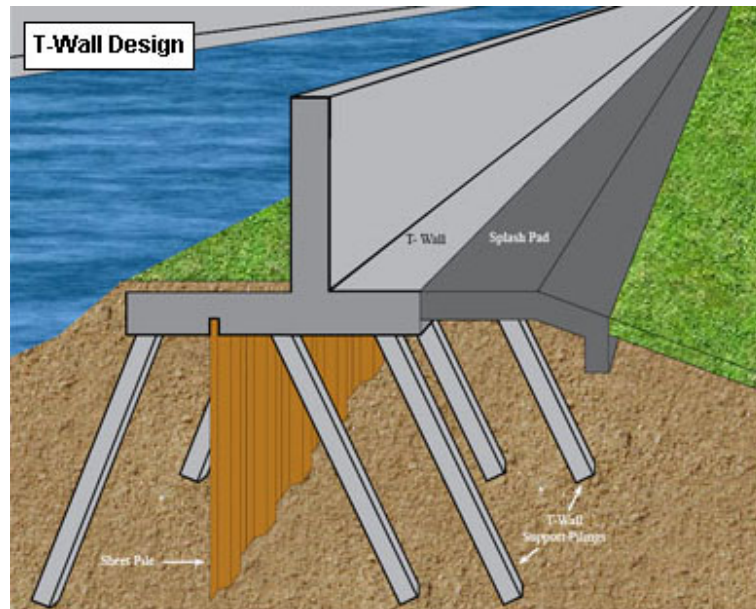

Figure 5.7: T-Wall design . Image from: www.mvn.usace.army.mil [33].

Displacements of the floodwall The floodwall has to be able to withstand the hydrostatic and wave induced forces. The forces are mostly horizontal and can only be countered with the wall's weight and the piles. It is important for the piles to reach until the firm sand layer of the Pleistocene. This layer is at a depth of MSL -25m (see Appendix B.5). The big advantage of T-wall design including the compression and tension piles underneath is that in a structural sense it is very robust. Because the piles are placed under an angle they form a triangle; this geometrical shape is hard to deform.

Erosion of the inner slope Due to overtopping the inner slope can get eroded. If the inner slope lacks a good protection this can happen rapidly, as the water comes from an altitude of about 6 m. Therefore a good revetment is necessary. In Figure 5.7 this revetment is indicated with the splash pad. Good options for this can be an asphalt layer or a layer of placed concrete blocks. Additional advantage of a good splash pad is that it acts as a philter, counteracting the piping.

Failure of individual T-wall sections A T-wall is a hard, concrete structure. It has to be prefabricated and then put into place. Therefore each section has limited length and the whole levee consists of several T-wall sections. Downside is that if one section fails, the whole levee will fail. So each of the sections must be placed precisely, and the connections to their neighboring T-walls must be watertight. Otherwise water could seep through the connections, causing erosion and eventually failure. The most common way to do this is to fill the joints with rubber.

MSL + 11.30m design height T-Wall

MSL + 5.60m present levee crest height

Splash pad

MSL

Tension piles

Compression piles

Sheet piles
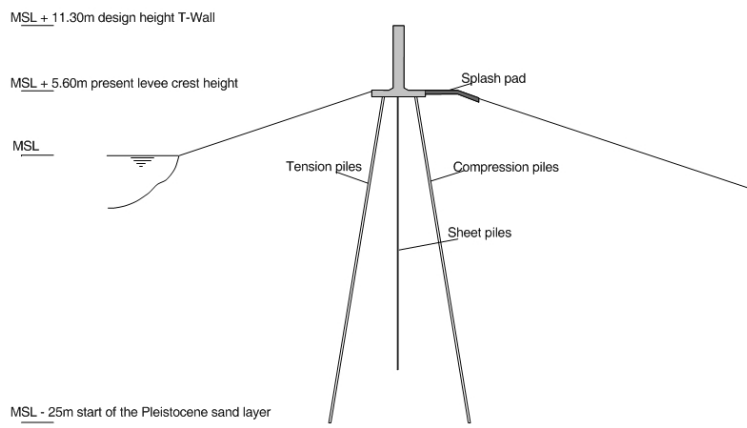
MSL - 25m start of the Pleistocene sand layer

Figure 5.8: Cross section of an T-wall levee design.

5.1.3 Alternative III Floating Levee

One of the problems of New Orleans is that the city is situated under MSL, and quite a bit too. The levee structures that are needed to protect the urban areas such as the Gentilly District have to be of considerable height to be functional. Disadvantages of this height are the costs that are needed for the large quantities of m^3 soil that are used to build the levees, but also the unsafe feelings people get when living near such a structure. A way to deal with these disadvantages is to make a structure that can be raised if the water level gets too high. There are many ways to do this, most of them are either working on manpower or on electrical power. The downside of these options is that these power sources can fail. During Katrina almost all the power in the city was gone; a man may forget to do his job or fall asleep. The alternative elaborated here doesn't have these disadvantages because the hydrostatic force of the surge raises the levee. The idea is illustrated in Figure 5.9.
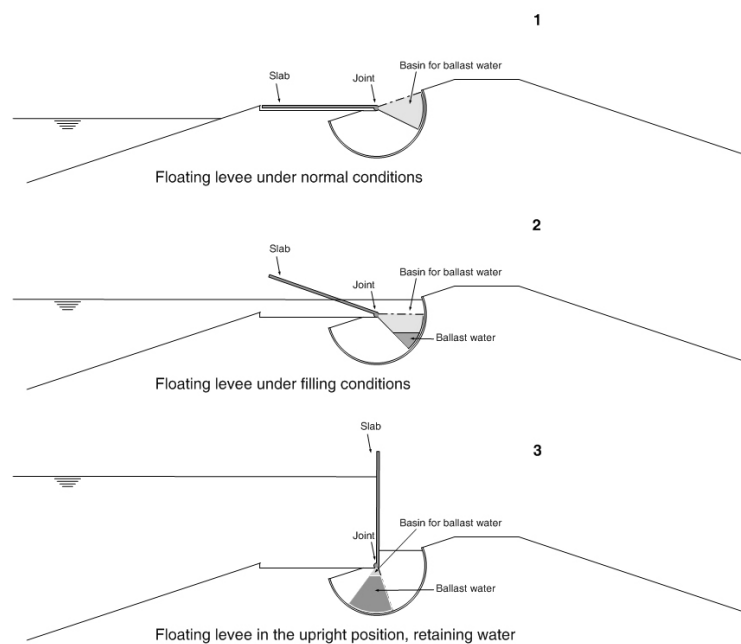
Figure 5.9: Functioning of the floating levee.

The idea is that if the water reaches a certain level, the basin starts filling up. If it is filled with enough water it will become heavier than the slab and the construction starts to pivot over the joint. The result is that the slab is upright, thus creating a wall that can withstand the surge.

The floating levee can be constructed on the present levee, but some adjustments will have to be made. First the levee must be made wide enough so that the slab is able to lie on it. Just behind the floating levee structure the present levee needs to be raised by at least 1m so that during the filling of the basin no or little overtopping occurs. This is necessary because otherwise

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a lot of erosion will have taken place even before the levee is raised, possibly that the levee is weakened to the extent it will fail. The level can be raised using a floodwall or by using earthen embankment, the lather having the advantage that the basin can be stored in here. There are some important details that need to be investigated well.

- The length of the slab. This is dependent on the level where joint is placed. In an upright position, the level the slab has to reach is at least MSL + 10.38 m. this is the same level the T-wall has to reach. If the joint is placed on top of the current levee (at a level of MSL + 5.60 m), and the subsoil subsidence is included, the slab will need a length of 5.70m.

- Weight of the slab. There is not a lot of margin in the weight of the slab. During low water (i.e. the basin is empty) the slab must exert a greater moment on the joint than the basin and during high water (i.e. the basin is full) it is the other way around. Best option is to keep the slab a light as possible and adjust the basin's dimensions to it. Of course the slab has to be able to withstand the pounding waves.

- Stability of the structure and the surrounding levee. Soil has to be removed in order to make room for the floating levee structure, but this has to be done in such a way that it doesn't undermine the levee's stability.

- Emptying the basin after a storm surge. If the water has reached a level that was so high that the floating levees had to be used, afterwards this floating levee needs to be lowered again. The only way to do this is to shed the water in the basin. This could be done using pumps (this is an option because probably the floating levees won't be used more than once or twice a year) or by creating an outlet that will open once the water level has lowered sufficiently.
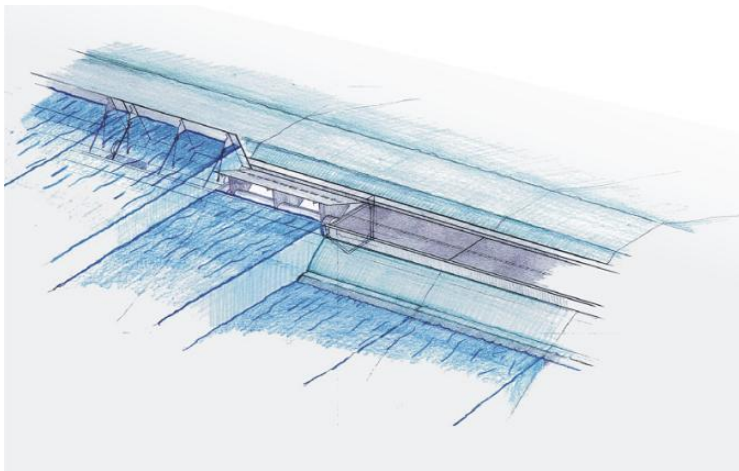
Figure 5.10: Sketch of the floating levee in three stages. Image by: IV-Groep 2006

5.1.4 Choice between the alternatives

The decision of which alternative will be elaborated to a final design is done by evaluating the alternatives with the help of criteria. The following criteria will be used:

- Complexity. The easier a design is to implement, the more favorable.

- Dimensions. Each of the alternatives has a different height and width. A slender and low structure scores better on this point than a high and wide one.

- Innovation. Every new project can be seen as a chance to improve or invent ways to construct.

- Esthetics. The peripheral levee will serve for at least a century, and it's a big eye catcher, so preferably it is a structure that is easy on the eyes.

- Multifunctional aspect. Here the question is whether it is possible to combine the water retaining function with other functions, e.g. infrastructure.

These five criteria are not equally important. In the analysis a weight factor is indicated, this can be 1 or 2.

	Earthen levee	T-wall levee	Floating levee
Complexity (2x)	+	0	-
Dimensions (2x)	-	+	0
Innovation (1x)	0	-	+
Esthetics (2x)	0	-	+
Multifunctional (1x)	+	-	0
Total	+	−	+

Table 5.3: Multi Criteria Analysis (MCA) for the peripheral levees

As can be concluded from the MCA above the earthen levee alternative is as favorable as the floating levee alternative. The floating levee alternative is preferred because the esthetics is considered of slightly more importance than the complexity.

5.2 Floodgate

The strength of a chain of is never stronger than its weakest link. The same rule applies for the New Orleans flood defenses. Presently the levees stretch many kilometers, of which every kilometer has need of regular inspection and maintenance. The failure of one small part of this line of defenses would compromise the level of safety for the entire system. Reducing the length and strengthening the remaining parts of this line would therefor greatly improve their performance and the overall reliability of the flood defenses.

Figure 5.11: Closing the canals with floodgates can significantly reduce the length of flood defenses. Image from: Google Earth [11].

Closing of the London Avenue Outfall Canal from Lake Pontchartrain at its outlet would already reduce the chain of defenses by 10 km. By preventing storm surges from entering the canal, its levees need less retaining height.

5.2.1 Functional demands of closing structure

The basic function of the closing structure is to prevent high water levels in Lake Pontchartrain from entering the canal during storm conditions. Also excessive water from the city's draining system needs to be shed from the canal into the lake. These two conflicting demands can be stated as:

- Retaining water during storm conditions

- Transport of water from the canal into the lake

If the canal is opened for navigation a third functional demand emerges:

- Transport of vessels in and out of the canal

To fulfill the water retaining function the structure needs to have the same retaining height as the surrounding peripheral levees. Also the structure needs to be able to withstand the harsh hydraulic conditions that a hurricane storm surge brings. Transport of water can be achieved by free flow or by usa of hydraulic pumps. Transport of vessels demands for a certain air clearance or headroom for ships to pass through. This clearance can be quite significant for recreational vessels (mast on sailing ships), but bridges further up the canal would also obstruct these ships their passage. Therefore the clearance of the closing structure does not have to be larger than the minimum clearance under the bridges which is about 4 meters above MSL, unless the bridges are elevated.

5.2.2 Alternative concepts

Closure of the canal can be achieved by placing either a fixed or a movable structure at the canal outlet.

5.2.2.1 Fixed or movable structure

A fixed barrier preserves the retaining height of the primary protection system at all times (see Figure 5.12). Drainage water from the canal is shed into Lake Pontchartrain through or over the structure by openings in the barrier. When the water level in the lake becomes too high, the openings can be shut with valves. The fixed structure can be made of concrete with use of steel valves or doors to close the spillway. Common appliances of the fixed barrier are scouring sluices and dam spillways.
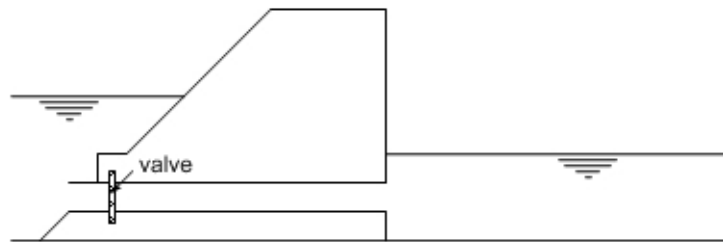
Figure 5.12: Fixed barrier with closable opening.

In contrast to this, a movable barrier or floodgate keeps the connection between the canal and the lake completely open during normal conditions and will only be closed temporarily when a storm surge is expected (see Figure 5.13). This way the canal wil remain open for transport of vessels.

The third demand of vessel transport cannot be met in the form of a fixed barrier. Therefore the choice is made to design a movable floodgate for the closure of the London Avenue Outfall Canal.

5.2.3 Types of movable floodgates

There are many different types of floodgates, most of them hydraulic steel structures (see Figure 5.13). In this section the different gate types and their characteristics will be explained. Subsequently an evaluation and a choice for one of these gates will be made.
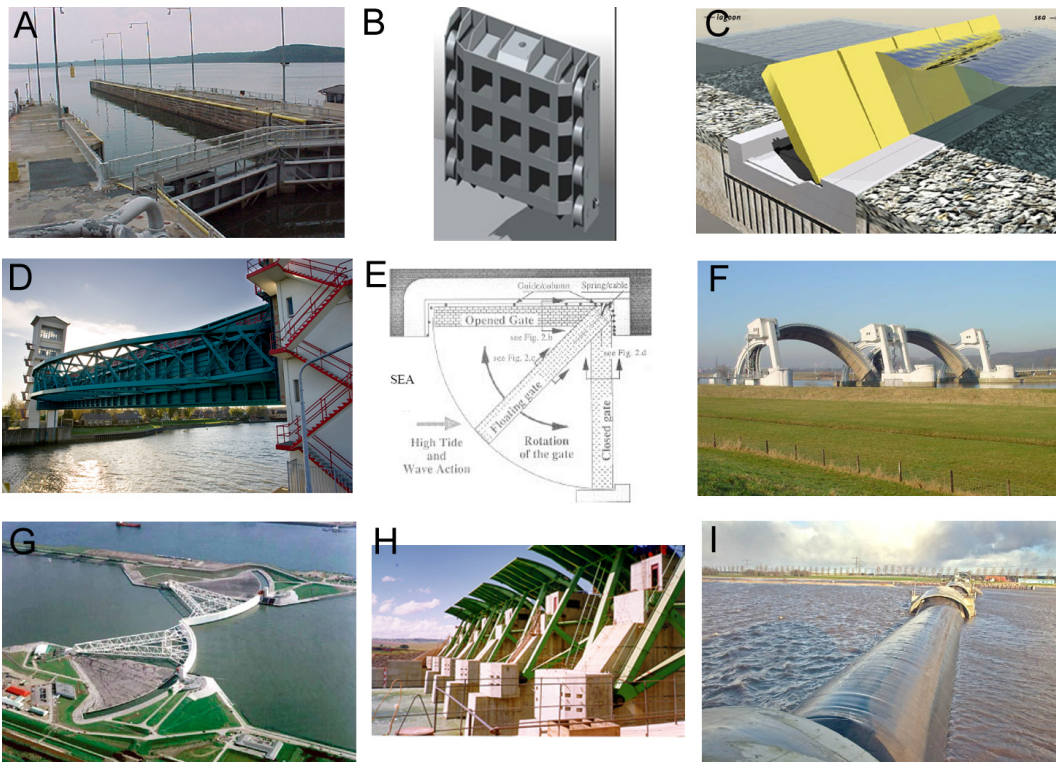
Figure 5.13: Some examples of movable barriers: A. miter gate, Nashville, USA; B. roller gate; C. flap gate, concept for Venice, Italy; D. vertical-lift gate, Hartel barrier, The Netherlands; E. swing gate; F. visor gate, Driel, The Netherlands; G. sector gate, Maeslant barrier, The Netherlands; H. tainter gate; I. inflatable barrier, Ramspol barrier, The Netherlands.

Considering their mechanisms of movement roughly five types of floodgates can be distinguished:

- Gates rotating around a vertical axis

- Gates rotating around a horizontal axis

- Horizontally translating gates

- Vertically translating gates

- Gates with no fixed form

5.2.3.1 Gates rotating around a vertical axis

Miter gate This gate type is frequently used for locks, primarily because they tend to be economical to construct and operate and can open and close more rapidly than other types of gates. Use for flood control is less common. Miter gates consist of two leaf shaped doors that are hinged on either side of the canal. The doors are only operated when the water level on both sides of the structure is equal and usually close at an angle of around 18 degrees to approximate an arch. Hydrostatic forces are transmitted through a series of horizontal and

vertical framework to the abutments.

