Master Thesis

Design of a flood proof storage tank

Stolt-Nielsen terminal, New Orleans

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Faculty of Civil Engineering and Geosciences
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Royal HaskoningDHV

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Preface

This graduation report is written as a part of the Master's program Structural Engineering, specialization Hydraulic Structures at the faculty of Civil Engineering and Geosciences at Delft University of Technology. With this graduation report "Design of a flood proof storage tank: Stolt-Nielsen terminal, New Orleans" I will finish my study. This report is executed in cooperation with Royal HaskoningDHV.

I would like to thank all my supervisors of my thesis committee for the guidance and support during my graduation. I also want to thank the employees of Royal HaskoningDHV for the work experience at the office in Rotterdam.

I am very grateful to my parents for their support and for giving me the opportunity to obtain my Master's degree in the Netherlands. I would also like to thank my family and friends who stood by me during my study period. Last but not least a special thanks goes out to my fiancé for his continuous support and patience during the time of my study.

Summary

The occurrence of Hurricane Katrina in 2005 and Hurricane Isaac in 2012 resulted in devastation in the region and killed many people. The surrounding damage was caused by high wind pressures and floodwater caused by the hurricane. The areas that are most affected are those located in the hurricane prone region.
Storage tank terminals which are located in these regions are also prone to hurricanes. An example is the storage tank terminal of Stolthaven Terminals (One of the largest operating units of Stolt-Nielsen Limited) which is located in New-Orleans. Stolthaven New Orleans was severely struck by Hurricane Katrina and Isaac. Damage to the terminal was caused by high wind pressures, but mainly due to flooding of the terminal caused by the overflowing levee.
These floods resulted in the disconnection of storage tanks from their foundation. Due to this event, storage tanks and pipelines were ruptured and chemical liquid stored in the tank had now found its way into the environment. This disaster resulted in high costs and insurance claims from the contamination.

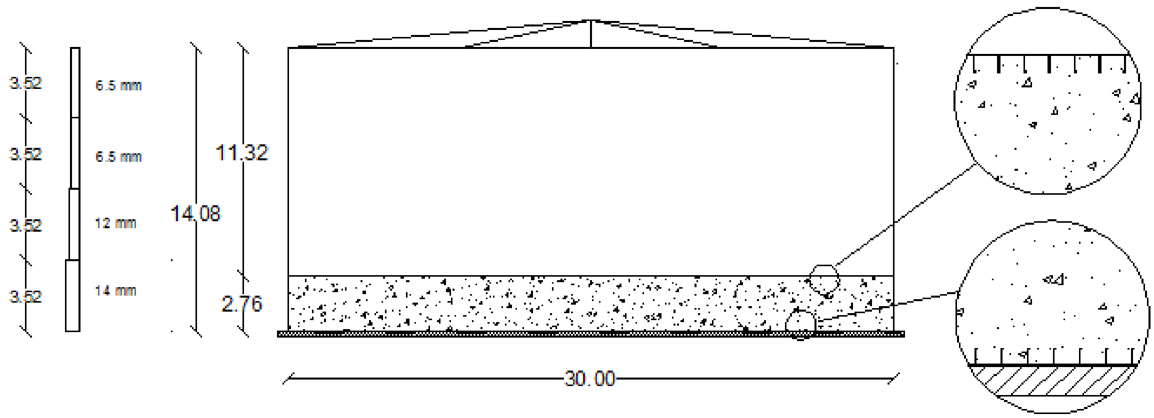
Stolthaven New Orleans considers acquiring more land next to the existing terminal to expand the terminal with more tanks. This new terminal will also be subjected to future hurricanes and is therefore also vulnerable to damage and chemical spill. It is of eminent importance that damage and therefore chemical spill is prevented, not only does the latter entail high costs, it also causes great devastation to the environment.

Stolthaven New Orleans is located in Louisiana where a design basic velocity of 259 km/h holds, this lies in the category 5 hurricane on the Saffir-Simpson Hurricane wind scale. The design water level (flood) on the terminal is according to the 1/100y water level and is equal to NAVD88 +5.6m (NAVD88 is the reference level).
Steel storage tanks are made of thin steel plates, which are welded together. The walls are usually tapered, where the thickest shell course is at the bottom to take up the internal hydrostatic pressure. The tank bottom can have different shapes, depending on the requirements for corrosion. The roofs can also have different shapes, depending on the type of stored material and the vapor accumulation in the upper part of the tank. There are different types of foundations, such as compacted soil, slab etc. In this report only slabs are taken into account.
During the hurricanes storage tanks were floating. A simple calculation shows that the storage tanks did not possess sufficient weight to counterbalance the buoyancy load generated by the flood. This lack of weight is due to the light self-weight of the structure and the insufficient fill level of stored materials in the storage tanks. Due to this, storage tanks were lifted off their foundation and damage occurred to the bottom plate and the connections.
The problem can be formulated as: "How can storage tank damage and chemical spill best be prevented during a flood?"

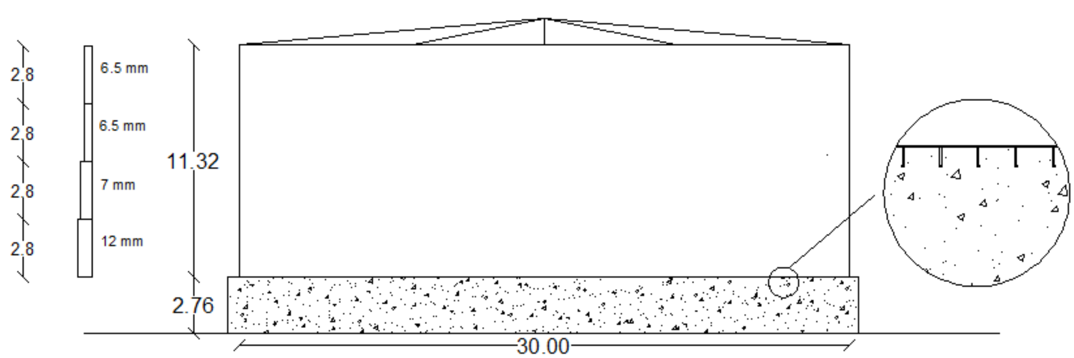
Alternative solutions are presented which are applicable to the still-to-be-built new terminal next to the present terminal. A selection is made between protecting the whole terminal against flooding (e.g. a floodwall) or making the tanks flood proof. Selective argumentation resulted in the selection of the flood proof storage tank. Also a combination of the flood proof tank and managing tank operations is considered. Managing of tank operations means to maintain a certain level of liquid inside the storage tank in order to increase the weight on the bottom plate.

Distinction is made between different alternatives of the flood proof tank. The buoyancy load acts directly on the bottom plate, so the weight on this plate should be sufficient to prevent uplift and local damage to the tank. 3 of the alternatives seem structurally feasible and are further elaborated in the report. These are:

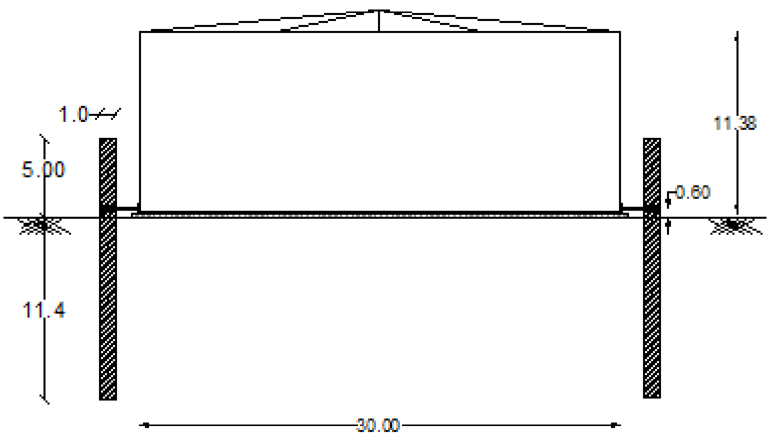
1. Adding weight to the structure with the use of a steel-concrete-steel sandwich slab (SCS slab), where the slab consists of 2 steel plates with a concrete core. The steel plates are connected by means of shear studs to the concrete core. This slab in situated inside the storage tank and should have sufficient weight to counteract the buoyancy load. [Tank alternative A]



2. Using the concrete foundation of the storage tank to provide sufficient weight to the bottom plate by anchoring the area of the bottom plate to the foundation by means of shear studs. [Tank alternative B]



3. Constructing a floating tank, where the tank is allowed to float during a flood. This tank is provided with guiding piles which allows the tank to go up and down, but keeps the tank at one location. [Tank alternative C]



Tank alternatives A and C require a concrete height of 2.76m with regard to the 1/100y flood. The buckling pressure of these tanks is also determined, where it is concluded that the tanks wind pressure (2.8 kN/m^2) and external hydrostatic pressure (46kN/m^2) on the tank are much more than the critical buckling pressure. The critical buckling pressure is determined by means of the weighted smeared method and the design criteria in API Standard 650. To prevent buckling to the shell, the tank should be provided with circumferential stiffeners. For Tank alternative C the required length of the piles is approx. 16.4m for a number of 4 piles around the tank.

Attention should also be given to some details of the structure. For Tank alternative B, the area around the perimeter of the tank between the shell and the foundation should be made watertight, so the accumulation of water between the shell and the foundation is prevented. Tank alternative C should be provided with flexible pipelines and connections to cope with the floating of the tank. It is here also important to prevent the accumulation of debris under the tank if the tank is floating (e.g. by providing a wire mesh around the columns (on the outside)).

Finally a simplified cost-benefit-analysis (CBA) is done to compare the alternatives to one of the basic solutions like the floodwall. The rough approximation of the costs of the variants show that the floating tank (Tank C) can be 30% more expensive than the costs of the current storage tank. For the other 2 alternatives this is more than 200%. The results show that the net present value of Tank C (floating tank) has the highest value. With this highest NPV, this alternative seems to be the most promising for implementation at the new terminal. The NPV of all scenarios are of the same order of magnitude and lie between €230 million and €300 million for a period of 20 years, so Tank A and Tank B are not ruled out. The flood proof storage tank is compared to the basic solution (like the flood wall) a flexible alternative, because the tank does not need to be protected against floods.

Contents

Preface 1-3
Summary 1-4
1 Introduction 1-12

Phase 1 Literature study & Introduction 1-14

2 Hurricanes, floods and storage tanks 2-15
2.1 Hurricanes 2-15
2.2 Floods 2-16
2.3 Storage tanks 2-16
2.4 Damage observations of tanks during Hurricane Katrina 2-16
2.5 Chemical spill 2-18
3 Stolt-Nielsen terminal, New Orleans 3-21
3.1 Location and site description 3-21
3.2 Environmental conditions 3-23
3.2.1 Earthquakes 3-23
3.2.2 Rainfall 3-23
3.2.3 Hurricanes 3-23
3.2.4 Floods 3-24
3.2.5 Geotechnical information 3-26
3.3 Terminal information 3-26
3.4 Damage during Katrina and Isaac 3-27
3.4.1 Damage overview of storage tanks 3-28
3.4.2 Damage analysis 3-31
3.5 Hurricane pre-storm damage mitigating proposals 3-34
4 Steel storage tanks 4-35
4.1 Classification of storage tanks 4-35
4.2 Tank farms 4-35
4.2.1 Terminal operations 4-35
4.2.2 Secondary containment 4-36
4.3 Storage tanks 4-36

4.4 Design 4-38

4.4.1 General design considerations 4-38

4.4.2 Structural design considerations 4-41

4.5 Impact of the height/diameter (h/d) ratio of steel storage tanks on loads 4-42

4.6 Loads 4-48

5 Problem statement: "How can storage tank damage and chemical spill best be prevented during a flood?" 5-51

Phase 2 Alternative study and concept design 5-52

6 Alternatives for protecting the tank from flood loads 6-53

6.1 Alternatives 6-53

6.2 Selection 6-57

7 Possible storage tank solutions 7-59

7.1 Modifications for a fixed structure 7-60

7.2 Modifying the tank into a floating structure 7-69

7.3 Manage tank operations 7-70

7.4 Combination 7-71

7.5 Alternative selection 7-72

7.6 Multi Criteria Analysis 7-73

8 Elaboration concept tank design 8-77

8.1 Steel-concrete-steel composite slab (SCS slab) 8-77

8.1.1 Concept design 8-78

8.1.2 Building procedure 8-81

8.1.3 Combination with tank operations 8-81

8.1.4 Elaboration on details 8-84

8.2 Foundation anchoring from the bottom plate 8-91

8.2.1 Concept design 8-91

8.2.2 Building procedure 8-92

8.2.3 Combination with tank operations 8-93

8.2.4 Elaboration on details 8-93

8.3 Floating structure 8-95

8.3.1 Concept design 8-95

8.3.2 Building procedure 8-98
8.3.3 Elaboration on details 8-99

Phase 3 Concluding analysis 8-102

9 Simplified Cost Benefit Analysis (CBA) 9-103
9.1 Introduction 9-103
9.2 New terminal layout proposal..........9-103
9.3 Tank costs..........9-104
9.3.1 Current tank costs 9-104
9.3.2 Modified tank costs..........9-104
9.3.1 Flood wall 9-107
9.3.2 Levee 9-107
9.4 Benefits 9-108
9.4.1 Probability of failure before ($P_{f,0}$)..........9-109
9.4.2 Probability of failure after ($P_{f,N}$) 9-109
9.4.3 Damage (D)..........9-109
9.5 Analysis..........9-110
10 Conclusions & recommendations..........10-113
10.1 Conclusions 10-113
10.2 Recommendations 10-114

Bibliography 10-115

11 Appendices..........11-117
Appendix I Water level and waves..........11-118
Appendix II Bore log 11-121
Appendix III Tank roofs 11-122
Appendix IV General design considerations..........11-124
Appendix V Surface roughness..........11-126
Appendix VI General on shells [34] 11-127
Appendix VII Loads 11-129
Appendix VIII Stress distribution in a cross-section of a common composite slab 11-138

Appendix IX SCS sandwich slab 11-141

Appendix X Stability of the structure 11-144

Appendix XI The maximum bearing capacity of a laterally loaded pile 11-148

Appendix XII Input data for calculations 11-150

Appendix XIII Calculations for Tank A 11-151

Appendix XIV External pressure 11-162

Appendix XV Paper: 11-167

Practical calculations for uniform external pressure buckling in cylindrical shells with stepped walls

Appendix XVI Results of the CBA 11-172

1 Introduction

This report gives a presentation of the work conducted for the Master thesis project:
“Design of a flood proof storage tank: Stolt-Nielsen terminal, New Orleans.”

The occurrence of Hurricane Katrina in 2005 and Hurricane Isaac in 2012 resulted in devastation in the region and killed many people. The surrounding damage was caused by high wind pressures and floodwater that comes with the hurricane. The areas that are most affected are those located in the hurricane prone region.
Storage tank terminals which are located in these regions are also prone to hurricanes. An example is the storage tank terminal of Stolthaven Terminals (One of the largest operating units of Stolt-Nielsen Limited) which is located in New-Orleans. Stolthaven New Orleans was severely struck by Hurricane Katrina and Isaac. Damage to the terminal was caused by high wind pressures, but mainly due to flooding of the terminal.
These floods resulted in the disconnection of storage tanks from their foundation. Due to this event, storage tanks and pipelines were ruptured and chemical liquid stored in the tank found its way into the environment. This disaster resulted in high costs and insurance claims because of contamination.

Stolthaven New Orleans considers acquiring more land next to the existing terminal to expand the terminal with more tanks. This new terminal will also be subjected to future hurricanes and is therefore also vulnerable to damage and chemical spills. It is of eminent importance that damage and therefore chemical spill is prevented, not only does this entail high costs, it also causes great devastation to the environment.

This report provides a damage overview and solutions to prevent damage to storage tanks due to floating and thepossible resulting chemical spill during a hurricane. Also a cost-benefit analysis is done to relate the benefits of the solutions to the costs required for the implantation of these solutions.

For this project Stolthaven New-Orleans is used as case study.

Report Structure

In the first part of the report, chapters 2 through 4 a description is given on hurricanes , storage tanks and the effects a hurricane can have on a terminal and the storage tanks located on that terminal. Here also an indication is given on the impact the the height-to-diameter (h/d) ratio of a storage tank has on the loads. These chapters are intended to give a clear view of the issues, to finally come to a problem statement.
Chapter 5 gives a clear statement of the problems and describes the further approach in the remaining report.
The following part of the report, chapters 6 through 8 an alternative study is done, where a selection is made on the best possible alternatives. These alternatives are here further elaborated.
Chapters 9 and 10 are the final phases of the thesis, where an concluding analysis is done with a cost-benefit analysis (CBA). Finally de conclusions and recommendations are stated.