Miter gates give no limitations on air clearance of passing vessels. They are unsuitable for use in reverse head conditions. Miter gates also become less efficient when their span becomes to large, the width-height ratio and the forces on the hinges become unfavorable. Ranges from 6 m to a maximum of 25 m are usual.

Swing gate The swing gate uses the same principle as the miter gate, yet with a single door instead of two. This door rotates around hinges on one side and close against the abutment on the other side. To improve handling these doors can be made buoyant during movement. Once locked into position the door is able to resist reverse head conditions. Analogue to miter gates heavy hinges are necessary when the length of the door increases.

5.2.3.2   Gates rotating around a horizontal axis

Flap gate Flap gates are submerged gates, hinged on the upstream side to a horizontal axis on the sill. In normal conditions the flaps rest on the bottom of the canal. During storm conditions, the downstream sides of the flaps are rotated upward to close the flow. Rotation is achieved by increasing buoyancy of the flaps, or rotating them mechanically. When locked into position the flap gate can withstand reverse head conditions. A disadvantage of the flap gate is that the movement mechanism is completely submerged and it is therefore difficult to inspect and to perform maintenance on.

Arch or visor gate The visor gate is a three-hinged arch that spans from one side of the abutment to the other side. It is hinged on the upstream side at the abutments and to open or close the canal the arch is rotated up- or downward. Visor gates typically need a large structure height, at least half of the canals width. Air clearance cannot be made larger by lifting the door more.

Sector gate Sector gates have steel doors shaped like circular sections that are fitted with a truss to vertical hinges on both sides of the canal. When opened the doors are rested into sockets on both sides of the canal. Because the hydraulic loads are directed radially inward to the axis there is very little unbalanced load. This enables the sector gate doors to be operated with different head conditions on both sides. Sector gates provide unlimited air clearance and are suitable to be used with large head differences.

(Submerged) Tainter gate or segment gate The tainter gate like the sector gate has a door with the shape of a circle segment, mounted with short radial struts on both sides of the door. Its rotation can be aided by appliance of contra-weights. Hydraulic loads are transmitted through girders to the hinges on the sill or abutments. When the gate is opened it lies submerged in a trench on the bottom sill, which provides unlimited air clearance for passing vessels. The door can also be lifted out of the water, but this restricts air clearance. The tainter gate can be used across long spans and does not need much space adjacent to the canal. It is suitable for use in reverse head conditions. A disadvantage of the submerged tainter gate is the large foundation depth.

5.2.3.3   Horizontally translating gates

Roller gate or slide gate The roller or slide gate is a door sheeted with steel plates on both sides and fixed with rolling wheels or sliding rail to translate horizontally. When open, the gate is stored in the abutment on one side of the canal. To close the flow the door is be moved into

the canal. Loads on the rail can be reduced by making the door more buoyant.

Roller doors can be used across long spans. When locked into closed position the door can withstand reverse head conditions. A disadvantage of the roller gate is its need for space on one side of the canal. The doors socket needs to be as long as the canal is wide and its guiding rails are quite expensive.

5.2.3.4 Vertically translating gates

Vertical-lift gate The vertical-lift has been used in lock gates and spillway gates. Its door consists of a stiffened plate structure, guided by rails on lifting towers on both sides. The door is raised out of the water to open the flow.

An advantage of the lift gate is the easy maintenance of the structure, which is mainly above water. Disadvantages are the limited air clearance and the considerable height of the structure. The lift gate can be operated across wide spans and under moderate heads but not under reverse head conditions.

Submerged vertical lift The submerged vertical lift works the same as the previous gate type, but is lowered into a trench in the sill instead of lifted out of the water to its open position. The submerged lift can be used across wide spans. Disadvantages of the submerged vertical lift are its large foundation depth and the difficult conditions for inspection and maintenance.

5.2.3.5 Gates with no fixed form

Inflatable barrier With the increasing availability of strong and durable synthetic rubbers the inflating barrier gains increasing popularity. A rubber tube is attached to its sill on the bottom of the canal and can be inflated with water, air or a mixture of both to close the flow. Although there is a lot of research in new materials, the vulnerability and durability of the rubber remains an issue.

Due to its flexibility and the way it is fixed to the bottom sill the inflatable barrier is not very suitable for use in reverse head conditions.

5.2.4 Choice of concept

To make a choice between the numerous floodgate types they will be compared in a multi-criteria score chart (see Table 5.4. This chart gives a fast and qualitative insight in how one gate type is advantageous over another. The criteria used to make this comparison are: air clearance, ability to cross large spans, ability to resist reverse head conditions, use of space in horizontal or vertical sense, ease of inspection and maintenance and finally building costs.

The criterium of reverse head conditions is added because of the unpredictable nature of hurricanes. Hurricane winds rotate around its center, the eye of the storm. On one side this causes wind directed landward, while on the other side of the eye wind directions are reversed. This causes a surge elevation on one side and a lowering of the water level on the other. If a hurricane passes the floodgate in such a way that the water from the canal is pushed up against the gate and becomes higher than the water level in Lake Pontchartrain, reverse head conditions exist. This may cause damage to the gate if its not designed to withstand these reverse forces. Furthermore, during a storm the movement mechanisms can be severely damaged and rendered (temporarily) inoperable. In this case the floodgate has to be able to withstand reverse head conditions untill the gate is opened manually or the water is pumped from the canal past the gate.

Although the multi-criteria comparison is no way to definitely find the best alternative, it is quick and useful tool to aid the choice of a structure. From this comparison of the alternatives the most favorable seem to the following gate types:

- Flap gate

- Submerged tainter gate

- Roller gate

- Submerged vertical lift

For further comparison of these four gate types a preliminary design should be made for each of them, thus exposing possible design difficulties and enabling a better cost estimate to be made.

In the limited time frame of this project the choice is made to elaborate on just one of these gates, namely the submerged tainter gate.

A remarkable example of the submerged tainter gate type is the Thames barrier in London (see Figure 5.14). This barrier was constructed in 1974-1982 to protect London against flooding by surge tides. It consists of ten separately movable tainter gates, with spans of up to 61 m. Each of the gates is 20 m high. When not in use the gates rest out of sight in concrete sills in the riverbed, allowing free passage of river traffic through the openings between the piers.

5.2.5 Concept design - Submerged tainter gate

For the closure of the London Avenue Outfall Canal a structure similar to the London Thames barrier can be designed, yet on a smaller scale.

Retaining height The London Ave Barrier will span 20 m (65,6 ft), and should be able to withstand the same surge height as its adjacent primary levees. Figure 5.15 shows the levee elevations along the canal outlet. The total retaining height will then be about 9 m (29,5 ft).

Gate type	Air clearence	Span width	Reverse head conditions	Use of space	Inspection/ maintenance	Construction costs	Total score
Miter	+	-	-	0	0	+	0
Swing	+	-	+	-	0	+	+
Flap	+	+	+	+	-	-	++
Visor	-	-	-	-	+	0	—
Sector	+	0	+	-	0	0	+
Submerged tainter	+	+	+	+	-	-	++
Lift tainter	-	+	+	-	+	0	+
Roller	+	+	+	-	0	0	++
Vertical-lift	-	+	-	-	+	+	0
Submerged vertical	+	+	+	+	-	-	++
Inflatable	+	+	-	+	-	-	+

Table 5.4: Multi-criteria score chart for different floodgate types. Scores: - unfavorable, 0 neutral, + good
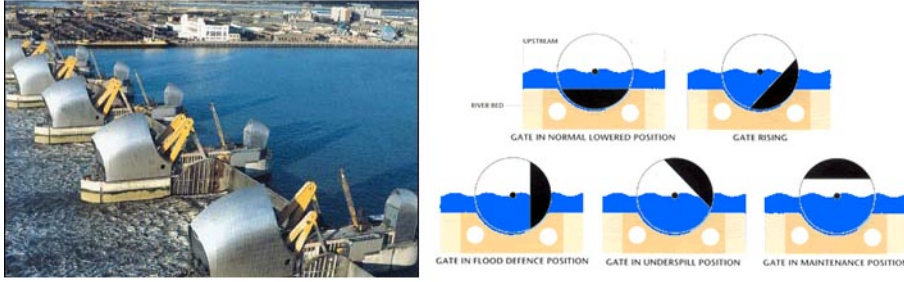
Figure 5.14: Thames barrier in London, United Kingdom.

To protect the secondary levees along the London Avenue Outfall Canal, the floodgate will close under hurricane category 1 conditions. The corresponding storm surge height of MSL+1.22m plus tidal elevation and future sea level rise results in closure at a water level of MSL+2.28m. After closure the gate has to withstand a head difference of several meters, depending on the height of the storm surge and the wave characteristics in front of the gate.
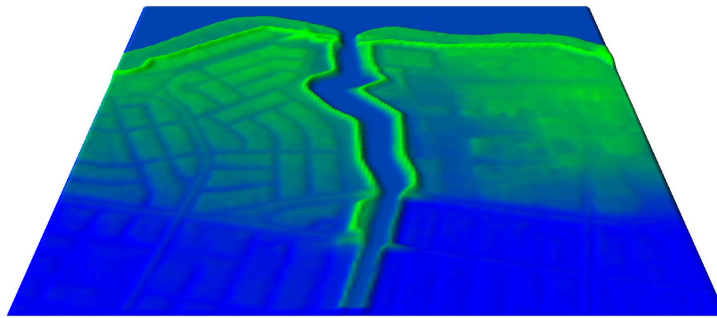

Figure 5.15: Primary protection around the London Avenue Outfall Canal. Green areas are elevated above sea level. Image from: USGS [39]

Location   The exact head differences and dynamic loads the gate will have to endure depend on the location of the structure in the canal outlet. Seen from Lake Pontchartrain the floodgate will be located about 700 m into the canal (see Figure 5.16). The bending shape of the canal here will cause diffraction of incoming waves. Also wave heights will be reduced significantly because of the relatively shallow waters in the canal entrance. Thus, the waves will have broken and have lost most of their energy when they reach the floodgate. This will prevent the gate from being battered by high braking waves. Due to their relatively short, but high impact loads breaking waves are not representative the total stability of a construction, but can cause (partial) structural failure. Therefore it is better to prevent breaking wave impacts on the structure.

Lifting mechanism   To move the gate into its open or closed position hydraulic arms are used. A hydraulic system is reliable and can exert high forces in two directions with only a limited supply of power. Disadvantages are the limited reach of the arms and slow movement. These

Figure 5.16: Location of the floodgate. The shape of the canal outlet will cause diffraction of incoming waves. Image from: Google Earth [11].

disadvantages however have no negative consequences for the radial floodgate.

Design sketches   Figures 5.17 and 5.18 shows a plan view of the floodgate and a cross section. These are just conceptual sketches; actual dimensions have to be determined.

PLAN VIEW
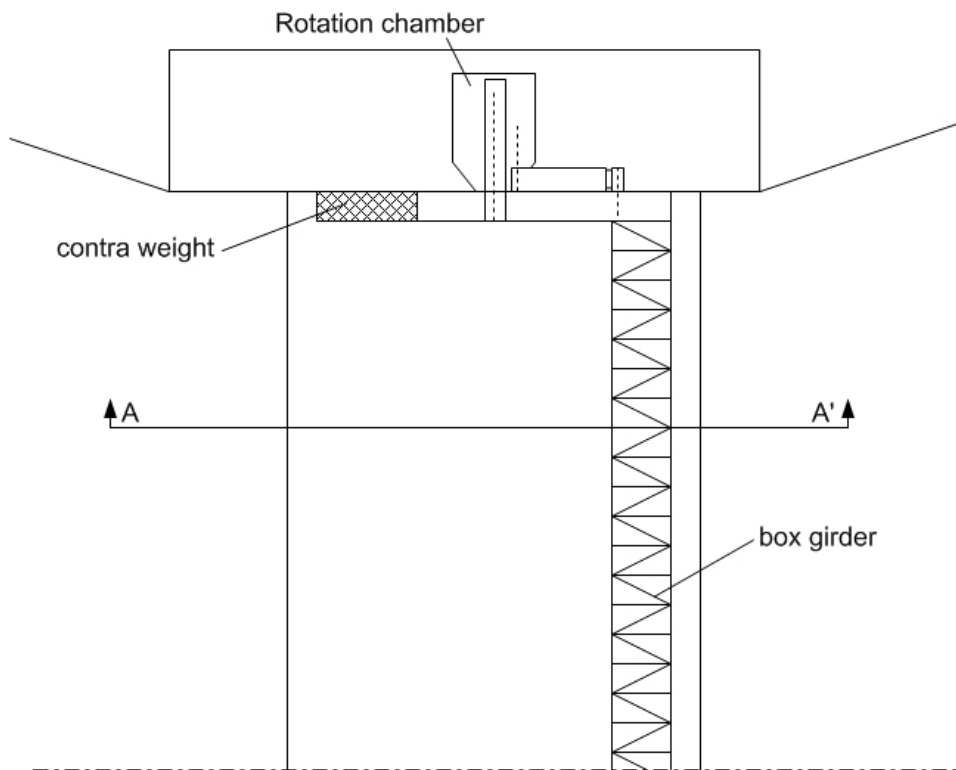
Figure 5.17: Conceptual drawing (plan view) of the London Avenue Outfall Canal floodgate.

CROSS SECTION A-A'

Hydraulic arm

Gate in closed position

Gate in open position

Bed protection

Sheet piles
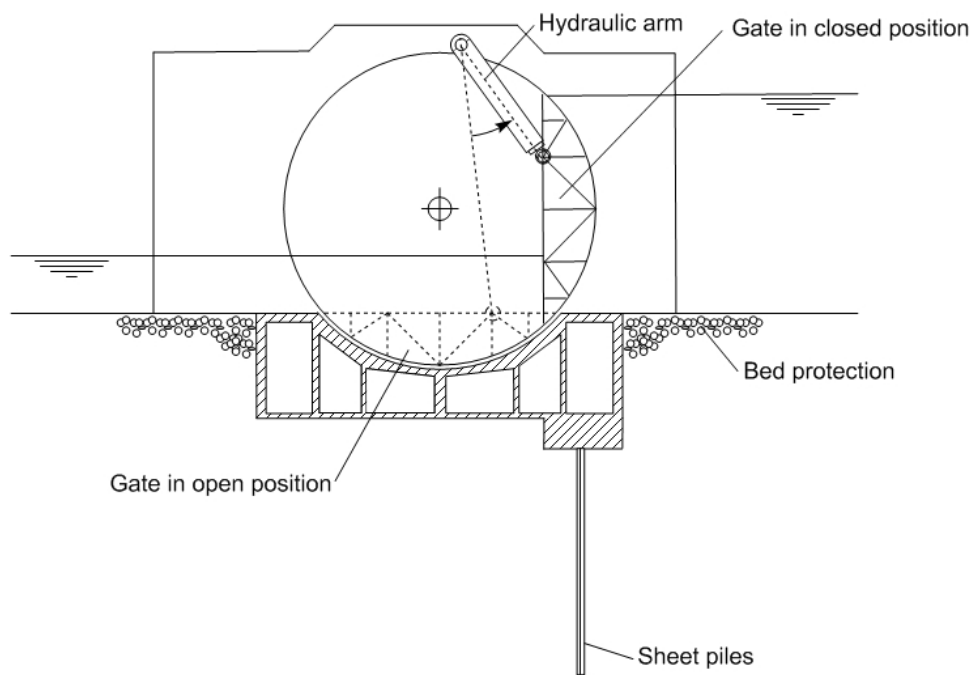
Figure 5.18: Conceptual drawing (cross section) of the London Avenue Outfall Canal floodgate.

6

Secondary protection

In this section a new design will be presented for the water management system of the Gentilly District. This system consists of several drainage facilities and the London Avenue Outfall Canal to transport drainage water to Lake Pontchartrain. First a brief overview of the present situation will be given. Subsequently a new solution for the water management of the area is described.

6.1 Present situation

6.1.1 Drainage pumps

The Gentilly District spans an area of about 17.5 km^2 in the north east section of the Orleans East Bank. Ground water levels in the area east of the London Avenue Outfall Canal are controlled with one drainage pump, pump #4, located on the east side of the London Avenue Outfall Canal. The west side of the canal is controlled by pump #3, at the southern point of the canal (see Figure 6.1). The maximum discharge of the pumps into the London Avenue Outfall Canal is 221.4 m^3/s (7820 cfs).
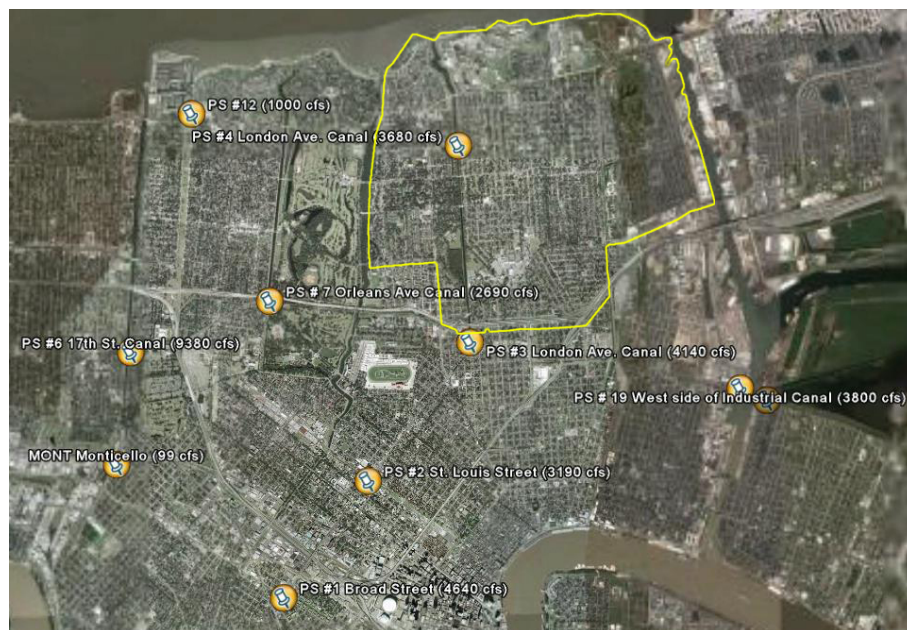
Figure 6.1: Pump locations and capacities (cfs) in the Orleans East Bank. The Gentilly District is indicated with a yellow line. Image from: Google Earth and USACE [11] [34].