Chapter 1
Introduction
- Introduction
- Report structure

Literature study & Introduction

Chapter 2
Hurricanes, floods and storage tanks
- Hurricanes
- Floods
- Storage tanks
- Damage observations
- Chemical spill

Chapter 3
Stolt-Nielson terminal, New Orleans
- Location and site description
- Environmental conditions
- Terminal information
- Damage during Katrina and Isaac
- Hurricane pre-storm damage mitigating proposals

Chapter 4
Steel storage tanks
- Classification of storage tanks
- Tank farms
- Storage tanks
- Design
- Impact of the height/diameter (h/d) ratio of steel storage tanks on loads
- Loads

Chapter 5
Problem statement
"How can storage tank damage and chemical spill best be prevented during a flood?"

Alternative study & concept design

Chapter 6
Alternatives for protecting the tank from flood loads
- Alternatives
- Selection

Chapter 7
Possible storage tank solutions
- Modifications for a fixed structure
- Modifying the tank into a floating structure
- Manage tank operations
- Combination
- Alternative selection

Chapter 8
Elaboration concept tank design
- Steel concrete steel composite slab (SCS slab)
- Foundation anchoring from the bottom plate
- Floating structure

Concluding analysis

Chapter 9
Simplified Cost Benefit Analysis
- Introduction
- New terminal layout
- Tank costs
- Benefits
- Analysis

Chapter 10
Conclusions and recommendations

Phase 1 Literature study & Introduction

2 Hurricanes, floods and storage tanks

This chapter is intended as introduction and gives a brief description of the main components of the thesis. This is done to give an idea of what this thesis is about. A brief description is given of hurricanes, floods, storage tanks, damage observations of tanks during hurricane Katrina and chemical spill.

2.1 Hurricanes

Hurricanes, also called tropical cyclones, are generally a characteristic of tropical and subtropical waters. They can develop in the Gulf of Mexico, the Caribbean Sea, the Pacific and Atlantic Oceans during the summer and fall months. Hurricanes are the most active between mid-August through October in the Atlantic basin (the North Atlantic Ocean, the Gulf of Mexico and the Caribbean Sea). The coastal areas of the United States are particularly exposed to hurricane disasters because of their topographical composition and large concentrations of population. Most of the U.S. hurricane-related fatalities have occurred in the areas along the Gulf and Atlantic coasts. This can be seen in
Figure 2-1. [1]

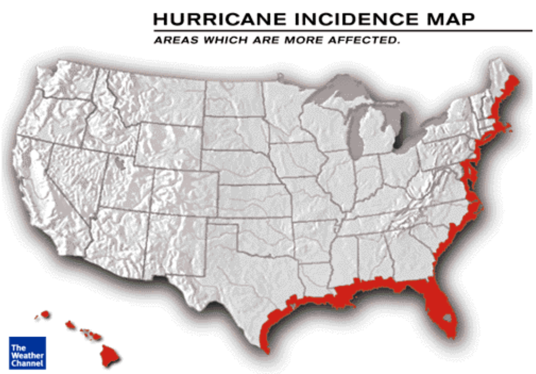


Figure 2-1 The Gulf and Atlantic coasts; areas which have high hurricane risk

The Saffir-Simpson Hurricane Scale, shown in Figure 2-2, categorizes hurricanes on a scale from 1 to 5 based on their wind intensity. This scale is used to alert the public of possible impacts and type of damage a certain type of hurricane can cause at landfall. The scale is based on peak 1-minute winds at the standard meteorological observation height of 10m over unobstructed exposure. [2]

Category	Winds	Damage
	Km/h	
1	119-153	Minimal
2	154-177	Moderate
3	178-208	Extensive
4	209-251	Extreme
5	>252	Catastrophic

Figure 2-2 The Saffir-Simpson Hurricane Wind Scale

Close to the ground the wind is slowed down, but takes on velocity when higher from the ground. Also long open spaces, airfields, lakes, sea are influencing the full force of the wind. Obstacles standing against the wind slow the wind down. [3]

2.2 Floods

Flooding can be described as the unusual presence of water on land that is normally dry. The water has a certain depth which affects normal activities. Flooding of land can have different causes such as overflowing rivers (river flooding), a short duration of heavy rainfall (flash floods), inflow of seawater onto land (ocean flooding) [4].
The inflow of seawater onto land can have different causes, such as:

- storms/hurricanes (storm surge)
- high tides (tidal flooding)
- seismic activities (tsunami)
- large landslides (also called tsunami at times)
- dam failure/levee overtopping

Flooding of land can have devastating consequences to man-kind and the environment. Some consequences of such a flood event are loss of lives, damage to structures and environmental damage.

2.3 Storage tanks

Steel tanks are used for various different applications, for the storage of liquid material such as water, oil, chemical liquids, Liquid nitrogen gas (LNG) etc. and also for solid materials. In this study we only take liquid storage tanks into account. A more detailed description on storage tanks is given in Chapter 4.

Figure 2-3 Example of a steel storage tank [http://www.stolt-nielsen.com/Stolthaven-Terminals.aspx]

The steel storage tank is made out of thin steel plates. The minimum shell wall thicknesses depend on the tank diameter. For example a tank with a diameter of 15m has a minimal wall thickness of 5mm and a tank with a diameter of 60m has a minimum wall thickness of 9.5mm. [6] The roof and the bottom plate are also made with this steel plates. The tank structure has therefore a relatively light weight when its empty.

2.4 Damage observations of tanks during Hurricane Katrina

In this paragraph some examples are given of damaged storage tanks during Hurricane Katrina and Rita. These observations are the result of several expeditions for the observation of damaged tanks at several oil refineries and facilities in Texas and Louisiana.

After Hurricane Katrina and Rita in 2005, an inspection was conducted by a team of experts in the states of Texas and Louisiana regarding the structural damage in aboveground storage tanks in refineries and facilities. [7] The observed damage modes are presented below.

	Damage mode	Cause
1	Buckling of the shell combined with fracture of the welding at the base	Wind pressure
2	Detachment of the tank from foundation, displaced and pipelines were damaged	Storm surge/flood
3	Global buckling of the cylindrical shell	Wind pressure
4	Local buckling of the cylindrical shell	Wind pressure
5	Loss of insulation	Wind pressure
6	Roof damage	Wind pressure

Figure 2-4 Overview of the observed damage modes of storage tanks during Hurricane Katrina and Rita

The first three damage modes can cause major damage to the structure of the tank and/or damage to the piping system. These damage modes can therefore lead to spill of liquid on the terminal.

A brief description of the damaged modes is presented:

1. Structural failure; buckling in a mode involving very large localized displacements of the cylindrical shell in unanchored tanks. These tanks had very large deflections in the cylindrical shell and in the base plate. The observed displacements were near 1m affecting an area with a central angle of 15-30°. This points out that the wind pressures were far above the buckling loads of the tanks. Initial results show that the buckling was due to wind loads which were lower than 192km/h. This failure mode was a combination of fracture of the welding between the cylindrical shell and the base plate and buckling of the cylindrical shell

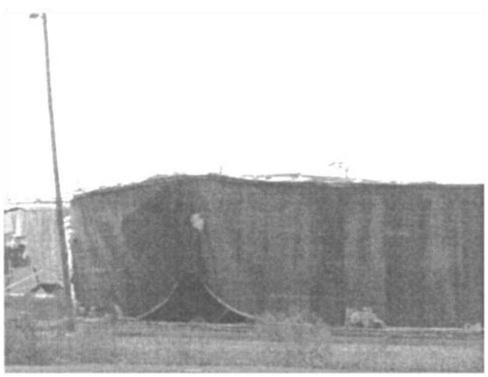


Figure 2-5 Example of a buckled tank in Port Sulphur, along the Mississippi river, Louisiana

2. Uplift of the tank off its foundation, caused by storm surge and flood. This resulted in displacements of the tank and damage of the piping system.

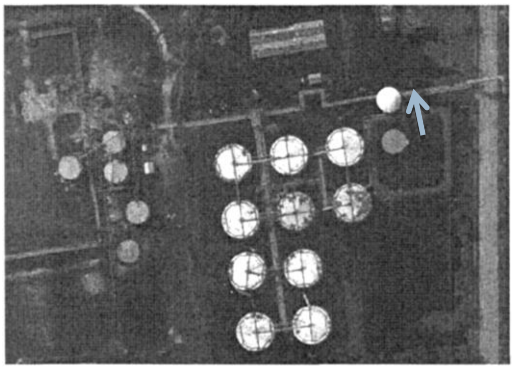


Figure 2-6 Example of a displaced tank in Port Sulphur, along the Mississippi river, Louisiana

3. Global buckling modes associated with deflections of a large portion of the cylindrical shell, fracture of the metal shell or foundation uplift.
4. Localized buckling of the cylindrical shell with amplitudes of buckling deflections nearly three times the thickness of the shell. Tanks are overall designed to resist liquid pressure, which is why the shell wall is designed with variable thickness. It is therefore understandable that the upper part of the cylinder is the most weakest. This can cause functional problems if there is an internal roof present. The internal roof demands small deflections of the shell walls in order to move in vertical direction.
5. Loss of shell insulation in the cylindrical shell and in parts of the roof. The damage of insulation at the center or at the top of the roof was caused by high wind pressures in that zone (concluded from wind tunnel experiments). The damage of shell insulation at the bottom of the tank is expected to have been caused by vortex effects, because wind pressures are small there.
6. Roof damage was related to wind suctions related to high wind pressures occurring at the cylinder-to-roof connection and at the center of the roof. (This corresponds to the results conducted from wind tunnel observations).