6.1.1.1 Outfall canal

The pump stations discharge drainage water into the London Avenue Outfall Canal, which has an open connection to Lake Pontchartrain. The canal has a length of about 5 km and a width varying from 30 m in the south, 40 m halfway, to a maximum of 90 m in the north. The outlet to Lake Pontchartrain has a width of about 40 m.

In the present situation these floodwalls are designed to withstand storm surge water levels for a hurricane category 3. Their height ranges from MSL + 4.38 m in the north to MSL + 4.51 m in the south.

6.1.1.2 Performance during Katrina

During Katrina the levees alongside London Avenue Outfall Canal breached on both sides, allowing the water to flow freely into the urban areas. Figure 6.2 shows the flood map of the Gentilly District. The blue parts in this picture show that the area acted like a huge bath tub with a width of approximately 1.4 km, with parts flooded as much as 14 ft (4.26 m). All the low leveled areas are in the present situation directly connected to one another, meaning that if one floods, the others will follow. Subsequently all this water will have to be pumped out again, which may demand a bigger pump capacity than there is available.
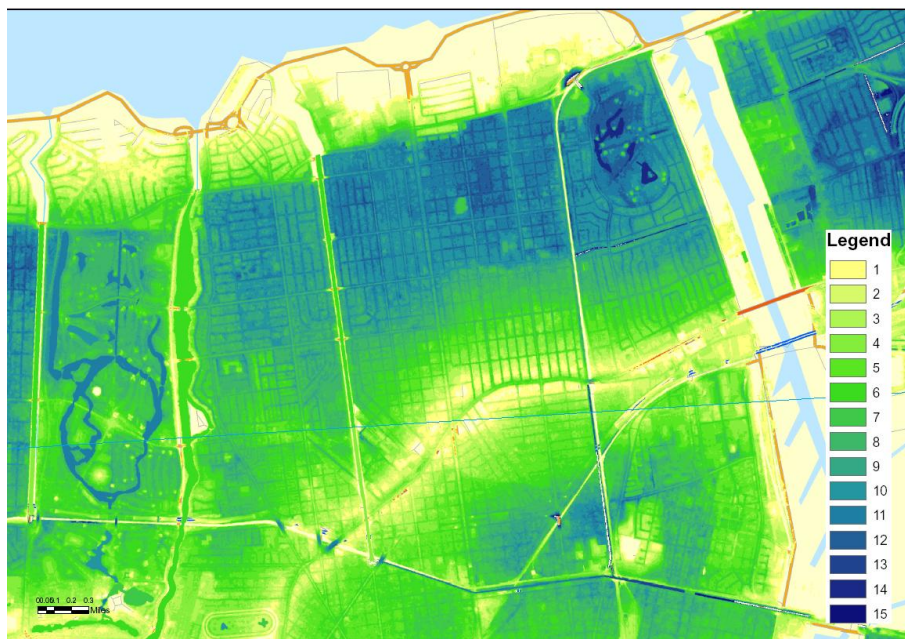
Figure 6.2: Flood map (ft) during Katrina. Data by Braud and Cunningham [31].

6.2 Future situation

Above some of the weak spots in the water management of the Gentilly District were mentioned. The improvements of these can be divided into the prevention and the mitigation measures. The prevention implies that there will be no flooding i.e. the levee system will function adequately; the mitigation implies that should a flooding occur, its consequences will be kept to a minimum.

6.2.1 Prevention: levee system of the Canal

The best option for the Gentilly District is the one that will stop the surge from entering the London Avenue Outfall Canal altogether. This is the reason a design is made for a floodgate at the outlet of this canal. This implies that the levees won't have to be able to withstand storm surges anymore. A suggestion could be to remove the existing floodwalls and transform the levees into earthen embankments. This has a positive effect on the spatial quality of the area; the canal is added to the urban areas and is no longer separated from them. This allows room for recreational opportunities (e.g. fishing and sailing on the canal).

In the design of the levee cross section this has to be taken into account. For the levee height the storm surge is no longer the priority; the canal must be able to convey the pumped water to Lake Pontchartrain. Design water levels will be a combination of the water pumped into the canal and the tidal fluctuation of Lake Pontchartrain.

During a hurricane the floodgate will be closed, keeping the storm surge out but also keeping the drainage water in. The levees must be designed in a way that they are able to store the precipitation that is pumped from the urban areas in the canal during the closure of the floodgate. The floodgate will close when a category 1 hurricane enters Lake Pontchartrain. Combined with the tidal elevation and the relative sea level rise the subsequent water level will be MSL + 2.28m (7.6 ft). After closure of the canal its water level remains the same and will only rise by storage of drainage water that is pumped in during this closure. In cases of excessive rainfall this could cause a problem.

A hurricane can be accompanied by excessive rainfall, for example precipitations of 0.5 m (20 inches) falling in a 12 hr period have been measured during hurricane Allison in 2001 [26] and comparable rainfalls with other hurricanes in the US south east coastal area. For the entire Gentilly District area this would amount to 8.75 million m^3 of water. To prevent inundation, precipitation water should be removed during the storm at a rate of 202.5 m^3/s. If the precipitation is so intense, some local flooding cannot be avoided. But the local flooding will be short term, in about a day the excess of water will be removed.

There is not enough retention capacity in the canal to store this much water. Increasing the retention capacity by widening the canal would still not provide the storage needed. Therefore the water should be removed from the canal into Lake Pontchartrain at the same rate. An extra pump facility at the outlet of the canal can provide for this.

6.2.2 Cross section of the levees

With the removal of the floodwalls the levee has gotten some additional functions. An important aspect is making the canal an integral part of the residential area. This implies that the levees need to be accessible to and attractive for people. One way to do this is to make the crest of the levee wide enough for recreational activities such as strolling, biking and fishing. A crest width of 8 m (26.2 ft) should be sufficient. The base width depends on the levee slopes on both sides and the local ground elevation.

To maintain the present hydraulic profile of the canal as much as possible the canal width will be kept at its present value. The sheet piles that are currently present in the canals can be maintained. The width of the canal at the southern point is 30 m, broadening to 40 m halfway and with a wider part in the northern section. The exact profile of the levees depends on were the cross section is taken along the canal.

A representative cross section is given in Figure 6.3. An inner slope of 1:3 and an outer slope of 1:3 are chosen. For the crest height the following value is chosen:

```
MSL      + 2.28 m      (water level at which floodgate closes)
         + 0.92 m      (general ground subsidence)
         + 0.50 m      (surplus height)
       - - - - - - - - +
MSL      + 3.70 m
         + 0.40 m      (subsidence of newly placed soil)
       - - - - - - - - +
MSL      + 4.10 m
```

The resulting design height is MSL + 4.10 m (13.7 ft). The canal width at this section is 40 m and the ground elevation is approximately MSL - 1.0 m on the west bank and MSL - 2.0 m on the east bank. The minimal water depth needed for the boats to be able to sail is 1.5m. Combined with the tidal fluctuation this implies that the bottom level will have to lie at a level of MSL - 1.9m at the most. To be able to comply there is little to no dredging required.
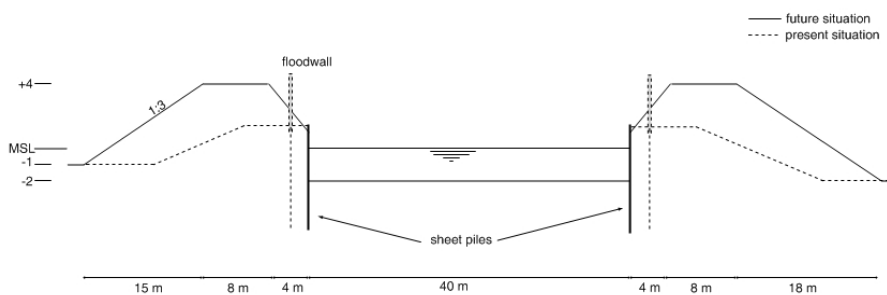
Figure 6.3: Representative cross section of the future canal levees.

7

Tertiary protection

After the secondary levees breached many houses suffered substantial damage from flooding and inundation. Though many homes remain abandoned, some owners are returning and are jacking up their houses to meet the new recommendations. In this chapter some proposals are discussed that offer alternative solutions to building houses on piles. First an overview is given of current building types and existing flood proofing measures. Dutch readers are recommended to read the Appendix with important design loads (see Appendix D). Afterwards each new solution is compared to each other and to the current practice of building on piles.

7.1 Overview and performance of current solutions

Currently existing residential structures can be distinguished and subdivided based on performance of their foundation type. Though all types of buildings were heavily damaged or completely destroyed, certain foundation types proved to be better equipped to cope with flooding. Furthermore, structures where the foundation was part of the structural frame, extending above the lowest floor, often survived with examples of steel, concrete and timber frame structures. Near the levee breaches along the London Avenue Outfall Canal, large flood waters overran and damaged all residential buildings. Foundation types observed were either slab-on-grade or built on pier columns.

7.1.1 Slab-on-grade buildings

Slab foundations are relatively simple and cheap. Concrete is poured in a mold set on site. Because there are no hollow or crawlspaces access to utilities and ducts can be limited. Special care has to be taken when assessing soil conditions. Soil compaction prior to pouring is often applied to avoid long-term settlements or possible subsidence.

It should be noted that because the whole structure is built on grade it is completely exposed to flood and water induced forces. Hydrodynamic forces will fully load the building envelope. Without piles carrying tensile forces, only the mass of the building can counteract buoyancy caused by hydrostatic forces.
Slab-on-grade foundations are also susceptible to erosion and therefore prohibited or discouraged in certain coastal areas by FEMA [17]. In New Orleans Parish the new recommendations requiring an elevation of at least 0,9 m (3 feet) above grade will make this type of foundation obsolete.

7.1.2 Elevated buildings

In Louisiana many examples of elevated buildings can be found. Within the Gentilly District many homes are sited on foundation piers. In coastal areas standards set by FEMA demand homes to be raised above a certain height, so structures raised on piles are a common solution. Both types have 'open foundations', while a third type of elevated structure, a building supported by a stem wall foundation, is referred to as a 'closed foundation'.
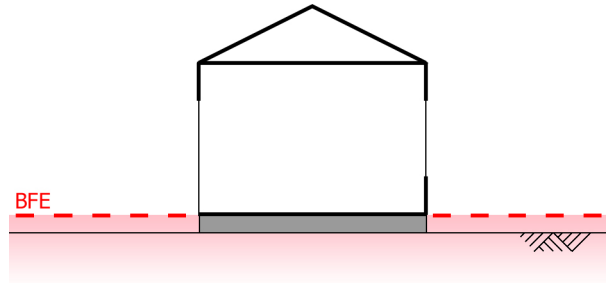
Figure 7.1: Slab-on-grade building needs to be raised to BFE or at least 0,9 m above grade, whichever is higher.

Figure 7.2: Slab-on-grade building in Gentilly District, New Orleans.

7.1.2.1   Pier column foundations

As mentioned, pier column foundations are common within New Orleans as a conventional solution for elevating structures in levee protected areas. Steel or timber piles are driven to a depth sufficient to provide the necessary load-bearing capacity. The piles support concrete grad beams which are poured on site. Pier columns, either concrete or masonry cinderblock, are fitted with grouted steel anchors. The typically wooden frame of the house is connected with bolts to the piers.

Pier failures during Katrina were usually a combination of insufficient reinforcement or inadequate splicing, shallow footings or poor connections between the piers and the footing. In some cases the flood elevation led to buoyant forces on the buildings. These buoyant forces in combination with insufficient anchoring caused them to float off of their piers [16].
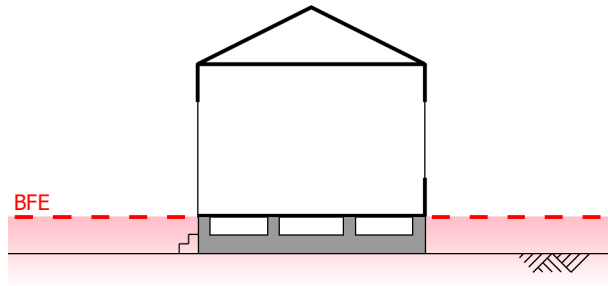
Figure 7.3: Building with concrete grad beams and concrete or cinderblock piers.

Figure 7.4: Pier column foundation in Gentilly District, New Orleans.

7.1.2.2 Pile foundations

In coastal zones, the lowest floor of a home has to be elevated to at least the Design Flood Elevation (DFE), which takes possible storm surges and corresponding wave height into account.

The conventional solution is a driven, treated timber pile foundation or 'stilt' foundation. Piles are driven deep enough as to provide sufficient embedment strength and well below the frost, erosion and scour depth. Pile foundations that met these standards generally performed well during the hurricane.
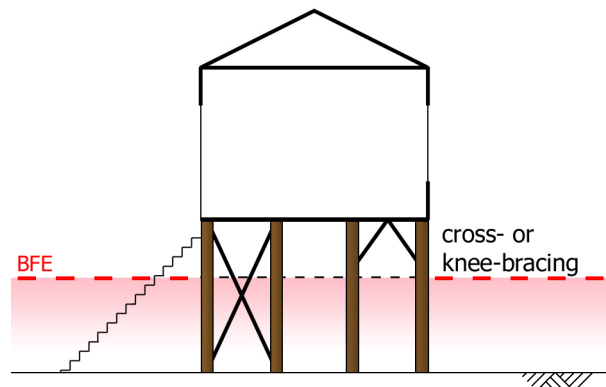
Figure 7.5: Building on timber or steel piles and cross- or knee-bracing.

Figure 7.6: Pile house in Florida.

The timber piles are typically tapered and have round or square cross-sections. Standard available piles allow elevations up to 3 m. The treatment of piles counteracts insect damage and rotting, though the latter still occurs if the pile is frequently, but non-continuously submersed in water.
In some cases higher elevations have been achieved by the addition of concrete grade beams

below a certain scour depth. In this case water retention between the connections of concrete to timber can decrease the structural integrity.

Steel rod cross-bracing, preferably perpendicular to the shoreline, is often used to improve the overall stability. In some cases knee-bracings are used to reduce obstruction for waves and debris.

Piles extended above the lowest floor and connected to a diaphragm on a higher level act as structural frames and also improve the resistance of the building to lateral forces and overturning moments.

Less common is the use of steel piles, which is more expensive. Open steel driven pipe pile foundation can be used for higher elevations or reducing the number of piles needed, due to the higher axial and bending strength of steel. Steel piling is also more resistant to erosion and scour. Additional cross-bracing or knee-bracing is possible and connecting the piles below grade with reinforced concrete grad beams is standard for steel pile foundations.

7.1.2.3 Stem wall foundations

A stem wall foundation can be used to raise the lowest floor above the surrounding ground. Stem walls anchors a foundation in the ground, hold a house down and tie it in the ground below. The space enclosed by the stem walls can be filled with engineered compacted fill.
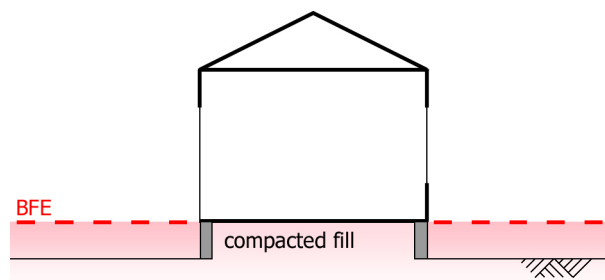
Figure 7.7: Building on a masonry stem wall with compacted fill.

This type of foundation is ideal for areas prone to high water, flooding and storm surge. When high water comes, it moves around the foundation, unable to get under the slab and lift it up. Preventing structural damage from storm surge ultimately saves the structure if it is additionally able to withstand the forces of wind uplift and wind-driven rain or debris.

7.2 Existing measures in flood-prone areas

Flood proofing is the method of making a structure resistant to flood damage, either by taking it out of contact with water, or by making it water resistant. Flood proofing can be subdivided into two types of categories:

- Dry flood proofing

- Wet flood proofing

And

- Active flood proofing

- Passive flood proofing

A last measure is the use of 'break-away' components which are sacrificed to altogether avoid the transference of loads to the structure.

7.2.1 Dry flood proofing

Sealing a building so water cannot enter is called dry flood proofing. The interior, spaces, equipment and contents of the building stay dry. All areas below the flood protection level are watertight. Walls are coated with waterproofing compounds or plastic sheeting. Doors, windows and ventilation openings are closed, with removable shields or sandbags. Flood shields, panels, doors and gates are all used to close openings. They can act as temporary closures that are installed only when a flood threatens, or they can be permanent features that are closed manually or automatically.