Some other observations during these inspections were:

1. Tanks with ring stiffeners (at the roof junction or in the cylindrical shell) did not display buckling despite the fact that some close-by tanks without stiffeners had buckled.
2. A lot of tanks that were damaged by wind were on an outer location in a plant. The tanks are here more exposed to the wind and were not sheltered by other tanks or structures (concluded from wind tunnel experiments).
3. Floating of a storage tank does not appear to be a design constraint in the standards and codes used for the design of storage tanks.

2.5 Chemical spill

A spill is an accidental release of a chemical from a type of containment, in this case storage tanks. Spill refers to a liquid chemical, but these releases can also be a solid or a gas.

Prevention

A secondary containment is a dike wall or impoundment area that is built around a couple of storage tanks (tank field) to capture the volume of the chemicals in case of spill. The spill can then be immediately removed and disposed of. The secondary containment should be able to contain the capacity of the largest tank inside the containment. [6] (see also section 4.2.2)

Consequences
These releases can lead to the entering of dangerous chemicals into the environment. This can take place through land, water or air. Spill containing poisonous or toxic chemicals can have a fast and dangerous effect on living organisms in the area. It can also have severe effects on the environment. [8] The contamination of the groundwater will have severe effects on the surrounding area and can be toxic to humans, animals and plants.
The consequences are not only devastating for the environment, but will have high costs related to the cleaning of the spilled area.

Cleanup costs
Cleanup costs of chemical spills are not easily determined. [9] They depend on a set of circumstances, which are unique at each spill. The main factors affecting the cleanup costs are the location and oil/chemical type, and possibly total spill amount. Other influencing factors are: timing of the spill; sensitive areas affected or threatened; liability limits in place; local and national laws; and cleanup strategy.

Location
Most experts agree that this is the most important factor. It involves geographical, political and legal considerations.
For example: Spills that have a large effect on shore are far more expensive to clean up than spills occurring offshore.

Oil/chemical type
The type of spilled material is another important factor to determine the cleanup costs. The composition and physical properties of for example oil will influence the degree of evaporation and natural dispersion and also the complexity. Oils with a lower relative density will evaporate and disperse more than heavier oils. Heavier oils are difficult to remove, resulting in higher cleanup costs.

Spill amount
The amount of spill can have a large effect on the cleanup costs. Generally, it can be said that the larger the spill the more expensive the cleanup costs. This is however not correct, the cleanup costs on a per-tonne basis decreases considerably with increasing amounts of oil spilled. Smaller spills are on a per-tonne basis often more costly than larger spills because of the costs related to the set up of the cleaning response, like equipment, labor and also experts for the evaluation of the location.

An approximation for the cleanup costs of a spill is very complex to determine. In 1997, the average cleanup cost per-tonne spilled in the region of the Gulf of Mexico was approximately $73,000.

3 Stolt-Nielsen terminal, New Orleans

In this part of the report more information is given of the Stolt-Nielsen terminal in New Orleans. A description is given on the location and the site as well as the environmental conditions and terminal information. This terminal was heavily struck by Hurricane Katrina and Hurricane Isaac, that is why an overview of this damage is also included in this chapter. This terminal is used as case study, therefore site specific information is required in order to formulate a clear problem definition and finally to generate solutions.

3.1 Location and site description

Stolt-Nielsen Limited provides transportation, storage and distribution of bulk liquid chemicals and other specialty liquids. One of Stolt-Nielsen's largest operating units is Stolthaven Terminals, where bulk liquid is stored and distributed to customers. One of Stolthaven's bulk liquid terminals is located in New-Orleans. It is located at the East Bank of the Mississippi River near Braithwaite, Louisiana. This is right off the coast of the Gulf of Mexico.

Figure 3-1 Location of Stolthaven Terminal in Louisiana

Stolthaven is situated in a polder, which is surrounded by two levees of 35 miles, the Mississippi River Levee (MR Levee) and the Gulf of Mexico side Levee (GoM Levee). The considered required land is also surrounded by these levees. The levee system is shown in **Error! Reference source not found.**.

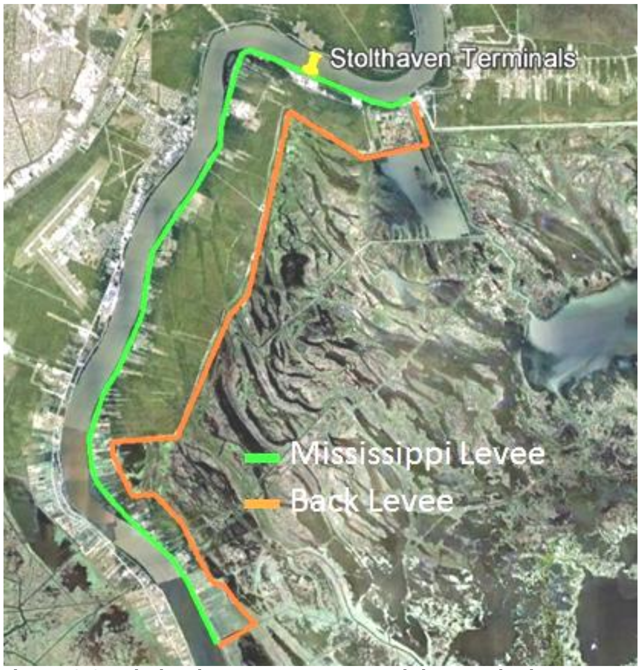


Figure 3-2 Existing levee system around the terminal

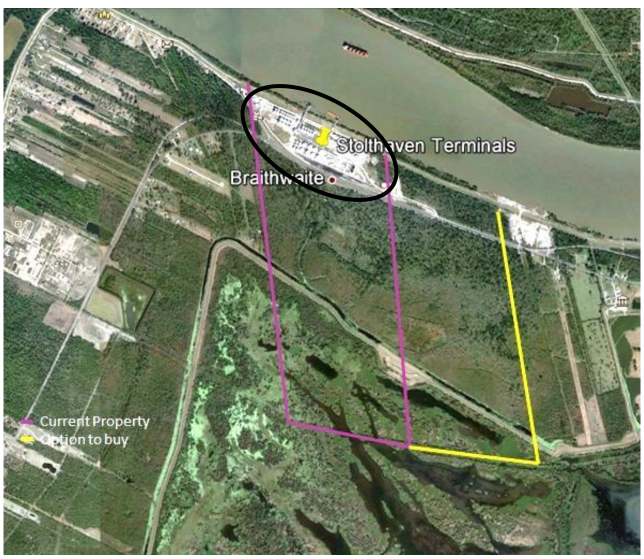


Figure 3-3 The existing property of Stolthaven (purple) and land under consideration for expansion (yellow)

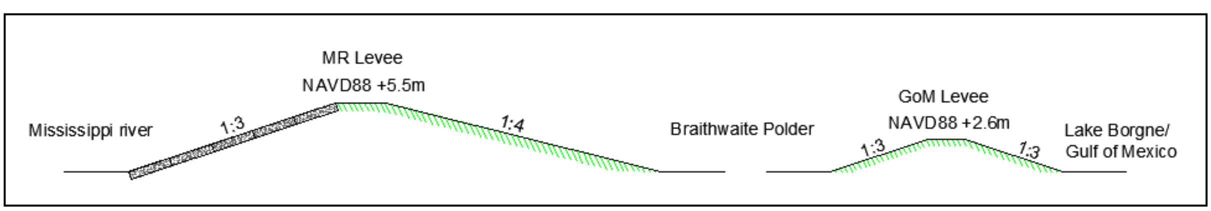


Figure 3-4 Schematic cross-section of the levees surrounding the polder (Not in scale in longitudinal direction)

The MR Levee has a safety level of about 1/100 years and the GoM Levee has a safety of 1/5 - 1/10 years. (According to water level statistics, see Figure 11-1 & Figure 11-2)
The crest level of the MR Levee is about 5.5m high. The reference level used in New Orleans is NAVD88 2004.65 (further referred to as NAVD88). The polder is therefore protected from (extreme) high river flows and high water levels due to hurricane surges from the Gulf of Mexico. The average height of the GoM Levee is 2.6m above NAVD88.
This height of the GoM Levee is low compared to that of the MR Levee. During Hurricane Katrina and Isaac the terminal became flooded due to overtopping of the GoM Levee.