Dry flood proofing is only appropriate for structures on a concrete floor slab, without basement and with no cracks. As a rule the maximum flood protection level is two feet above the ground slab. Higher water levels will put pressure on the walls and floor slab, which they cannot withstand. The walls will collapse and the floors buckle [24].

Water will cause lateral hydrostatic forces and the vertical buoyant forces on a structure containing a dry flood proofed area. These forces can be quite large, and in the case of small or lightweight buildings, can be sufficient to cause walls to fail or buildings to float out of the ground. All structural components must be capable of resisting hydrostatic and hydrodynamic flood forces, including the effects of buoyancy and anticipated debris impact factors.

The use of dry flood proofing is required when constructing a non-residential building located below the BFE in an A, AE, AH or AO flood zone. The rule also applies to mixed-use structures where the ground floor is identified as non-residential. Dry flood proofing is not permitted in V-zones [14].

7.2.2 Wet flood proofing

Wet flood proofing means letting the water in and removing everything that can be damaged by a flood. The water pressure around the building will be equalized, by the entry and exit of water. Fuse and electric breaker boxes should be located so you can safely turn the power off to the circuits serving flood prone areas.

There are two methods to calculate the total required net opening. The prescriptive method calls for one square inch of flood opening for each square foot of enclosed area below the DFE, and is based on conservative assumptions regarding flood characteristics and flow through the openings. The engineered method allows designers to size openings based on site-specific flood characteristics and opening shapes. The engineered method usually results in a smaller total net opening area than the prescriptive method does [24].

Building materials exposed to flooding must be resilient enough to sustain a certain amount of water exposure in order to avoid complete replacement after the flood. FEMA defines a flood-resistant material as any building material capable of withstanding direct and prolonged contact (i.e. at least 72 hours) with floodwaters without sustaining significant damage. The following are examples of flood-resistant materials [15]:

- Lumber (pressure-treated or naturally decay resistant, including redwood, cedar, some oaks, and bald cypress)

- Concrete (a sound, durable mix, and when exposed to saltwater or salt spray, made with a 28-day compressive strength of 5.000 pound per square inch (psi) minimum and a water-cement ratio no higher than 0.40)

- Masonry (reinforced and fully grouted)

- Structural Steel (coated to resist corrosion)

- Insulation (plastics, synthetics, closed-cell foam, or other types approved by local building officials.

Wet flood proofing is not attainable for one storey houses because the flooded areas are the living areas. But many basements, garages and accessory buildings are flood proofed.

Application of wet floodproofing as a flood protection technique under the NFIP is limited to specific situations in A Zones (including A, AE, A1-30, AH, AO, AR zones). For certain uses and types of structures described in this bulletin, communities may allow wet floodproofing only through the issuance of a variance from certain floodplain management requirements. For structures in V zones (includes V, VE, V1-30 zones), more stringent design and construction requirements have been established for the portion of a structure below BFE [13].

7.2.3   Active and passive flood proofing

Active flood proofing is also known as emergency flood proofing and requires human intervention to implement actions that will protect a building and its contents from flooding. This kind of proofing works when there is ample warning time to mobilize people, equipment and flood proofing materials.

Examples of active flood proofing methods for buildings are [24]:

- Temporary flood shields or doors (on building openings)

- Temporary gates or panels (on levees and floodwalls)

- Emergency sand bagging

- Temporary relocation of vulnerable contents and equipment prior to a flood, in conjunction with use of flood-resistant materials for the building

Passive flood proofing requires no human intervention. The building (and/or its immediate surroundings) is designed and constructed to be flood proof without human intervention.

Examples of passive flood proofing methods for buildings are [24]:

- Waterproof sealants and coatings on walls and floors

- Permanently installed, automatic flood shields and doors

- Installation of backflow prevention valves and sump pumps

- Continuous dikes, levees or floodwalls, with automatic interior drainage systems

- Use of flood-resistant materials below BFE

- Installation of flood vents to permit automatic equalization of water levels

- Elevation of vulnerable equipment above BFE

7.2.4 Break-away components

A break-away wall is a wall that collapses under specific lateral loading conditions (wind, water) and that is not part of the structural support of the building. They are designed for use on the ground floor of buildings in coastal flood zones. The NFIP suggest that buildings in coastal zones be built on pilings and that the ground floor be used only for access, parking or storage. Property owners who choose to enclose this space are urged to use walls that will break-away from the rest. A break-away wall must collapse without causing collapse, displacement or other structural damage to the elevated portion of the building or supported foundation system. But the wall must withstand forces of wind and every day use. Current NFIP regulations require that the break-away wall shall have a design safe loading resistance of not less than 10 and no more than 20 pounds per foot.

NFIP minimum standards require that buildings constructed in V Zones be elevated on piles or columns so that the bottom of the lowest horizontal structural member of the building is above the BFE. The area below the lowest horizontal member must be left free of obstructions or enclosed with non-structural break-away walls, insect screening, or latticework [10] [32] (For the specifications of a break-away wall see Appendix D.3).

7.3 Alternative solutions for tertiary protection

7.3.1 Floating or amphibious structures

The idea of using water rather than soil to support a home is hardly a novel one. The concept of a house boat exists all over the world. In many cases, living on the water was an answer to issues of mobility, efficient use of space, legislation or was simply because of the high quality space that lakes and rivers offer. House boats can often be found in fishing communities, swamps and dense urban areas.

The idea of a floating structure presents a simple solution for flood prone areas. Normally a structure needs enough weight to counteract buoyant forces and is usually not able to handle long-term inundation. This solution however uses its lightweight composition and buoyancy to stay afloat and wait out any period of flooding. Because this particular house will primarily be supported by ground soil and merely has to be capable of floatation in a worst case scenario the term 'amphibious structure' is perhaps more appropriate.

Figure 7.8: House boat.

7.3.1.1 Existing solutions for floating structures

The offshore industry provides some examples of floating structures, fixed to a specific location. The principal materials for offshore construction are steel and concrete. Steel is coated to prevent corrosion. The preferred material for permanent floating homes is concrete because of its low maintenance requirements and reliability when it comes to water tightness. Standard concrete requires 8 to 10 cm wall thickness for adequate water tightness, so often solutions are sought by adding fiber-reinforcement or high strength, lightweight composite materials [20]. The nature of these concrete mixtures offers possibilities to optimize the shape of the structure and stress distributions by using tailor-made polystyrene formwork. In rare cases plastics or other similar synthetic materials have been used as well. In some cases up to four times less concrete volume was required.

In the Netherlands, some institutes and companies are investing in research and development of floating and amphibious systems for residential housing, greenhouses and infrastructure. Several prototypes and pilot projects exist and have proven this to be a feasible solution. Relatively new is the 'floating brick', a building block providing the basis for a modular system to create completely floating communities. The building blocks are either orthogonally or hexagonally oriented and are made of polystyrene foam and high strength, lightweight concrete.

7.3.1.2 Design issues of floating or amphibious structures

Many typical issues associated with marine structures are also applicable to this amphibious solution, such as integrating services and ductwork. The main problems that have to be dealt with are the possibility of sinking, stability and positioning.

7.3.1.3 Stability and positioning

An important issue of marine and floating structures is stability. The stability is determined by the center of gravity, the center of buoyancy and the waterplane moment of inertia. The center of gravity G is a geometric point through which all gravity forces of the structure act as one. Likewise the center of buoyancy B is a point below the waterline through which all the buoyant forces act. In a stable situation these points are aligned along the geometric centerline of the structure. As the structure rolls, for instance due to wave action, the geometric centerline

is no longer vertical and the center of buoyancy shifts. The centerline and the vertical line intersect through the axis of rotation, called the metacenter M. These parameters along with the waterplane moment of intertia determine the stability of a floating structure and indicate if it is able to right itself.
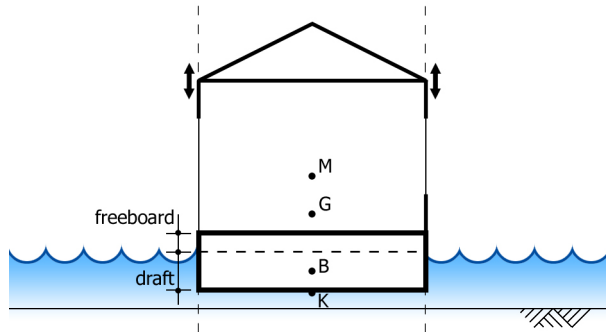
Figure 7.9: Stability of a surface floating vessel.

The formula for stability of a surface floating vessel is:

$$\overline{GM} = \overline{KB} - \overline{KG} + \overline{BM} \tag{7.1}$$

Where

K   =   Geometric centerline at hull bottom
G   =   Center of gravity
B   =   Center of Buoyancy
M   =   Metacenter
I   =   Transverse moment of inertia of the water plane area

$$\overline{BM} = \frac{I}{V} \tag{7.2}$$

Where

V   =   Displaced volume

$$\begin{aligned} I &= 1/12lb^3 \\ V &= \frac{b}{d} \\ I/V &= \frac{b^2}{12d} \end{aligned} \tag{7.3}$$

Where

l   =   length
b   =   beam
d   =   draft

The formula also shows that the width has a quadratic and the draft an inversely proportional relation to the stability of the system. In other words, a shallow, wide base with a low center of gravity provides relatively higher stability.

Other measures can be taken to stabilize the system. These include:

- Mooring lines and anchors

- Fixed guide structures

Because the house has a fixed location, it is essential to implement some of these measures to prevent it from floating out of position. However, when no flooding occurs, a stabilizing system could likely be visible and as such has to be esthetically pleasing or hidden. As stated, a shallow base improves the stability, which is also ideal as it limits the overall height of the structure. Mooring lines, anchors and fixed guide structures could all be integrated or hidden within the structure. The latter offers the best possibilities for precisely fixing the house to its regular position. The structure itself still has to be stable enough in the event these stabilizing elements should fail.

Two major issues arise that are unique to an amphibious structure within an urban environment. The substructure required for adequate buoyancy is quite substantial in volume and would be visible on ground level, which could be esthetically unpleasing. This also introduces the need for impractical steps at the entrances, effectively removing the major advantage of a floating structure; that it no longer needs to be elevated and is accessible from grade. An obvious solution would be to use a basement level for the substructure during normal operation, though this adds considerable costs to the design.

To give an indication of the size of the substructure, a calculation is made assuming 'floating bricks' are used for the substructure and the required freeboard is 0,3m [21].

$$F_{grav} = F_{house} + (\rho_{conc}g\phi_{conc} + \rho_{eps}g\phi_{eps}) * (f + d) \tag{7.4}$$

Where

F_{house}	=	5,8 kN/m^2
ρ_{conc}	=	density of concrete, or 2400 kg/m^3
ϕ_{conc}	=	volume fraction of concrete
ρ_{eps}	=	density of expanded polystyrene foam, or 16 kg/m^3
ϕ_{eps}	=	volume fraction of expanded polystyrene foam
g	=	gravitational constant
d	=	draft
f	=	freeboard

F_{grav}	=	5,9 kN/m^2 + (24,0 kN/m^2 * 0,05 + 10,5 kN/m^2 * 0,95) * (0,3 + d)
F_{buoy}	=	10,0 kN/m^2 * d
ΣF_i	=	0

The draft is calculated by:

F_{grav}	=	$F_{buoy} \rightarrow$ 5,9 + 1,35 * d + 0,4 = 10,0 * d
d	=	6,3 / 8,65 = 0,73 m

The minimum total height of the substructure is 1,03 m.
For a traditional concrete system, the same calculation with 20% concrete volume and 80% air leads to a minimum total height of 1,71m (5,6 ft).

A second problem is the presence of debris during floatation. As the structure floats, floodborne debris could end up beneath it, posing problems as the floodwaters recede. The house would no longer return to its former position and foundation, but for instance would end up resting on top of a car. To prevent such a situation the perimeter of the structure should be secured or somehow take debris into account. These two issues will have to be addressed in a final design stage.
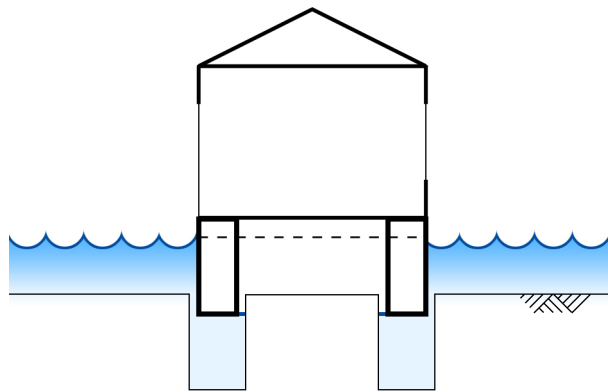
Figure 7.10: Amphibious floating structure with secure perimeter and below grade.

7.3.2 Structures raised on fill

Fill can be used to elevate sites above the BFE and protect them from flood damages. Elevation on fill has been used to protect against flooding depths in excess of 10 feet depending upon the characteristics and availability of fill material. The use of fill for elevating structures is widely practiced throughout the United States [7].
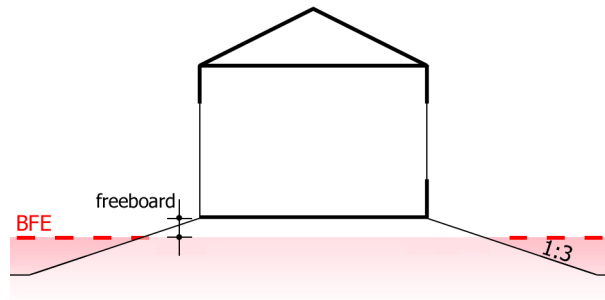
Figure 7.11: Fill used to elevate the site above BFE

The design of an earth fill primarily consists of establishing the soil profile. A geological cross-section showing the horizontal and vertical extend and classification of soil strata is needed, with the following information:

- Soil profile of the fill site [2].

  - Nature and origin of the natural soils
  - Nature of potential and/or existing drainage problems.
  - Position source and the history of any fluctuation in groundwater
  - Types of foundation used in the area
  - Probably thickness of the fill
  - Types of structure and foundation proposed

- Settlement

  - Ultimate settlement
  - Differential settlement
  - Time rate of settlement

A problem of the design of a fill is the settlement. Sands and gravels that contain only a small percentage of fine clay materials are the most suitable for fills used to support buildings.

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- Slope stability
  The design of an earth fill primarily consists of establishing its geometry (a safety factor of 1,5 for slopes no steeper than 1:2). The standard design for earthen fills is three horizontal for each vertical (1:3 slope). As a result, you should plan on needing an area six feet wide (at a minimum) for each foot in height.

- Bearing capacity
  Structures on fill can be constructed and designed, using the standard rules, materials and procedures. A properly constructed fill may often provide a better building foundation than the original material underlying the fill. However, the effect on soil saturation on foundations may still have to be considered. If soil saturation is probable, the foundation support and components of the structure should be designed to withstand all hydrostatic pressures, including uplift forces. This problem would be applicable for fill areas that are highly permeable and subject to extended periods of flooding.

- Drainage and erosion control
  Little maintenance is required for elevated fills. Fills in high stream areas may require some repair to the rip rap embankment protection (vegetation may provide protection for velocities up to 5 ft s). The frequency of repair is a function of the frequency of flooding and the adequacy of the original erosion protection. Some fills may include perforated drainpipe as part of the sub drain system. A well-designed sub drain system needs to be cleaned out every twenty to thirty years.

  Fill placed in a floodplain may, however, cause increased flood heights or velocities. Because of this the potential damage in the area is increased. In particular, fill material cannot be placed within a NFIP designated 'floodway' unless it can be shown that such a placement will not cause a significant increase in flood levels [15].

Structures raised on fill in the Gentilly Destrict In New Orleans, especially in older parts such as Edgewood, where poor/weak soil conditions are found, deep foundations may be needed to provide the required bearing capacity and to limit settlement. Buildings on fill will cause large settlements.

Fill is not recommended for coastal A zones, but may be appropriate for non coastal A zones. Sites can be filled to help elevate structures. The NFIP floodway standard restricts new development from obstructing the flow of water and increasing flood heights. However, this provision does not address the need to maintain flood storage. Especially in the flat areas, the floodplain provides a valuable function by storing floodwaters.

A solution is to require compensatory storage to offset any loss of flood storage capacity. The developer is required to offset new fill put in the floodplain by excavating an additional floodable area to replace the lost flood storage area. This should be done at "hydraulically equivalent" sites [27].

7.3.3 Mechanically elevating structures

The following solution was inspired by a type of structure used at Dutch farms. The 'hooiberg' or hay barrack is a structure that houses varying amounts of hay, grain, flax or straw by utilizing a vertically sliding roof. This solution will explore possibilities of mechanically raising not just a roof, but entire housing unit above flood elevation. A short introduction of the hay barrack is given as an introduction.
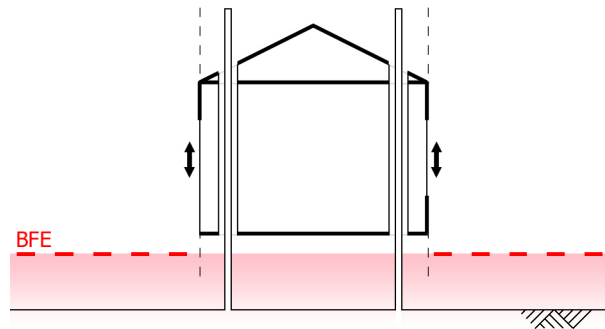
Figure 7.12: Concept of elevating structure

7.3.3.1 Historical background of the hay barrack

Hay barracks have been built as early as the 13th century and continue to be built, though mainly for restoration of historical farms or for recreational purposes as pavilions. In the 17th century the Dutch introduced the hay barrack to the Americas and they became commonplace in Pennsylvania, New York and New Jersey.