3.2 Environmental conditions

The environmental conditions of Solthaven are listed below. These include: earthquakes, rainfall, hurricanes, floods, and geotechnical information of the location.

3.2.1 Earthquakes

Louisiana is located in a region with low seismic risk. Historic data shows that small earthquakes do occur from time to time [10]. The largest earthquake that occurred in Louisiana was in 1930 and had an intensity of VI on the Modified Mercalli Intensity Scale (MMI) [11]. A VI on the MMI means the damage is small. For the remainder of the study, earthquakes are not taken into account.

3.2.2 Rainfall

The average annual rainfall of Louisiana is 1587mm (62.45inch)[1]. This is lower than the design water level of a flood, which is used in the remainder of the thesis, therefore rainfall is not taken into account. The rainfall is important for the design of pumps on the terminal.

3.2.3 Hurricanes

The Stolthaven terminal New Orleans is located at the East Bank of the Mississippi River near Braithwaite, Louisiana. This is right off the coast of the Gulf of Mexico, which lies within the area of high hurricane risk. (see Figure 2-1)
Table 3-1 shows a total of 53 hurricane hits in Louisiana from 1851 to 2009. In this period Louisiana encountered 1 category 5 hurricane, which was Hurricane Camille in 1969.

Direct hurricane hits between 1851 and 2009					
	Saffir-Simpson Hurricane Scale Category				
	1	2	3	4	5
Louisiana	18	15	15	4	1

Table 3-1 Number of direct hurricane hits in Louisiana between 1851 and 2009 [http://www.hpc.ncep.noaa.gov/research/lahur.pdf]

The probability of any hurricane passing with windspeeds of 175km/h or higher within a radius of 1.6km of New Orleans is 12.5% per year. For a major hurricane this is 3% per year. The windspeed belongs to the category 3 hurricanes and gives an indication of the relative danger that can occur.[2]

<u>Wind speed during hurricane</u>
The terminal was heavily struck by Hurricane Katrina and Hurricane Isaac. Hurricane Katrina is known to be the most costly and one of the deadliest natural disasters ever to strike the United States.

[1] http://www.usclimatedata.com/climate/louisiana/united-states/3188
[2] http://usatoday30.usatoday.com/weather/hurricane/history/probabilities-table.htm

In Plaquemines Parish, Katrina made landfall as a category 3 hurricane on the Saffir-Simpson Hurricane Wind Scale with wind speeds of approximately 204.4km/h. Winds in hurricanes increase from the ground upward to a few meters high. [12] [13]

Basic wind speed

This basic wind speed should be used as a condition in calculations. Figure 3-5 shows the wind speed at the western side of the Gulf of Mexico. The wind speed is based on a 3-second gust wind speeds in m/h and m/s at 10m (33 ft.) above ground for Exposure C category. Stolt-Nielson lies within the region with a basic wind speed of 72 m/s (259 km/h).

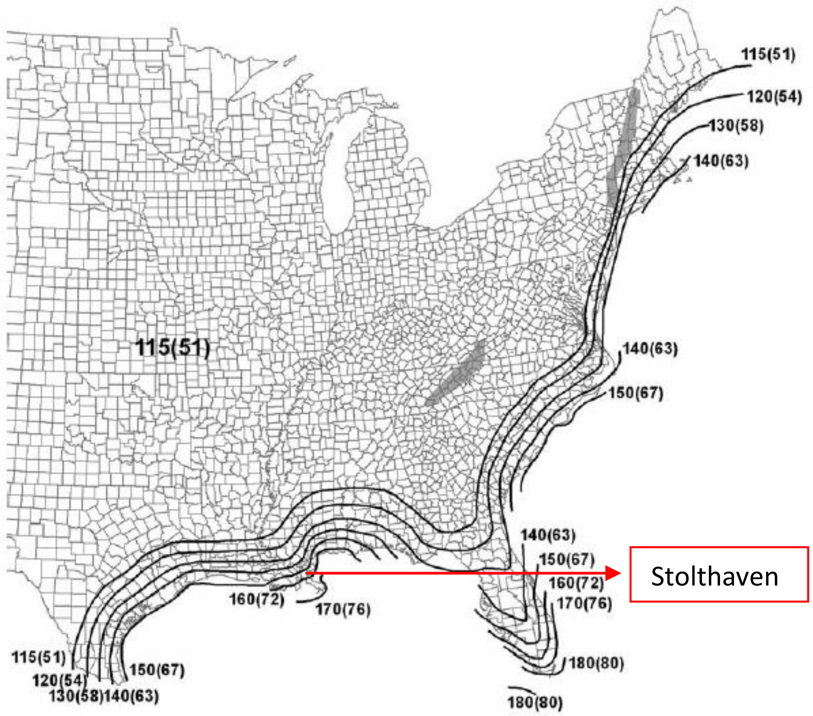


Figure 3-5 Basic wind speed in miles per hour (m/s) - Western Gulf of Mexico hurricane coastline [14]; with the location of Stolthaven

3.2.4 Floods

Approximately 30% of Louisiana's land mass lies in the Special Flood Hazard Areas (SFHAs). The SFHA is the land area covered by the floodwaters of the base flood on National Flood Insurance Program (NFIP) maps. In this area the NFIP's floodplain management regulations must be enforced and the mandatory purchase of flood insurance applies. This is land which is subject to a 1 percent or greater chance of flooding in any given year (base flood). These areas are delineated on a community's FIRM (Flood Insurance Rate Maps) as A-zones or V-zones.

Coastal V-zone: extends from the offshore to the inland and is subject to high-velocity wave action from storms or tsunamis.

Coastal A-zone: lies landward of a V-zone or landward of an open coast without mapped V-zones. The main source of flooding is coastal storms, with a potential base flood wave height between 0.46m and 0.91m (1.5-3.0 feet).

A-zone: in these areas the potential source of flooding is runoff from rainfall, snowmelt or coastal storms where the potential flood wave height is between 0 and 0.91m (0-0.3 feet).

X-zone: the flood hazard is less severe here than in the SFHA.

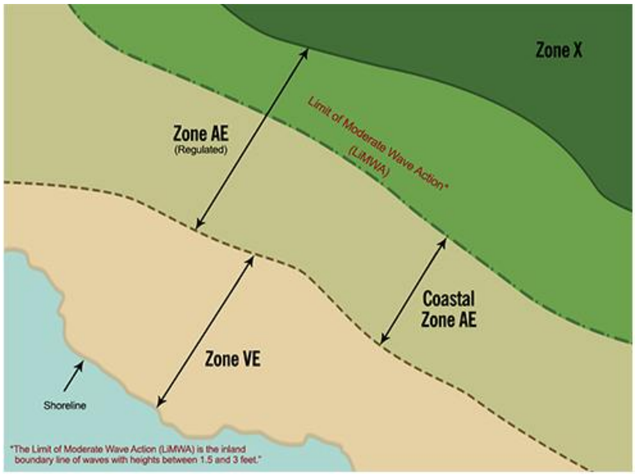


Figure 3-6 Flood Insurance Rate Map [https://www.floodsmart.gov/floodsmart/pages/coastal_flooding/coastal_flood_maps.jsp]

The Stolt-Nielsen terminal is located near Braithwaite, in Plaquemines Parish, Louisiana. On the Effective Flood Insurance Rate Map, Braithwaite is categorized as an A-zone [3]. Also the RoyalHaskoningDHV study on waves shows a 1/100y wave height of 0.34m, which indicates that the area most likely lies in a A-zone.

Water level

Near the levee in the Gulf of Mexico (GoM) and at the Mississippi River (MR) water level statistics are gained from a Joint Coastal Surge Study for Louisiana. It was conducted by the US Army Corps of Engineers after Hurricane Katrina. The study focuses on return periods between 1/50 – 1/500 years and presents results of surge levels and wave statistics of a combination of detailed surge modeling and a probabilistic analysis. [15]

For surge levels with return periods higher than 50 years, the back levee will be overpowered by hurricane surge. Therefore the surge level near the terminal will be the same as the surge statistics at the Gulf of Mexico.

The water level (flood level) with return period of 100years is used in the remainder of the thesis. This is equal to NAVD88 +5.6m. For the ground elevation 1m is considered (see section 3.2.5), this means flood level on the terminal is approximately 4.6m high. This height will generate an hydrostatic pressure of 46 kN/m^2 on the storage tank.

During hurricane Katrina and Isaac, a water level of respectively NAVD88+2.1m, NAVD88+4.3m was registered at the Stolthaven terminal.

Waves

The waves in front of the GoM Levee are predicted by Royal HaskoningDHV using the 1D SWAN Model (Simulating WAves Nearshore). The findings of this study are presented in Appendix I. The wave height with return period of 100years is used in the remainder of the thesis. This is equal to 0.34m.

[3] http://maps.riskmap6.com/LA/Plaquemines/

3.2.5 Geotechnical information

There is no geotechnical information available for the new considered area. The geotechnical data available of the existing terminal area will be used throughout this thesis research.

Ground profile

From the boring and test results [16] the following can be concluded. At a total of 33 locations on the existing terminal area, boring activities have been performed. The ground penetration depth varied from 24-30m. Up to this depth the ground mainly consists of soft gray clay. A small layer of medium to dense gray silty fine sand is found throughout the whole area between 8 and 14m. The ground profile of one bore log of the existing terminal is given in Appendix II.

Ground elevation of the existing terminal

The figure below shows the ground elevation for the existing terminal. The elevations are with reference to NAVD88.

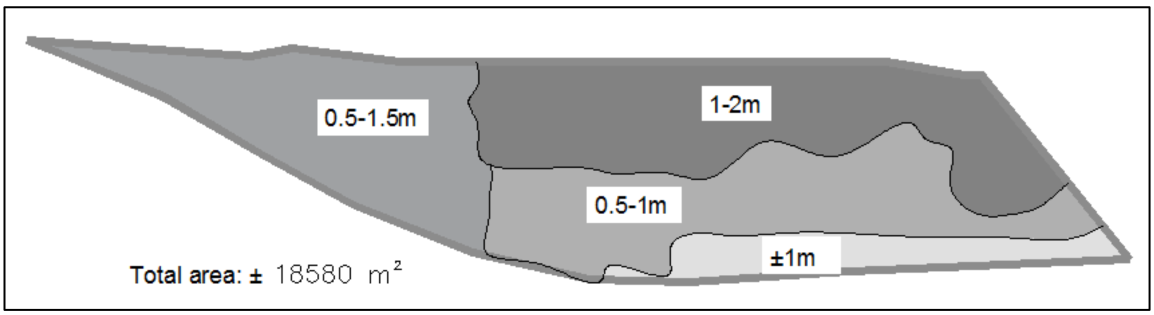


Figure 3-7 Schematic map of the ground elevation of the existing terminal [17].

All levels are above NAVD88 reference. An average ground elevation of NAVD88+1m is considered in the report.

3.3 Terminal information

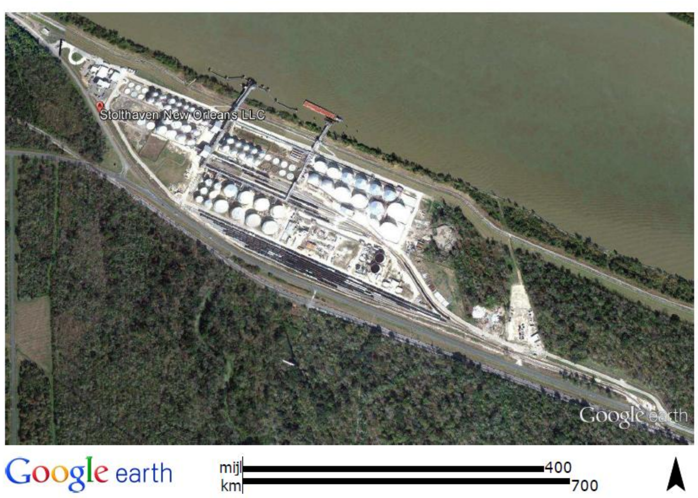


Figure 3-8 Storage tanks on the existing terminal site

The terminal has berthing facilities at the Mississippi River, storage for bulk liquid and facilities for backland connections (rail and highway). Because the Mississippi River Levee is situated in between the river and the terminal, liquid is transported with pipelines across the levee. The storage facilities and hinterland connections are positioned inside the existing levee system.

Figure 3-9 Pipelines crossing the Mississippi Levee [Report: M903703_site_investigation_stolt_nielsen]

Existing tanks

The terminal site consists of a total of 68 storage tanks divided over 7 types in different sizes. The total capacity of the tanks is 308636 m^3. The products that are stored in these tanks are: petroleum products, chemicals and vegetable oil. (see section 4.3 for a full description of the types of tanks)

The tanks are built according to API (American Petroleum Institute) 650 Standards. There are 7 types of tanks located at the terminal, these are:

1. Cone Roof Storage Tank
2. Cone Roof Shovel Bottom Tank (with internal floating roof)
3. Umbrella Roof Shovel Bottom Tank (with internal floating roof)
4. CRT (Cone Roof Tank) Storage Tank (with internal floating roof)
5. Dome Roof Shovel Bottom Tank
6. Umbrella Roof Shovel Bottom Tank
7. Dome Roof Storage Tank

3.4 Damage during Katrina and Isaac

In this section an overview will be given of the damage which occurred to the storage tanks during Hurricane Katrina and Isaac. Information was gathered from reports of Royal HaskoningDHV after a site visit was conducted to the Stolthaven terminal.

General

The Stolthaven terminal in New Orleans has been exposed to major floods during Hurricane Katrina (2005) and Hurricane Isaac (2012). Due to these floods some tanks have been lifted off their foundations, which resulted in damage to the structure and pipelines and caused liquid spill.

1. The occurred damage can be classified as:
2. Damage to electrical components
3. Damage to mechanical components (pipes, etc.)
4. Tanks lifting off their foundations
5. Loss of chemicals of the tanks
6. Environmental impact (and subsequent claims) due to the loss of chemicals

Damage from class 3, 4 and 5 (and partially class 2) are the result of the uplift of the tanks off their foundation. These categories have the largest damage with regard to costs. A study carried out by Royal HaskoningDHV indicated that a small excess of water can already lead to uplift of tanks, which causes damage to the tanks and environment.
Stolthaven Terminal considers acquiring more land next to the existing terminal. It is intended to expand the storage plant with more tanks. This new land will also be subject to future hurricane floods.

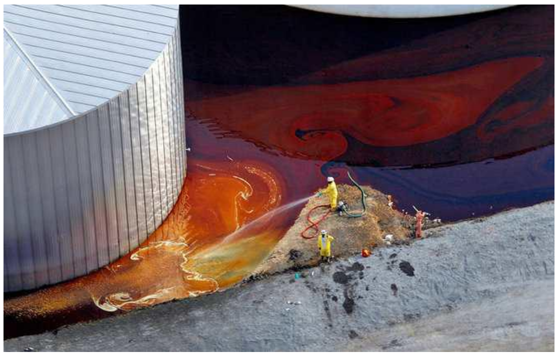



Figure 3-10 Spill of chemicals on the terminal [18]

The elevation map (Figure 3-7) shows that the ground level south of the terminal (near Gulf of Mexico) is lower than the other parts. Storage tanks that are located in this area are more vulnerable to flooding. This is the reason that there is more damage in this area.

3.4.1 Damage overview of storage tanks

The following drawing presents an overview of the tanks situated on the Stolthaven terminal. The red and blue dots are the tanks which are currently situated on the existing terminal. The blue dots indicate the tanks which were damaged during Hurricane Katrina and Isaac. Tanks from which information was provided are indicated with a cross.