Figure 7.13: Hay barrack in use and during construction. Image from: www.hooiberg.info [19].

A typical hay barrack is constructed on brick foundation piers. One to six square or round oak posts act as vertical guides for the sliding thatched roof. Nowadays foundations are usually concrete and the posts are either timber or steel. Throughout history different systems were used to raise and lower the roof. In principle the posts have drilled holes and tapered slots at regular intervals to provide a range of levels to where the roof may be raised. The holes are used for pin-connections between the roof and posts. The rafter of the roof would fit in the slots.

One of the oldest methods was using a 'boom en ketting', literally 'tree and chain', to manually raise or lower the roof by leverage (see Figure 7.15).

Another method would use a winch and rope to apply force through a pole. Teeth on one end of the pole would lock against the edge of the roof.

Later methods include the use of iron or steel elements. One common type uses a screw, called a 'ronsel', to jack up the roof along the posts.
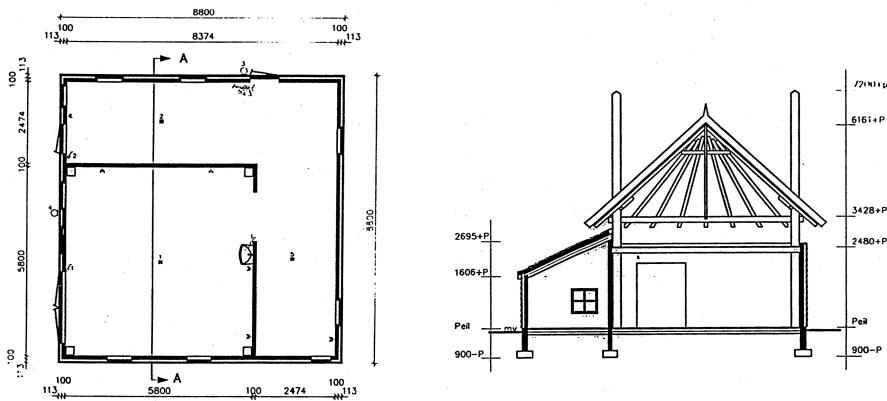
Figure 7.14: Floorplan and elevation of typical hay barrack. Image from: www.hooiberg.info [19].
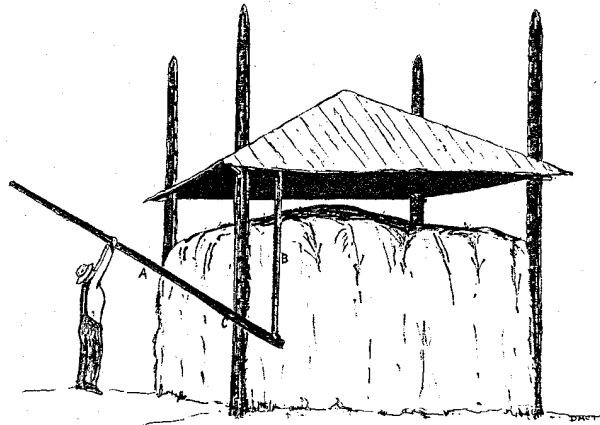
Figure 7.15: Manual leverage provided the system for elevation. Image from: www.hooiberg.info [19].

Modern designs include the use of steel cables and steel or concrete posts to decrease the dimensions or increase the load of the roof system. The preferred system to raise the roof is a winch and pulley, either manually or electrically powered.

7.3.3.2 Design issues of elevating structures

There are various issues that arise when translating an elevating structure to the particular problems that a flood in hurricane conditions poses.

A major difference between this proposal and the hay barrack is the amount of gravity loading. Obviously an entire housing unit including live loads presents a much larger strain to the load bearing posts than a single roof does. Furthermore, gravity loads normally transfer through the framing of the house to the foundation, but in this case a secondary or different type of load-bearing structure is needed when the house is raised.

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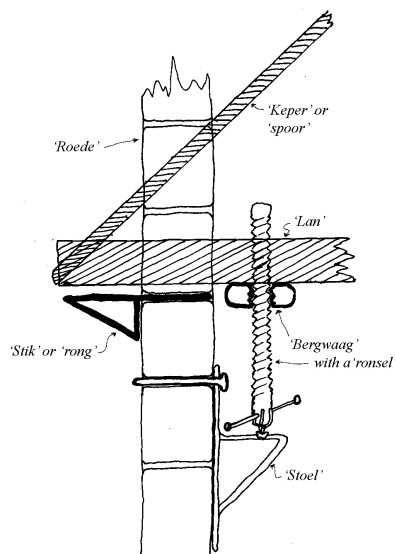
Figure 7.16: A screw or 'ronsel' worked as a conventional jack. Image from: www.hooiberg.info [19].

This will require additional material able to carry gravity loads and resist wind loads. Note that some additional structure might be needed as well if the housing unit hangs from the elevation system and structural members are also subject to tension instead of only compression.

The design of the hay barrack was not intended for issues with flooding or inundation, therefore some new aspects have to be considered. The elevating systems described earlier assume that the system is operated from the ground. However, in case of flooding the system cannot be exposed to water, so the controls should elevate as well instead of being stationary on ground level.
A second consequence of flooding would be the risk of collision between the load bearing posts and floodborne debris. The stationary, exposed parts of the structure should therefore be designed with acceptable risks or some redundancy to allow for some degree of damage.

When raised, the structure is also more susceptible to wind loading and becomes increasingly important as it elevates. The vertical guidance posts don't allow for bracing in a way that a structure on piles would. Resistance to lateral loads can be achieved by moving the housing unit along stable vertical structures instead of a system of single vertical elements. Another option would be to rigidly fix all the connections.

Because a mechanical system and moving parts are inherently subject to mechanical failure, keeping check on overall reliability is important to achieve a feasible design. The reliability can generally be improved by opting for a simple elevating system and implementing redundancy to anticipate possible failures. It would also be recommended to test the system on an annual basis to verify that the system continues to function. Accessibility of the system has to be kept in mind in order for regular maintenance to be possible.

7.3.3.3 Possible systems for elevation

The hay barrack employed several different solutions for elevation that include levers, mechanical jacks and pulley systems. More options can be considered when looking to modern applications elsewhere in other fields of mechanical engineering.

- Pulley and counterweight

- Gears

- Screws and linkages

- Hydraulics or pneumatics

Many elevator systems use steel ropes and a drive sheave as a hoist mechanism. A stationary electric motor powers the drive sheave at the top to move the elevator cabin. The power needed is reduced by a counterweight on the opposite end of the ropes. Usually the counterweight weighs as much as the cabin plus half of its capacity. It should be noted that for this solution a counterweight would at the most weigh as much as the house plus half the design live load.
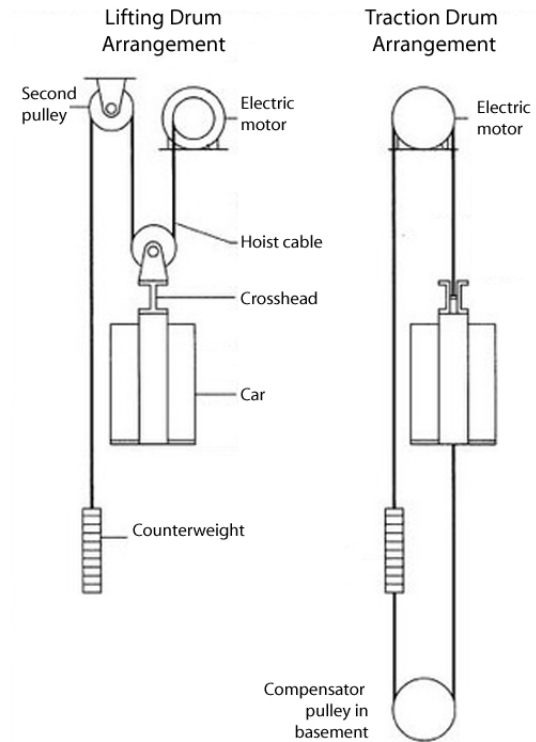
Figure 7.17: Typical elevator systems.

A relatively simple solution is a rack and pinion found in steering mechanisms of cars, sluice gates or material hoists at construction sites. A circular gear (pinion) engages teeth on a flat bar (rack) to convert a rotation to a linear motion, in this case, vertical. The rack and pinion requires a locking mechanism for the elevating structure to remain at a specific height, such as a ratchet and crank or a self locking worm gear. The connection between either the rack and
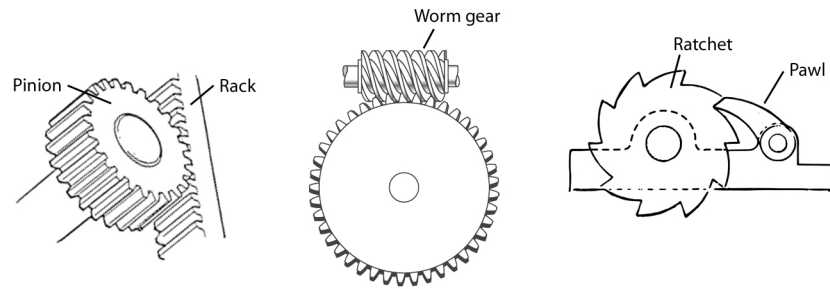
Figure 7.18: Rack and pinion gears, a self-locking worm gear and a ratchet and pawl.

pinion or the locking mechanism should be able to transfer the weight of the housing unit to the load-bearing elements.

A car jack uses a jackscrew to lift a car so its owner can change a wheel. A horizontal central screw thread is rotated so a parallelogram shaped linkage can raise the car. The thread however is not elevated at the same rate as the top of the linkages. Left unsolved, this would be highly impractical for use in this solution.
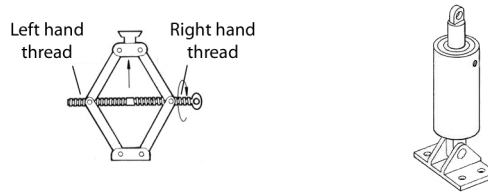
Figure 7.19: Left: A carjack with jackscrew and linkage. Right: A standard hydraulic or pneumatic cylinder.

Pneumatic and hydraulic cylinders are often applied in heavy equipment. Gases and fluids are pressurized to allow machinery to transfer large amounts of energy through relatively small areas. By using telescoping cylinders the system can extend over a large distance. In general pneumatic cylinders are less expensive than hydraulic cylinders of the same size. Besides cylinders, pneumatic power can be enabled through air springs, which can inflate to elevate a structure.

Though the roof of the hay barrack was raised using manpower, in this case a motor would be preferable for general comfort and especially for the high loads involved. The housing unit doesn't need to be lifted at a high speed, merely fast enough to do so in time and to evacuate after a hurricane warning has been issued. Also a high torque has to be generated to apply enough force to lift the house. A transmission is therefore required to reduce the speed and increase torque. Because the gear ratio can be constant, the transmission design can be fairly simple. If however the system retains the option of manual operation, it has to shift gears and will need a gear box.

7.3.3.4 Choice of elevation system

The different options can be divided in two groups; systems with a counterweight and systems with a self locking mechanism. For calculation, the total load of a building is assumed to be 5,86 kN/m^2 * 125 m^2 = 732 kN.

A counterweight is normally made of concrete or lead. If lead were used as a counterweight for the housing unit, the required mass and more importantly, the volume would still be substantial. The weight of lead is 113,4 kN/m^3. The counterweight(s) has to weigh as much as the dead load plus half the live load, or (3,94 kN/m^2 + 0,50 * 1,92 kN/m^2) * 125 m^2 = 613 kN. It would have to be larger than a total of 5,4 m^3 of lead. To reduce the volume needed a block and tackle pulley system could be introduced. A block and tackle contains additional pulleys to reduce the required force applied when raising the house. Note that a counterweight now needs to travel a relatively larger distance to move the housing unit. This would mean that when the structure is raised, a counterweight has to hang below grade in some type of basement level or ground shaft. Another problem is that such a counterweight should not come into contact with the floodwater as the hydrostatic pressure would reduce its ability to act as a counterweight.
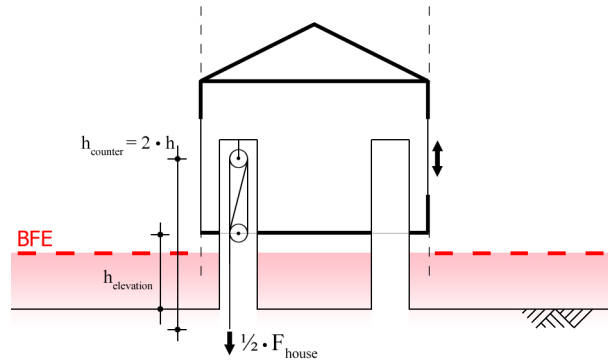
Figure 7.20: Mechanically elevating structure using a counterweight

A self-locking mechanism doesn't need any components below grade. Also while the motor powering a pulley system is stationary at the top, a self-locking system will elevate as it moves the building along a guidance structure making it more accessible to users. One drawback is the relative complexity of self-locking systems such as worm gears. It remains difficult to say if maintenance is needed more often compared to a counterweight system.

The motors driving both systems differ in their power output. The self-locking elevation system needs to apply loads higher than 732 kN to raise the house. A counterweight will almost double the total gravity load, double the required strengths of the loadbearing structure but reduce the amount of power needed to raise the house. As stated earlier, the counterweight would have to be 613 kN. The motor would now only have to apply loads up to the difference, or 119 kN.

The fact that a counterweight needs to move below grade is a big disadvantage. In the final design stage a self-locking mechanism would be the best option for elevating structures.

Advantages
More loads on stabilizing structure; more stability
Lower power output needed to elevate
Relatively simple system
Disadvantages
Additional material for counterweight
More material for loadbearing structure
Space below grade for a counterweight system
Stationary motor powers the top pulley

Table 7.1: (Dis-)advantages of both types of the counterweight system

Advantages
Less material needed
Motor elevates with building
Disadvantages
Relatively complex machinery = less reliability, higher cost
Higher power output needed to elevate

Table 7.2: (Dis-)advantages of self-locking mechanisms

7.3.4 Structures with watertight ground floor

In Louisiana a few examples of watertight buildings (dry flood proofing) can be found. Dry flood proofing is only appropriate for non-residential structures on a concrete floor slab, without basement and with no cracks. As a rule the maximum flood protection level is two feet above the ground slab, since most walls and floors in buildings will collapse under higher water levels (see Section 7.2.1).

This Section describes an alternative solution in the form of watertight residential buildings; structures with dry flood proofed ground levels with entrances at second level. Collapse from hydrostatic pressure is a major concern with this solution. Other problems are keeping the water out, subsequent detailing of water tightened connections, and preventing the building from shifting or floating away when submersed in floodwaters.
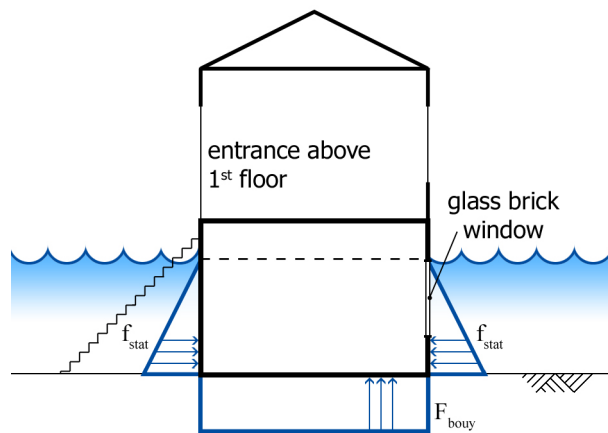
Figure 7.21: Building with a watertight groundfloor

Structure The building (ground level) must be constructed with concrete block or brick veneer on a timber frame. Weaker construction materials, such as a timber frame, will fail at much lower water depths from hydrostatic forces.

Most wall materials, except for some types of high-quality concrete, will leak (seepage to cracks in the walls) unless special construction techniques are used. The most effective method of sealing a brick faced wall would be to install a watertight seal behind the brick when the building is constructed. For flood proofing existing structures, the best way to seal a wall is to add an additional layer of brick with a seal "sandwiched" between the two layers. It is possible to apply a sealant to the outside of a brick or block wall with a waterproofing compound, but any coating must be applied carefully. Cement or asphalt based coatings are the most effective materials for sealing a brick wall, while clear coatings such as epoxies and polyurethanes tend to be less effective. But when using a better sealant coating, the aesthetic advantages of a brick wall are gone. All structural joints, such as those where the walls meet foundations or slabs, require treatment.

Facilities The building's utilities and sanitary facilities, including heating, air conditioning, electrical, water supply, and sanitary sewage services, must be completely enclosed within the building's watertight walls, or made watertight and capable of resisting damage during flood

conditions. As a precaution the majority of services could be mounted or installed above expected flood elevations.

Windows/openings   The windows on the ground level must be watertight and have to resist flood related forces and wind forces. Standard plate glass cannot withstand flood induced forces, so a form of protection is needed. A solution is to use glass block, which allows natural lighting. Replacing the glass with heavy plexi-glass is another solution.

The structure must be very rigid to prevent displacements along the edges of the glass. Because of the enormous forces, the glass should be placed at the outside of the houses.

Forces   The maximum flood level above the ground slab is 2 m (6,56 ft). (See Section B.4). All of the building's structural components must be capable of resisting flood-related forces. These are the forces that would be exerted upon the building as a result of a floodwater of 2 m above the ground level. For a flood proofed building design, the calculations of hydrostatic flood forces must include saturated soil pressure on any portion of the building that is below grade (see Figure 7.21and Section D.2).

7.4 Multi-Criteria Analysis of alternative solutions

The alternative designs must conform to certain criteria. There is no alternative design solution that is optimal for each of the criteria concerned. A Multi-Criteria Analysis (MCA) is an approach for choosing from a set of alternatives when there are multiple criteria. Using a multi-criteria analysis the best compromise with respect to the different criteria can be found.

This MCA must facilitate the selection between the four alternatives, or at least get a well-structured overview of all criteria of importance to the final choice. This also gives an insight into the degree in which the various criteria contribute to the choice (see Appendix E).

7.4.1 Identifying the alternatives

The four alternative solutions are:

- Watertight structures
- Floating or amphibious structures
- Structures raised on fill
- Elevating structures

Also the existing alternative, the pile house, is used as a reference so that the alternatives can be evaluated as marginal or incremental to this existing type of building.

7.4.2 Identifying the criteria

The criteria are developed by dividing the overall goal into several sub-criteria. These sub-criteria are divided into a hierarchy of criteria through further decomposition. This final list of criteria is consistent and without overlapping each other.

The criteria include a range of different perspectives of the designs, including the following:

- Surroundings
- Safety
- User comfort
- Technical impact
- Construction

7.4.3 Scoring the Alternatives in Relation to the Criteria

The scores are presented in a matrix, displaying the criteria in the rows and the alternatives in the columns. The scores range from 6 - 10, where the most favored score is 10 and the least 6. A score is provided for each alternative against each criterion in relation to the existing alternative.

7.4.4 Weighting the scores according to the weights assigned to the criteria

The total score of 100% is divided between the five main criteria, in line with their perceived weighting relative to each other. The scoring process results in a relative scale for each criterion.

7.4.5 Evaluating and ranking of the alternatives

Table 7.3 shows the outcome of the multi-criteria analyses.

Floating structures	8.48
Raised on fill	8.25
Elevating structures	8.14
Pile house	8.12
Watertight	7.41

Table 7.3: Ranking of the alternatives

7.4.6 Recommendations

The scores of each alternative can be compared with the existing pile house as a reference. Other than the watertight structure, all new proposals could potentially offer a better solution for designing homes in a floodprone area. The cost of these alternative solutions will ultimately determine their feasibility. A comparative cost analysis will be made after the next and final design stage.

Structures raised on fill, floating and elevating structures all merit further research and design, but the first solution poses a problem. The use of fill in A zones in Louisiana is, according to the NFIP regulations only permitted when the cumulative effect of the fill, when combined with other existing and anticipated development, will not increase the water surface elevation of the base flood more than one foot at any point. For this reason, the option of structures raised on fill will no longer be explored.

The remaining floating and elevating structures score better than pile houses. Furthermore, both offer possibilities not only for new buildings, but for retrofitting existing buildings as well. Currently there is great demand for building new homes and for salvaging existing ones. In the final design stage both solutions will be examined further.

Acronyms and Abbreviations   A

ABFE   Advisory Base Flood Elevations

BFE   Base Flood Elevation

cfs   cubic feet per second

FEMA   Federal Emergency Management Agency

FIRM   Flood Insurance Rate Map

GNOCDC   Greater New Orleans Community Data Center

IPCC   Intergovernental Panel on Climate Change

LSU   Louisiana State University

mph   miles per hour

MSL   Mean Sea Level

NEN   Nederlandse Norm

NFIP   National Flood Insurance Program

TAW   Technische Aanbevelingen Waterkeringen

TGB   Technische Grondslagen voor Bouwconstructies

UNEP   United Nations Environment Program

USACE   U.S. Army Corps of Engineers

B

Analysis of boundary conditions

B.1 Extreme conditions induced by hurricanes

The Saffir-Simpson Hurricane Scale gives a one to five rating based on the hurricane's intensity. Wind speed is the factor used to determine the scale, as storm surge water levels are highly dependent on the near shore slope and shape of the coastline. Nevertheless the values presented in Table B.1 give a good indication of the water level elevations that can be expected during hurricanes.

Hurricane category	Wind speeds (km/hr)	Storm surge water level elevation (m above normal)	Return period for the Louisiana coast (yrs)
Category 1	119-153	1.22-1.52	8
Category 2	154-177	1.83-2.44	18
Category 3	178-209	2.74-3.66	31
Category 4	210-249	3.96-5.49	65
Category 5	>249	>5.49	170

Table B.1: The Saffir - Simpson Hurricane Scale (National Hurricane Center) [6].

The National Hurricane Research Center Risk Analysis Program (HURISK) gives return periods on hurricane categories along different parts of the US coast. The return periods for the Louisiana coast are also given in Table B.1. If a life span of 100 years is chosen for the flood protection structure, is not unlikely that a hurricane of category 5 will occur. If the Poisson distribution is assumed the chance that a category 5 hurricane will pass the area at least once in 50 years is 25%, for 100 years this chance is as large as 46%. Therefore this hurricane category will be used to come up with a design.

The table shows that a category 5 hurricane causes winds with great speeds. In normal storm wind also causes surges, but in this case the surge will be negligible. This is because the surge needs time to develop, and a hurricane is always on the move. The surge will not be able to grow in time. Also, the wind needs to blow unidirectional for a considerable fetch to cause a surge. With a hurricane this will not be the case.

B.2 Tides

The Gulf of Mexico has a M1 tide, meaning that there's only one high and one low water a day. The tidal behavior of Lake Pontchartrain follows that of the Gulf of Mexico. Figure B.1 gives an overview of the tidal values, measured at the Industrial Canal, east of the London Avenue Outfall Canal. Figure B.1 shows the gage height, which is the height of the water surface above the gage datum (zero point). This shows that the highest tidal water levels that can be expected lie around 40 cm above Mean Sea Level (MSL is about 1.0 foot gage height).
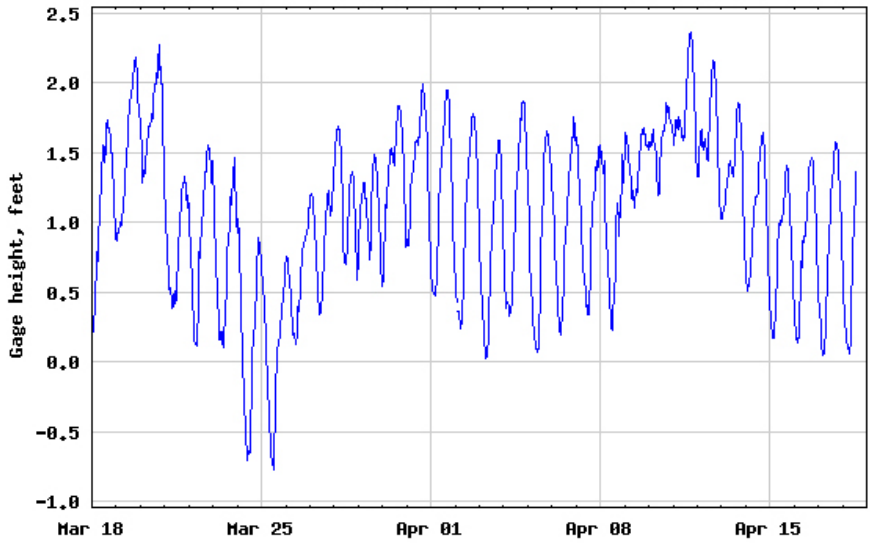
Figure B.1: Tidal values during one month (2006) at USGS 0738023321 Industrial Canal at I-10 at New Orleans, LA. Image from: USGS 2006 [38].

B.3   Sea level rise

The USACE has monitored tide gages in Louisiana since 1933. Daily water levels were averaged in annual tables, which when plotted against time show a slowly rising Mean Sea Level (Figure B.2). According to this data the south point of Lake Pontchartrain the sea level has risen approximately 1.0 cm/year during the period of 1949-1984.
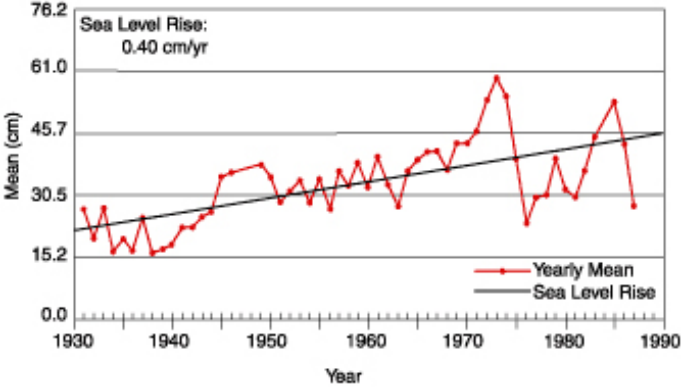
Figure B.2: Mean annual water level measurements for Lake Pontchartrain at West End (1931-1987) including rate of sea-level change for the area. Image from: USGS 2006 [37].

Over the last couple of years there has been an ongoing global debate on the magnitude of the sea level rise. This debate can be summarized in Figure B.3, showing the high trend line as the pessimistic and the low trend line as the optimistic scenario. A best estimate scenario is given in the figure, giving a global sea level rise of 66 cm in the next century. This number reasonably agrees with the trend line in Figure B.3. Therefore the best estimate scenario is chosen for the

design.
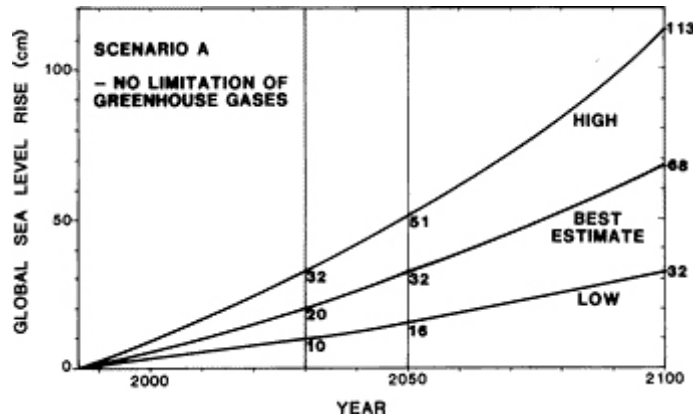
Figure B.3: Sea level change scenarios for the next century. Image from: UNEP/IPCC 1990.

B.4 Flood elevations

In Figure B.4 the flood elevation of the area that will be investigated is given. The range of the ground elevation is between 4.0 m below and 2 m above MSL. The dark blue zones indicate the lowest areas.

Besides the adopted statewide codes, new buildings will likely have to follow the recent FEMA recommendations to qualify for flood insurance with reasonable premiums.

New construction and substantially damaged residential and commercial buildings within a designated FEMA floodplain should be elevated to either the Base Flood Elevation (BFE) shown on the current effective Flood Insurance Rate Map (FIRM) or at least 0,9m (3 feet) above the highest adjacent existing ground elevation; whichever is higher. 'Substantial damage' is defined as repairs costing more than 50 percent of the cost to completely rebuild the house.

FIRM's use several letters as flood zone designations to denote risk of flooding and to determine flood insurance premiums accordingly. As shown on the map, the greater part of Gentilly consists of A zones, varying from 0 to 7, while the area along Gentilly Blvd. is a B zone. The numbers are the minimum elevation allowed for the lowest floor.

Homes in the A zones are at risk of flooding and subject to 100-year flood. These zones range from A1 to A30 depending on drainage and topography, where a higher number indicates a higher risk of flooding. At lakeside the risk is relatively low and an A0 zone is only subject to shallow flooding once every 100 years, with shallow being 0,9m (3 feet) of water or less. The B zone is high ground and will flood less than once every 100 years. Flood insurance in not required in a B zone.

The cross sections (see Figure B.6 and B.7) show how the BFE and 9 ft elevation rule combine and will affect future building projects. The lowest floor of a new building has to be above both lines. Prentiss Avenue cuts along some of the lowest parts of Gentilly District. The cross section shows that there is a sustained difference between the ground elevation and the BFE of 2 m. New designs for housing structures in the Gentilly District will therefore assume a required design elevation of 2 m above grade.
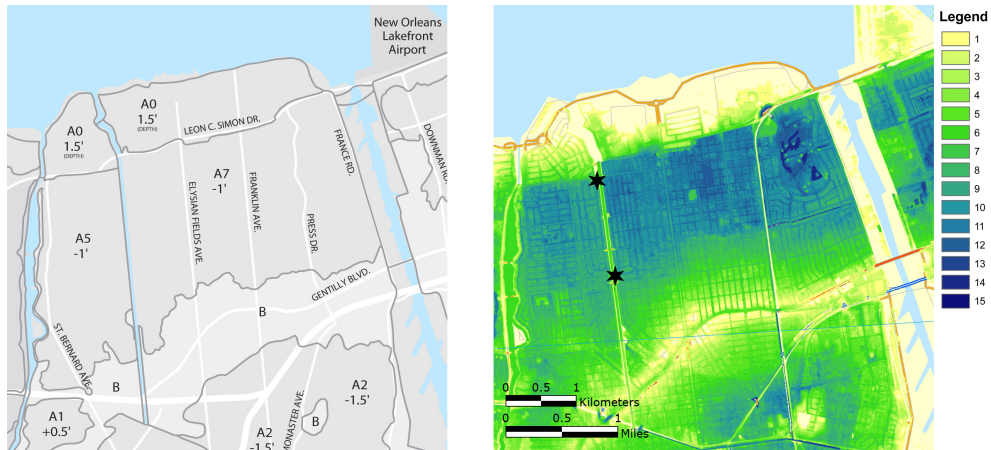
Figure B.4: Current 1984 FIRM for the Gentilly District. Image from: www.Nola.com [23]. GIS Katrina flood depths (ft). Data by Braud and Cunningham [31].

Figure B.5: Two cross sections of Gentilly District. Image from: Google Earth [11].
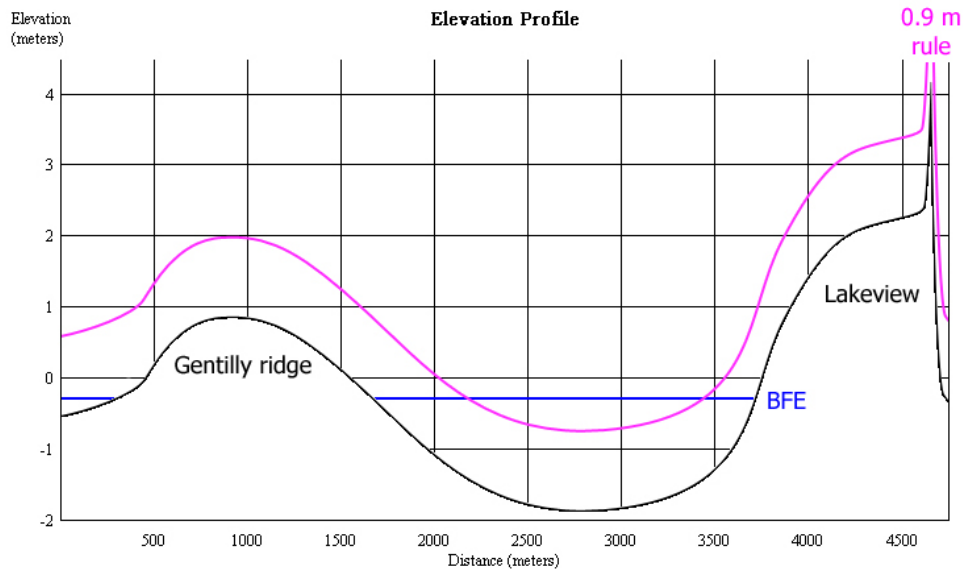
Figure B.6: North - South cross section along Elysian Fields Avenue. Data by: USGS [39].
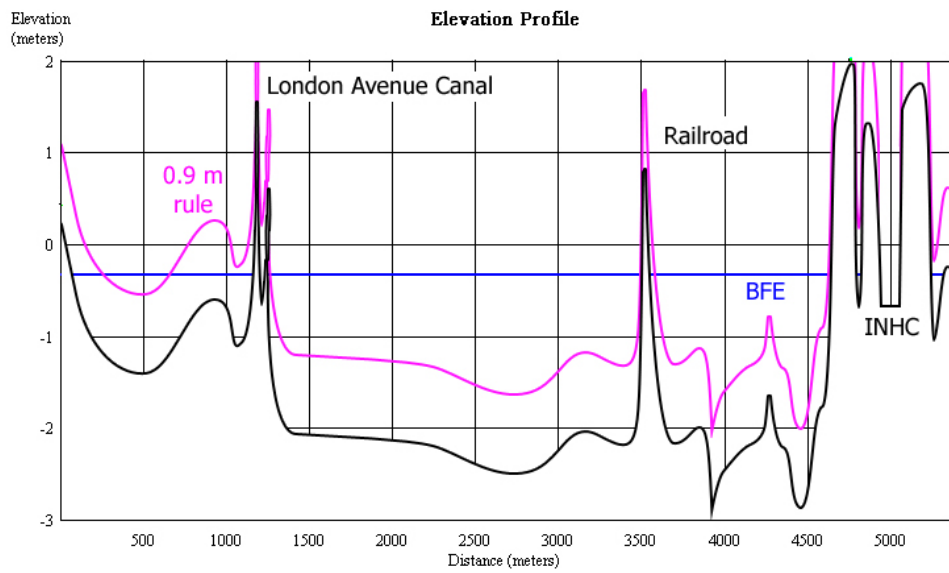
Figure B.7: West - East cross section along Prentiss Avenue. Data by: USGS [39].