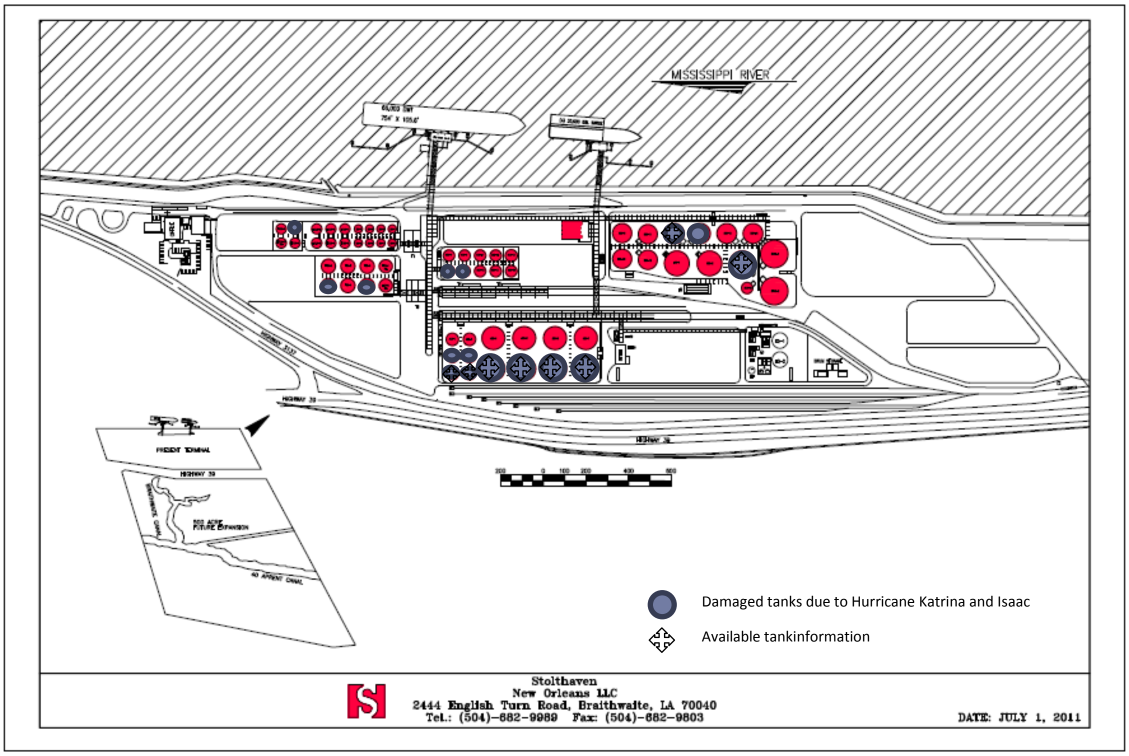


Figure 3-11 Overview of tanks on Stolthaven; including damaged tanks

Hurricane Isaac caused a higher water level on the terminal than Hurricane Katrina. Therefore the number of floating tanks was greater. The water levels that were observed on the terminal were NAVD88+2.1m during hurricane Katrina and NAVD88+4.3m during Isaac.

Description figure	Katrina (2005)	Isaac (2012)
Water level	NAVD88 + 2.1m	NAVD88 + 4.3m
Tanks on site	60	68
Tanks with minor uplift	-	1
Tanks with severe damage	-	5
Tanks lifted off their foundations	4	8

Table 3-2 Number of floating/damaged tanks during Hurricane Katrina and Isaac [17]

The main problem was the uplift and floating of the storage tanks into different directions during the storm. Because of this, the connected pipelines were damaged. The severe damage of the other 5 tanks was supposedly fracture of the tank floor due to buoyancy forces and damage to the shell caused by a combination of buoyancy forces and anchor bolts holding the tanks on its position.

Damage examples are as follows:

1.		Uplift of tanks off their foundation and floating into different directions. Figure 3-14 shows that a small excess of water can already lead to floating tanks.
2.		Buckling of the tank walls. Due to floating, tanks collapsed to other obstacles on the terminal, this caused the walls to buckle. Another reason for the buckling of the walls could be the high wind pressure against the walls of tanks with vacuum inside (partially filled).
3.		Damage to the pipelines that were connected to the tanks. This was caused by the uplift and floating of tanks.
4.		Floating of the tank caused the steel columns inside to lose contact with the foundation. The integrity of the structure was lost. Heavy wind pressure and/or uplift could be the cause of the cylindrical shell to buckle. The columns inside lost contact with the foundation, which resulted in tank failure.

5.		One of the uplifted tanks had anchoring bolts. Due to the uplift of the tank, the bolts were destroyed and were not able to hold the tank to the foundation.
6.		To some tanks that were bolted uplift of the tank floor caused leakage at the bottom of the tank and distortions to the shell.

Table 3-3 Damage examples at Stolthaven

The failure modes which caused spill of liquids during the hurricanes are:

- Rupture of the connection between the tank and the pipelines, due to uplift and displacement of the tank.
  Cause: flooding, storm surge and wind pressure
- Damage of the tank floor, due to uplift of the tank floor
  Cause: flooding

3.4.2 Damage analysis

To get an indication of the effects of the wind and a flood on tanks during a hurricane, a wind check is done with API Standard 650 and some calculations for the stability of the storage tanks.

<u>Wind</u>
The specifications of Tank A50-2 (one of the damaged tanks) states a design wind velocity (*V*) of 120mpa (= **164 km/h**) for the stability of the shell. This used design wind velocity corresponds to a maximum wind pressure of: $1.48\ kPa \cdot \left(\frac{164}{190}\right)^2 = 1.1 \frac{kN}{m^2}$.
API Standard 650 states that for the design velocity of wind, a basic wind speed should be used which depends on the location of the project area. For Stolthaven, which is located in Louisiana this is ≈ **259 km/h** (see Figure 3-5). This corresponds to a maximum wind pressure of: $1.48\ kPa \cdot \left(\frac{259}{190}\right)^2 = 2.8 \frac{kN}{m^2}$.
From this it can be concluded that the design wind speed used in the calculation of tank A50-2 is lower than that given in the standards.
With the design criteria of API Standard 650 (see Appendix XIV A), the allowable design external pressure is calculated for Tank A50-5 to compare this value to the used design wind velocity. The design external pressure corresponding to the wall thicknesses and diameter of Tank A50-5 is: P_s = 0.97kN/m^2.
This is approx. the same as the wind load of the used design wind velocity, so this is okay.

Katrina made landfall with wind speeds of approximately 204.4km/h (≈ 1kN/m^2). Comparing this value to the design external pressure P_s, it can be concluded that the tank can barely withstand the wind speed (1kN/m^2≈ P_s).

Allowable design external pressure according to wall thickness $\boldsymbol{P_s}$ (API)	Katrina's wind pressure	Design wind pressure according to used wind velocity (used in design)	Design wind pressure by API (should have been used)
0.97kN/m^2	1kN/m^2	1.1kN/m^2	2.8kN/m^2

It can be concluded that Tank A50-5 has been dimensioned with a low design wind pressure. This tank will not be able to withstand a category 4 hurricane, which can have wind speeds up to 251km/h. Buckling of the cylindrical shell will occur.

<u>Checks on vertical stability for existing tanks</u>

For the tanks that have been damaged by Isaac, the uplift and overturning stability will be checked. The locations of the tanks can be seen in the drawing below.

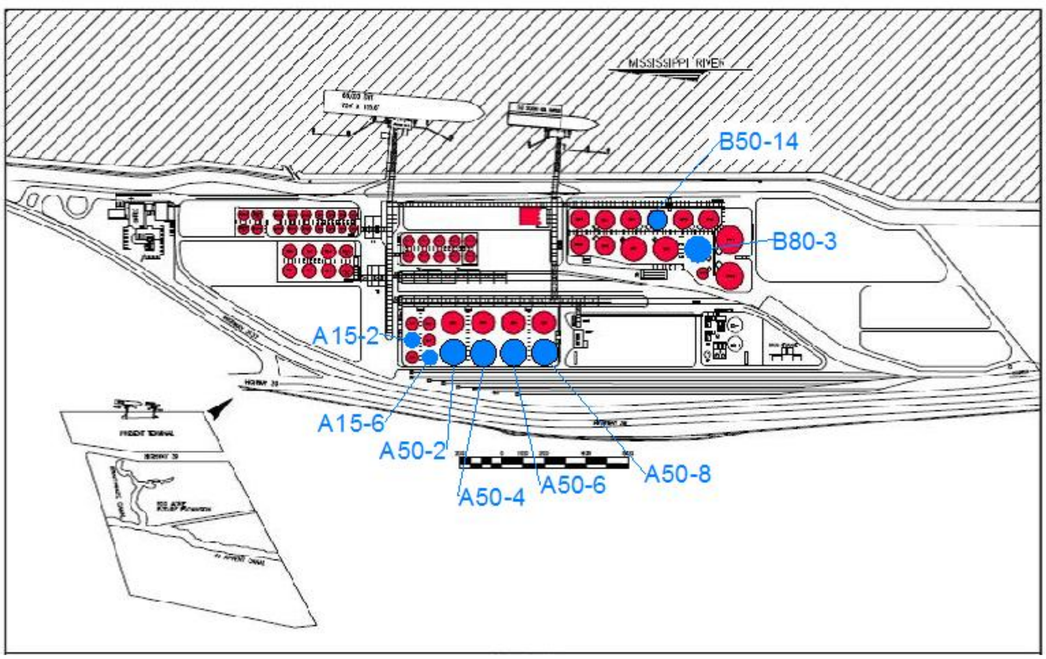


Figure 3-12 Tank numbers and locations of damaged tanks from which information was available