B.5 Levee heights

Figure B.8 shows the levee heights alongside Lake Pontchartrain and the canals. The peripheral levees are 5.33m (17.5 ft) above NGVD, the levee heights alongside the canal vary from 4.11m (13.5 ft) till 4.24m (13.9 ft). This corresponds with respectively 5.60 m, 4.38 m and 4.51 m above MSL.
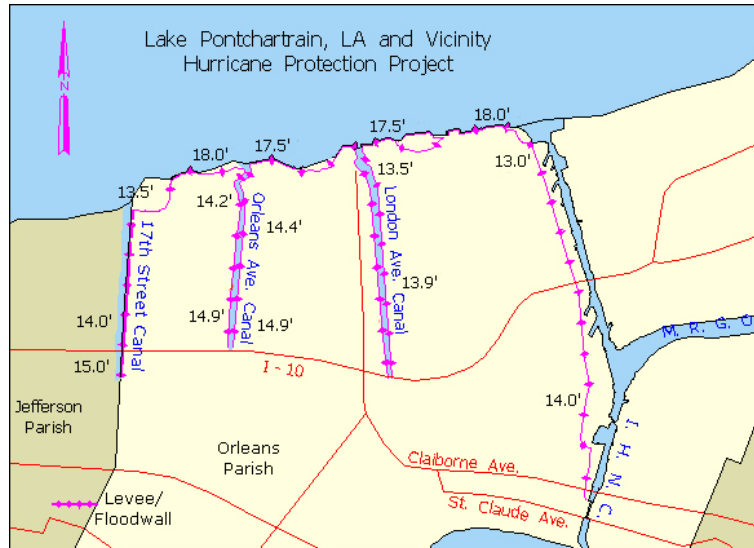
Figure B.8: Levee heights.

B.6 Soil properties

New Orleans is situated in the Mississippi River delta, which implies that the soil will contain clay and peat. This is shown in Figure B.9. Although this is not a very detailed picture, one can say that the solid and firm sand layer of the First Pleistoceen Formation doesnt really start until a depth of app. 25m (82 ft). Above that there is a layer of silt that reaches up to a level of -10m (33 ft). On top of that layer there is either a layer of sand (denoted as Barrier Island) or silt (denoted as Pro-Delta). There is obviously a big difference between these two types of soil when it comes to the properties. The last layer lies on top of the rest, it is made of organic and high water content clays. This is app. 2 m thick.

Representative values of the properties for these different soil types are needed. In Table B.2 an extraction from the NEN 6470 is made, containing the most applicable properties (TGB 1990 Geotechniek NEN 6740). The given values are indicative and used in The Netherlands; they will give a good estimate of what to expect. For more accurate values soil test have to be made.
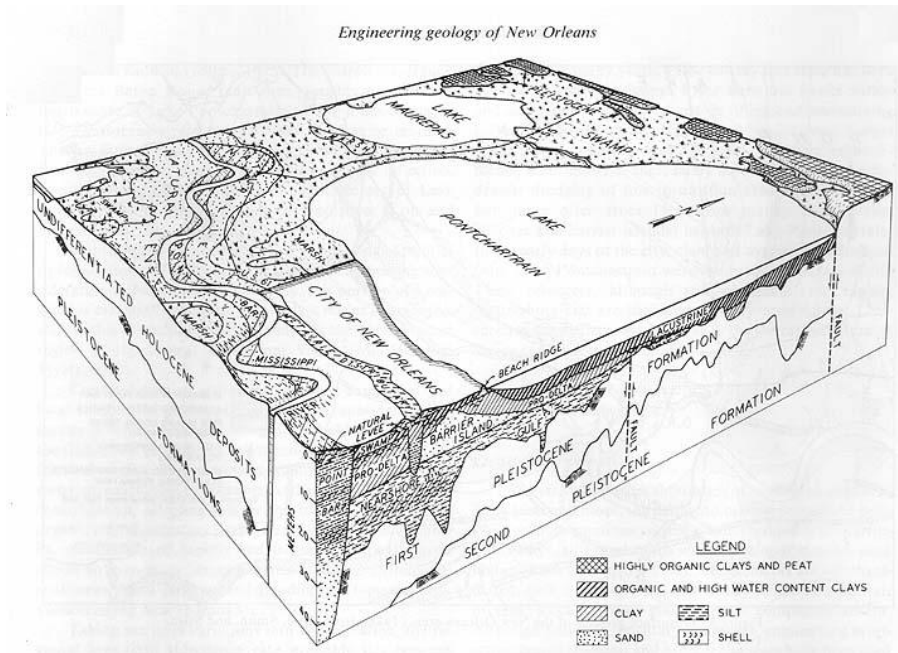
Figure B.9: Idealized distribution of depositional environment and soil types in the vicinity of New Orleans. Image from: Kolb and Shockley 1959 [22].

	Sand	Clay and peat	Silt
Density ρ [$kg/m3$]	2700	1300	3000
Cohesion c [$N/m2$]	0	25-50	2-5
Angle of internal friction ϕ [-]	30-45	15-30	27.5-32.5
Permeability k [m/s]	10^{-6} - 10^{-3}	10^{-10} - 10^{-8}	10^{-8} - 10^{-6}
Coefficient of compression C [-]	50-500	2-100	25-125
Coefficient of compression C_{10} [-]	20-200	1-40	10-50

Table B.2: Indicative soil properties according TGB 1990 Grondtechniek NEN 6740

B.7 Roadmap

A part of the studies will be about the water management. In the present situation the area of interested acts as one single basin, with Lake Pontchartrain in the north, 17th Street Outfall Canal in the west, Mississippi River in the South and the INHC in the east as its boundaries. An option would be to compartmentalize the area, with the result that only parts would flood, and not the whole area. In Figure B.10 the roadmap is given.
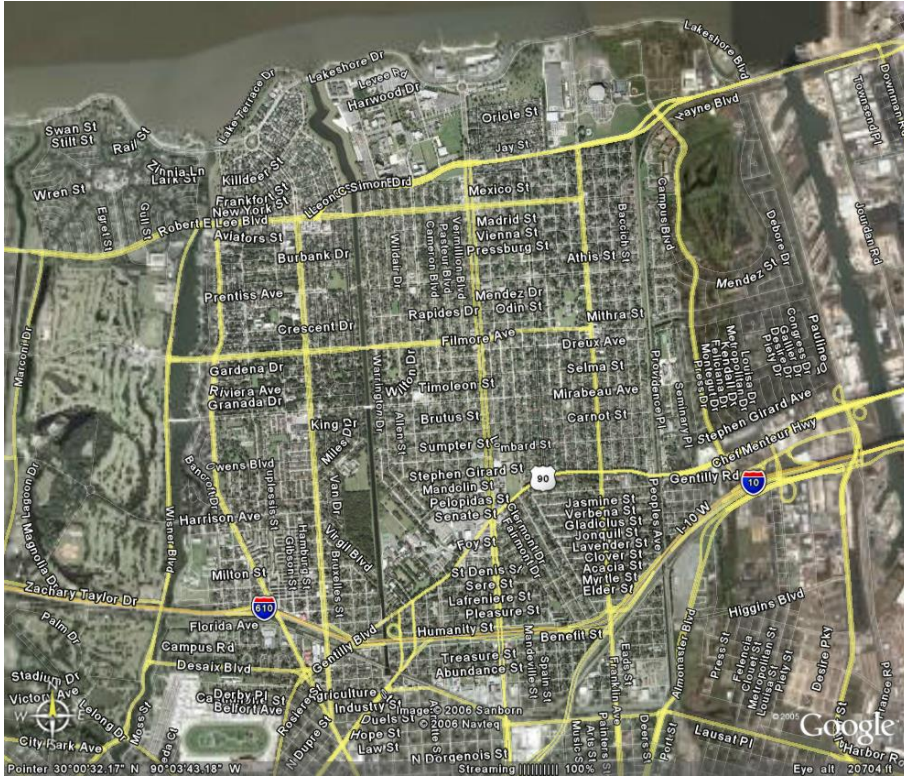
Figure B.10: Roadmap of the area. Image from: Google Earth [11].

B.8   Water management system

The New Orleans levee system not only keeps water from Lake Pontchartrain and the Mississippi river out of the low lying areas of the city, but also tend to keep rainfall and floodwater in. To remove this excessive water, the construction of an extensive drainage system started in the 1830s. This system contains drainage pump stations, which pump water out of the city into the outfall canals, which transport the water into Lake Pontchartrain.

London Avenue Outfall Canal is located in the Orleans East Bank, which has 12 pump stations containing 67 pumps with a total capacity of 35,739 Cubic Feet per Second (CFS), which is 1,012 m^3/s. The London Avenue Canal has 2 pumping stations (see Figure B.11), number of pumps and capacities are given in Table B.3. Pump station A is situated at the most southern point of the canal, pump station B is situated at the east side of the canal.
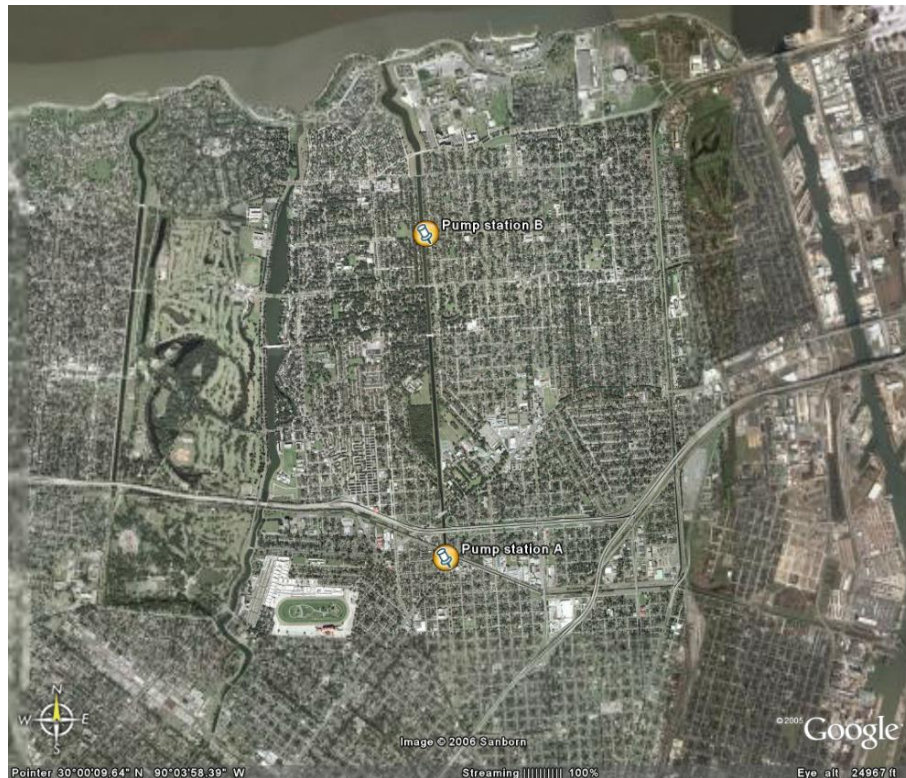
Figure B.11: Location of the London Avenue Outfall Canal pump stations. Image from: Google Earth [11].

Pump station	Number of pumps	Total capacity (CFS)	Total capacity (m^3/s)	Primary power
A	7	4140	117.2	Electricity
B	6	3680	104.2	Electricity

Table B.3: Pumping capacities of the London Avenue Outfall Canal pump stations, September 2005. Source USACE [35].

During Katrina the pump stations in London Avenue Canal were out of order, due to the

breaches in the canal. The pump stations could not operate until the breaches were fixed.

Determination of the significant wave height H_S and period T_p

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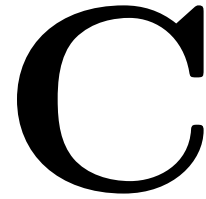
Short waves (the wavelength L_0 is less than half the water depth) are caused by wind. During storms wind is blowing over the water surface and pressure differences cause waves to develop. The significant wave height is the average wave height of the 1/3 largest waves of a wave record. The significant wave period is the average wave period of the 1/3 waves with the longest period. This significant wave height and period are dependent on a number of factors:

- Wind fetch (F). The wind needs time (and therefore distance) to get the waves fully developed. Once the waves are fully developed, the fetch is no longer important. The maximum fetch in Lake Pontchartrain is app. 50 km $= 50 * 10^3$ m.

- The storm duration (t_d). The longer the storm endures, the higher the waves will get. A typical t_d for a hurricane event is 6 hours $= 21600$ sec.

- The wind velocity (U_{10}). The higher the wind velocities are, the higher the waves get. The velocities measured are those that occur at an elevation of 10 m above the sea level. In Lake Pontchartrain the strongest expectable hurricane is one of category 3. U_{10} for a category 3 hurricane is assumed to be 210 km/hr $= 58.3$ m/s.

The theoretical H_s and T_p can now be computed. In this report the JONSWAP (Joint North Sea WAve Project) method is used [18]. The following steps need to be taken:

- The calculation of F^* $\quad$ ($F^* = \frac{gF}{U_{10}^2} = 144.3$)

- The calculation of t^* $\quad$ ($t^* = \frac{gt}{U_{10}} = 3635$)

- The calculation of F_{eff}^* $\quad$ ($F_{eff}^* = (\frac{t^*}{68.8})^{1.5} = 295.6$)

F^* and F_{eff}^* are used to determine if the wave is fetch limited or if it duration limited. It is fetch limited if $F^* < F_{eff}^*$; if $F^* > F_{eff}^*$ then the wave is duration limited. In this case the wave is clearly fetch limited, a smaller value for F^{**} (smaller than F^*) has to be chosen. Instead of $F^{**} = 144.3$ the new value will be $F^{**} = 80$. Now the final steps can be taken:

- The calculation of H_s
$\frac{gH_s}{U_{10}^2} = 0.0016(F^{**})^{0.5}$
$H_s = 4.96$ m

- The calculation of T_p
$\frac{gT_p}{U_{10}} = 0.286(F^{**})^{1/3}$
$T_p = 7.32$ s

The JONSWAP method unfortunately doesn't take the interaction with the bottom into account. Lake Pontchartrain is a shallow lake, with an average depth of about 5 m (compared to MSL). If a category 5 hurricane makes landfall it will lose strength due to the increased friction (induced by the land). It will reach Lake Pontchartrain as a category 3 hurricane at

the most. In combination with sea level rise, the hurricane induced surge and the tidal eleva-
tion this average depth can become 9.7 m. Still, this a shallow water condition ($\frac{d}{L_0} = \frac{d}{\frac{gT_p^2}{2\pi}} = 0.1$).

In Figure C.1 the relation of the dimensionless wave height versus the relative depth is
given. In general this Figure shows that the relation between H_s and H_{s0} is app. 0.9. This
means for this case the H_s will be 4.46 m.
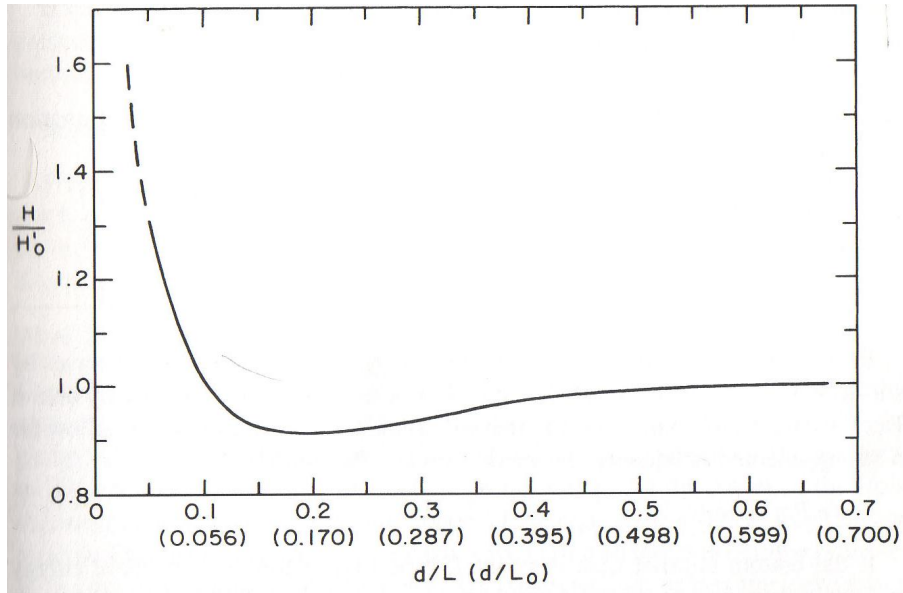
Figure C.1: Dimensionless wave height versus relative depth. Image from: Sorenson 1997[29]

D Loads

D.1 Building codes

The primary building codes for residential and commercial buildings in the United States are developed and maintained by the International Code Council (ICC). Despite the name, the ICC actually only operates on a national level and is dedicated to 'developing a single set of comprehensive and coordinated national model construction codes'. The ICC codes, or I-Codes, include the International Building Code (IBC), the International Residential Code (IRC), the International Existing Building Code (IEBC) and series of other codes concerning various aspects of building engineering.

Previously any statewide building codes in Louisiana applied to state-owned buildings only. Prior to hurricane Katrina each parish had its own building code which could lead to differences between neighboring parishes even though minimum national standards had to be adhered to. Since a new bill was signed in late November of 2005, the state of Louisiana adopts several I-Codes as a statewide standard for all new buildings and rebuilding efforts.

Some building codes, relevant to this design project, are given in Table D.1:

IRC-2003	ICC	International Residential Code for One- and Two- Family Dwellings
ASCE 7-02	American Society	Minimum Design Loads for Building and Other Structures
ASCE 24-05	American Society of Civil Engineers	Flood Resistant Design and Construction
FEMA 55	Federal Emergency Management Agency	Coastal Construction Manual
ASCE 5-02 ACI 530-02 TMS 402-02	American Society of Civil Engineers American Concrete Institute The Masonry Society	Building Code Requirements for Masonry Structures
ACI 318-02	American Concrete Institute	Building Code Requirements for Structural Concrete
ANSI/AFPA NDS-2001	American Forest and Paper Associates American Wood Council	National Design Specifications for Wood Construction

Table D.1: Relevant building codes [16]

D.2 Principal loads on building structures

In a previous chapter the types of damage that occurred during and after Katrina were discussed. A brief overview of the principal loads on building structures is given to provide additional information when considering new solutions. These loads also give an insight into American design loads for hurricane prone areas for those who are not familiar with them.

D.2.1 Live loads

Live loads are temporary loads applied to a building, which can change in magnitude. They are caused by occupancy (and not the construction) of the building. The floors, roofs and other surfaces must safely support load combinations of the uniformly distributed live loads.

The live loads used in the design calculations for the houses shall be the maximum loads expected by the intended use or occupancy, but shall in no case be less than the minimum uniformly distributed unit load described by ASCE 7-05. For residential dwellings (one and two family), the minimum uniformly distributed live load L_0 is $1,92kN/m^2$ (10 psf).

D.2.2 Dead loads

Dead loads are permanent and stationary loads. The weight of all the construction materials (including walls, roofs, floors and cladding) is the dead load. In estimating the dead load for a design, the actual weight of materials should be used.

D.2.3 Environmental loads

To properly begin analyzing existing solutions and new possibilities for coping with flood-prone areas, a summary is given of the three main loads associated with hurricanes. Since no Dutch codes exist for calculating hydrodynamic or hydrostatic forces (other than hydrostatic pressures from groundwater) some calculation methods from the ASCE and FEMA are shown in Appendix D.

D.2.3.1 Hydrostatic forces

During a hurricane, wind and storm surges will result in high velocity flow and waves; hydrodynamic forces (see Equation D.3 and D.4). Other than direct loading, moving water can also carry debris or cause erosion and scour. Flood borne debris can damage buildings on impact but can also, when lodged in the structure itself, transfer additional hydrodynamic loads to the building structure (see Equation D.5).

Vertical (buoyancy) hydrostatic force is calculated by

$$F_{Buoy} = y(Vol) \tag{D.1}$$

Where

F_{Buoy} = Vertical hydrostatic force (lb) resulting from the displacement of a given volume of floodwater

Vol = Volume of floodwater displaced by a submerged object (ft^3)

Lateral hydrostatic load is calculated by:

$$f_{stat} = \frac{1}{2}\gamma d_s^2 \tag{D.2}$$

Where

f_{stat} = Hydrostatic force per unit width (lb/ft) resulting from flooding against vertical element

γ = Specific weight of water (lb/ft^3), 62.4 for freshwaters, 64 for saltwaters

Inundation occurs when an area below the BFE floods. Long-term effects of flooding are rot and fungal growth. Initially though, flooding will cause buoyancy and might cause floatation of a building (see Equation D.1). The rate at which the water rises combined with the amount of seepage into the building will determine how the water levels on opposite sides of a structural component differ. The difference in water levels result in hydrostatic pressures. The buoyant forces will have to be counteracted by the mass of the building, the strength of its structural connections and the capacity of tensile forces of the foundation piles. Flood waters will also exert lateral hydrostatic forces, but will only cause significant deflections or displacements of buildings or components in case of substantial differences in water levels (see Equation D.2).

D.2.3.2 Hydrodynamic forces

The following equation from FEMA 55 can be used to determine hydrodynamic load from flows moving at less than 10ft/sec:

$$F_{dyn} = 1/2 C d \rho V^2 A \qquad (D.3)$$

Where

F_{dyn}	=	Horizontal drag force (lb) acting at the Stillwater mid-depth (half-way between the still water elevation and the eroded ground surface)
Cd	=	Drag coefficient (2,0 for square or rectangular piles and 1.2 for round piles)
ρ	=	Mass density of fluid (1000 kg/m^3 (1.94 $slugs/ft^3$) for freshwaters and 1025 kg/m^3 (1.99 $slugs/f$
V	=	Velocity of water (ft/sec)
A	=	Surface area of obstruction normal to flow (ft^2)

When the flow velocities do not exceed 10 ft/sec, ASCE 7-02 permits converting the hydrodynamic loads to an equivalent hydrostatic force by calculating an equivalent surcharge depth (dh)

$$dh = \frac{aV^2}{2g} \qquad (D.4)$$

Where

A	=	Coefficient of drag or shape factor (>1.25)
V	=	Average velocity of water (ft/sec)
G	=	Acceleration due to gravity ($32 ft/sec^2$)

The following equation to calculate the magnitude of impact load is provided in the Commentary of ASCE 7-02:

$$F_i = \frac{\pi W V C_i C_o C_D C_B R_{max}}{2g\Delta t} \qquad (D.5)$$

Where

F_i	=	Impact force acting at the still water level (lb)
W	=	Weight of debris (lb), suggest using 1,000 if no site specific information is available
V	=	Velocity of object (assume equal to velocity of water) (ft/sec)
C_i	=	Importance coefficient
C_o	=	Orientation coefficient = 0,8
C_D	=	Depth coefficient
C_B	=	Blockage coefficient
R_{max}	=	Maximum response ratio for impulsive load
g	=	Gravitational constant ($32,2ft/sec^2$)
Δt	=	Duration of impact

Additionally FEMA 55 [16] provides formulas for calculating localized scour.

D.2.3.3 Wind forces

Besides the effects from water, the winds speeds associated with hurricanes can pose problems for structures and will determine many aspects of design and construction. Connections especially have to take wind forces into account.

A hurricane is a tropical cyclone with winds that have reached a constant speed of at least 33 m/s (74 mph). Hurricane wind speeds are very strong and will cause significant pressure and suction on building surfaces. The possibility of overpressure also poses the risk of losing entire sections of the building, such as the roof or gable end.

Building structures in New Orleans are designed for a Category 3 hurricane. The design wind speed is 58 m/s (130 mph) for a 3-second gust at 10 m. Katrina had 1-minute sustained winds of 57 m/s (127 mph) upon landfall making her a Category 3 at that particular time.

In the Netherlands wind loads are calculated with NEN 6702 (see Equation D.6). For example the pressure in urban areas near the west coast (e.g. Rotterdam) is 590 N/m^2 at 10 m above grade. The design wind speed is 31 m/s or merely 53% of the 3-second design wind speed in New Orleans or the sustained wind speed during Katrina. This is not surprising as the Netherlands only deal with traditional storms and aren't subject to hurricanes and hurricane force winds.

In the Netherlands wind loads are calculated with NEN 6702 and wind speeds are calculated with:

$$v_w = \sqrt{1.6 * p_w} \tag{D.6}$$

Where

v_w	=	wind velocity (m/s)
p_w	=	wind pressure (N/m^2) according to table A1

The Saffir Simpson Scale categorizes hurricanes according to their relative strength and minimum central pressure. Hurricanes of category 3 or higher are consider major hurricanes.

Strenght	1-Minute Sustained Wind Speed (mph)	3-Second Gust Wind Speed (mph)	Pressure (millibars)
Category 1	74-95	89-116	>980
Category 2	96-110	117-134	965-979
Category 3	111-130	135-159	945-964
Category 4	131-155	160-189	920-944
Category 5	>155	>189	<920
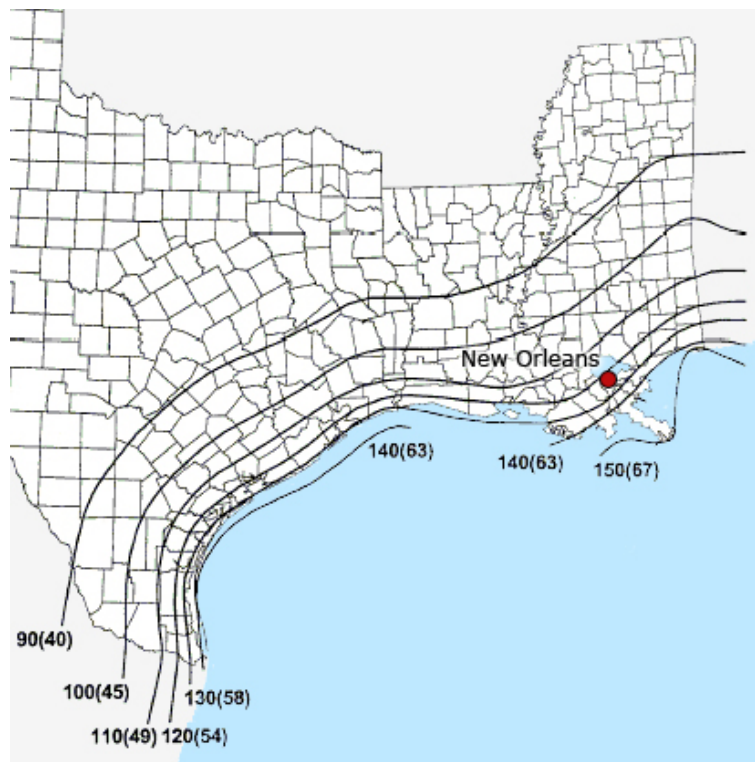
Figure D.1: Basic 3-second gust wind speeds in mph (m/s) at 33ft (10m) for Western Gulf of Mexico. Image from ASCE 7-02 [25].

D.2.4 Combining loads

The buildings must be designed and constructed to safely support the factored loads in load combinations (strength limit states). Also to have adequate stiffness to limit deflections, lateral drift, vibration, or any other deformations that adversely affect the intended use and performance of buildings (serviceability requirements).

According to the Building codes (ASCE 7-05) the buildings shall be designed so that their design strength equals or exceeds the effects of the factored loads in the following combinations (for a structure located in a non-coastal A-zone) [1]:

1. $1{,}4$ (D + F)

2. $1{,}2$ (D + F + T) + $1{,}6$ (L + H) + $0{,}5$ (L_r or S or R)

3. $1{,}2$ D + $1{,}6$ (L_r or S or R) + (L or $0{,}8$ W)

4. $1{,}2$ D + $0{,}8$ W + $2{,}0$ F_a + L + $0{,}5$ (L_r or S or R)

5. $1{,}2$ D + $1{,}0$ E + L + $0{,}2$ S

6. $0{,}9$ D + $0{,}8$ W + $2{,}0$ F_a + $1{,}6$ H

7. $0{,}9$ D + $1{,}0$ E + $1{,}6$ H

Where

D	=	dead load
E	=	earthquake load
F	=	load due to fluids with well-defined pressures and maximum heights
F_a	=	flood load
H	=	load due to lateral earth pressure, ground water pressure, or pressure of bulk materials
L	=	live load
L_r	=	roof live load
R	=	rain load
S	=	snow load
T	=	self-straining force
W	=	wind load

D.3 Break-away walls

A break-away wall with a capacity outside the 10- to 20-psf using the following specifications [10]:

1. The building must be elevated on a piling or other open foundation designed to withstand wind and water loads acting simultaneously and any other loads prescribed by applicable State or local building codes or other applicable laws, ordinances, or regulations.

2. break-away walls shall be designed to meet or exceed all applicable building code requirements for wind, earthquake, and other criteria.

3. The floors of areas enclosed with break-away walls are assumed to be soil or unreinforced concrete slabs less than 4 inches thick. An unreinforced concrete slab has no wire mesh or steel rods. Floor slabs shall not be structurally attached to the pilings or other vertical foundation members.

4. break-away wall panels may be attached to the pilings or other vertical foundation members, elevated floor support beams, or slab floor with nails or other comparable capacity fasteners. All four sides of the wall panel may be attached to the foundation and elevated building. High-capacity connectors such as bolts, lag screws, metal straps, or hurricane fasteners (e.g., clips or straps) shall not be used.

5. he exterior sheathing on break-away wall panels placed between pilings or other vertical foundation members may overlap and be attached to the vertical foundation members.

6. break-away walls may be constructed as continuous, non-bearing walls, attached to the floor and elevated floor joists, with or without attachment to the pilings or other vertical foundation members.

7. break-away wall sheathing and siding may overlap and attach to elevated floor beams and joists provided a separation joint is present to prevent damage to the sheathing or siding of the elevated building if the break-away wall collapses.

8. Utilities, including electrical wiring, breaker boxes, power meters, plumbing, conduits, and ventilation ducts, shall not be placed in or on a break-away wall panel. Building supply lines and other utility line components, such as light switches or electrical outlets, may be attached to pilings or other vertical foundation members as allowed by applicable building codes and floodplain management ordinances and laws (which generally require that utilities be elevated above the BFE).

9. break-away wall panels shall be positioned such that on failure, they do not collapse against cross-bracing or threaten other foundation components.

10. Standard residential garage doors may be considered break-away panels.

11. Because enclosures below buildings in V zones must be constructed with break-away walls that meet the performance requirements of the NFIP regulations, flood vents or openings are not required for such enclosures.

Multi-Criteria Analysis for building solutions

E

Main criteria	A	Subcriteria	B	Weight (AxB)	Reference Piles Score	Result	Alternative 1 Raised on fill Score	Result	Alternative 2 Watertight Score	Result	Alternative 3 Amphibious Score	Result	Alternative 4 Elevating Score	Result
Surroundings	0.2	Architectural	0.2	0.04	6	0.24	8	0.32	8	0.32	9	0.36	8	0.32
		Perception	0.4	0.08	6	0.48	8	0.64	8	0.64	8	0.64	8	0.64
		Durability	0.4	0.08	8	0.64	8	0.64	6	0.48	7	0.56	7	0.56
			1											
Safety	0.3	Wind	0.25	0.08	8	0.6	10	0.75	10	0.75	9	0.675	9	0.675
		Inundation	0.25	0.08	9	0.675	8	0.6	9	0.675	10	0.75	10	0.75
		Flooding	0.15	0.05	9	0.405	9	0.405	6	0.27	9	0.405	10	0.45
		Reliability	0.35	0.11	9	0.945	9	0.945	7	0.735	9	0.945	7	0.735
			1											
User comfort	0.1	Accessibility	1	0.10	6	0.6	8	0.8	6	0.6	10	1	10	1
			1											
Technical impact	0.3	Loss of water retention	0.3	0.09	9	0.81	6	0.54	6	0.54	9	0.81	9	0.81
		Design complexity	0.2	0.06	9	0.54	10	0.6	7	0.42	8	0.48	7	0.42
		Structural stability	0.3	0.09	8	0.72	9	0.81	10	0.9	7	0.63	8	0.72
		Maintenance	0.2	0.06	9	0.54	7	0.42	7	0.42	8	0.48	6	0.36
			1											
Construction	0.1	Building time	0.4	0.04	8	0.32	6	0.24	6	0.24	8	0.32	7	0.28
		Availibility of material/labor	0.6	0.06	10	0.6	9	0.54	7	0.42	7	0.42	7	0.42
			1											
1				**Total**		8.12		8.25		7.41		**8.48**		8.14

Figure E.1: Multi-Criteria Analysis for building solutions.

1. Surroundings

   Architectural
   The first sub-criterion, architectural means the integration of the new constructions in the urban redevelopment by preserving the neighborhoods identity. The architecture must be considerate and sensitive to the scale of the original buildings. Like the traditional buildings, the alternatives are also one- and two-family dwellings. The floating and elevating structures will resemble the old slab-on-grade or shotgun houses the most.

   Perception
   Perception means how people experience and view the buildings within their environment. When the whole area consists of houses raised on piles, they might seem indifferent to ground level, cast more shadow and result in many dark spaces beneath them. This could cause the neighborhood to be regarded as uncomfortable or unsafe. The new alternatives look more like houses built on grade and will offer a more familiar surrounding to people.

   Durability
   The useful service life for a house in The United States is 100 years. The durability is strongly influenced by the types of materials used and to what degree these materials can be reused. Some alternative solutions use more materials, some of which have limited possibilities for renewed use.

2. Safety

   Wind, inundation and flooding
   The buildings must be designed and constructed to safely support the loads and they must be reliable. All the alternatives will be designed to resist the forces from wind loading and flooding, but some differences in performance exist based on their principle design.

   Structures, raised on fill, and buildings with a watertight ground level are placed in the floodplain, which have a negative influence on the flood levels.

   The other solutions try to move as much of the structure above a recommended level making them better equipped to handle flooding and inundation, but subject to higher wind loads. A structure on piles has a limit to its elevation and scores slightly lower than floating and elevating structures.

   Reliability
   The reliability of the designs in comparison with the existing pile house is the same, except for the elevating structures, because of the relative failure change (mechanical elements).

3. User comfort

   Accessibility
   Pile houses and watertight structures with entrances at the second level are not accessible for disabled people. This explains the low score in comparison with the other alternatives.

4. Technical impact

Loss of water retention
Houses raised on fill and buildings with a watertight ground level are placed in the floodplain, which have a negative influence on the flood storage capacity.

Design complexity
Pile houses are commonly applied throughout the United States. For this reason, there is much knowledge about the designs. The elevating structure is an innovative design requiring moving parts and mechanical components. Also the buildings with a watertight ground level are complex because of the many components and materials needed for water tightening.

Structural stability
Each solution inherently possesses a certain degree of stability determining to what extent additional stabilizing systems or components are needed. For example, a floating structure requires additional measures to guarantee stability during floatation. The watertight solution already has a larger mass to resist hydrostatic pressures.

Maintenance
The pile house is, when good constructed, low maintenance-intensive. The raised on fill house (erosion) and elevating structures (mechanical elements) are maintenance-intensive and have a lower score for maintenance.

5. Construction

Building time
Floating houses can be built with pre-fabricated elements; therefore they are faster to erect than conventional building methods. Buildings with a watertight ground level on the other hand, should be built with high accuracy and this will take more time.

Availability of material/labor
Some of the designs need new construction technologies and off-site production of building components. The floating houses for example, need pre-fabricated high strength concrete elements. The mechanical systems for elevating structures have to be pre-produced as well and a skilled technician needs to oversee their installation on site.

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