Calculations have been made regarding the uplift of storage tanks. For the damaged tanks, the ratio (*r*) between the buoyancy and total weight is determined. This ratio depends on the total weight of the tank (including storage product) and the height of water outside the tank. Uplift occurs when *r* is larger than 1. All 8 tanks that were lifted off their foundation (**Error! Reference source not found.**) have an *r* value greater than 1, this explains the uplift.
The following input is needed for the calculation of the uplift stability:

The total weight of the structure including the contained liquid is calculated with:

$W_{tot} = (W_t + W_l) \times g$

where:

W_{tot}	= The total weight of the tank including liquid	[kN]
W_t	= Weight of the tank	[kg]
W_l	= Weight of the liquid inside the tank = $V_l \times \rho_l$	[kg]
V_l	= Volume of the liquid	[m^3]
ρ_l	= Liquid density is assumed to be 800	[kg/m^3]
g	= Gravitational acceleration	[m/s^2]

The buoyancy force acting on the structure is calculated with:

$B = V_{dis} \times \rho_w \times g$

where:

B	= The buoyancy force [kN]	
V_{dis}	= The displaced volume of the tank = $1/4 \times \pi \times d^2 \times (h_w - h_g)$	[m^3]
d	= Diameter of the tank	[m]
h_w	= Water level	[m+NAVD88]
h_g	= Ground level	[m+NAVD88]
ρ_l	= Water density = 1025	[kg/m^3]
g	= Gravitational acceleration	[m/s^2]

	Diameter	Height	Total weight [W_{tot}]	Buoyancy [B]	Ratio	Safety against uplift
Tank	[m]	[m]	[kN]	[kN]		If Ratio>1 ; not safe
B50-14	27.7	14.6	1632.6	14446.4	8.8	not safe
A50-2	33.2	9.8	10395.7	29914.7	2.9	not safe
A50-4	33.2	9.8	17388.5	29025.6	1.7	not safe
A50-6	33.2	9.8	8116.4	28489.0	3.5	not safe
A50-8	33.2	9.8	7617.4	25719.6	3.4	not safe
A15-2	18.3	9.8	5235.6	8736.0	1.7	not safe
A15-6	18.3	9.8	2354.2	8914.3	3.8	not safe
B80-3	34.9	14.6	3243.7	24784.6	7.6	not safe

Figure 3-13 Uplift stability of floating tanks during Hurricane Isaac

The tanks shown above were not safe against uplift, which explains the floating of the tanks during hurricane Isaac.

For the damaged tanks it is also determined at which water level (with reference to NAVD88) the tanks will start floating. The tanks are assumed to be empty. The given water level equals the water height on the terminal + the ground level. The ground elevation depends on the location of the tank and is given in Figure 3-7. For the remaining part of the thesis an approximated ground elevation is used of NAVD88+1m.

	Buoyancy=empty weight	water height	water level
Tank	[kN]	[m]	[m+NAVD88]
B50-14	1509.8	0.26	2.11
A50-2	1427.7	0.17	0.94
A50-4	1427.7	0.17	1.04
A50-6	1427.7	0.17	1.11
A50-8	1427.7	0.17	1.43
A15-2	650.6	0.25	1.16
A15-6	650.6	0.25	1.09
B80-3	3206.7	0.34	2.00

Figure 3-14 Water level (m+ NAVD88) at which tanks start to float

Since no other information was provided, it is assumed that the tanks that were not damaged had enough liquid inside to prevent the tanks from lifting off their foundation.

3.5 Hurricane pre-storm damage mitigating proposals

After the severe damage caused by Katrina and Isaac, the existing pre-storm preparation procedures were improved by damage mitigation proposals. These mitigation proposals are specified for tanks, equipment preservation, personnel preservation, asset preservation and information preservation. Based on the forecasted upcoming storm and its predicted threats, the following measures can be taken for storage tanks [19]:

- Emptying of the contents of the tanks into trucks, barges, railcars or a ship → cleaning of the tank → tank should be left empty with the manways open during the storm →this can lead to physical damage to the tank, but no environmental damage.
- Filing of the tanks, also non-bolted tanks with (river) water or product to a level which will resist the flood level inside the terminal → this can lead to contamination of the product.
- Addition of ballast, anchors to the bottom of the tanks → this should prevent uplift, release of product and floor damage.

Filling of an oil tank is a measure that has already been implemented in Hawaii and a number of plants in the United States. This was also implemented at Stolthaven during Isaac, but it was not sufficiently done.

4 Steel storage tanks

The following section focuses on the description of steel storage tanks. Since the topic of the thesis is about storage tanks, it is important to know what the main components, types and general design considerations are of storage tanks. These topics are important for the structural analysis of the storage tanks with respect to hurricane loads.

Steel tanks are used for various different applications, for the storage of liquid material such as water, oil, chemical liquids, Liquid nitrogen gas (LNG) etc. and also for solid materials. In this study only liquid storage tanks are taken into account.

4.1 Classification of storage tanks

According to the nature of the use, buildings and other structures are classified into different categories. The classification is based on the consequences of environmental loads or distortions on a structure and the impact that it has on the hazard to human life. The classification varies from category 1 where structures are included with a low risk to human life at failure to category 4, where structures are included with a high risk to human life at failure of the structure.
A storage tank is placed in category 2, because it consists of enough quantities of hazardous chemicals which would cause a danger to the public if released.

4.2 Tank farms

4.2.1 Terminal operations

A tank farm, or bulk plant, is an extensive facility for receive and distribution of oil and petroleum products. It includes storage tanks, storehouses, railroad tracks, truck loading racks and other related elements. It is a network of structures and equipment for the intake, storage, transport and distribution of oil and petroleum products. Tank farms are categorized in terminal, refinery and distribution bulk plants.

- *Terminal bulk plants* are intended to transport products between vehicles or between other means of transportation (such as tankers, barges, river vessels).
- *Refinery bulk plants* can be distinguished into the category which deals with raw-material (intake, storage and preparation for refining) or the commercial type (intake, storage and shipment). In general, the raw-material and commercial bulk plants are combined into one complex which is situated at or near the refinery.
- *Distribution bulk plants* provide petroleum products directly to companies and also ship them in small containers. These plants have provisions for railroad, water-railroad, waterway, pipelines.

A lot of bulk plants operate as terminal, refinery and distribution bulk plant at the same time. [20]

The picture below gives an indication of the main facilities on the Stolthaven terminal.
[Note: This is an old picture, some tanks have not yet been built.]

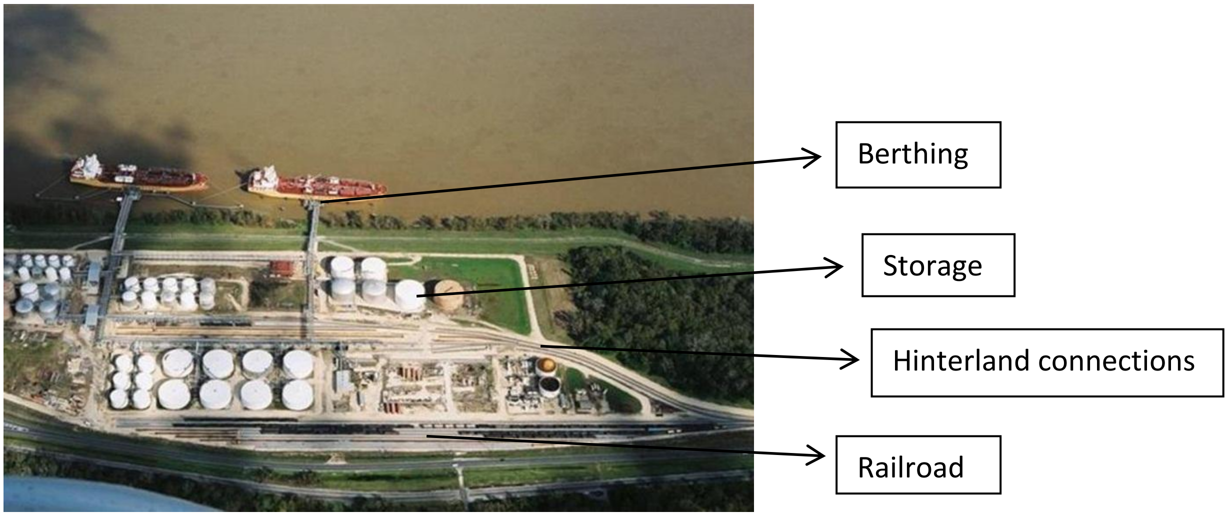


Figure 4-1 Main components of the Stolthaven terminal [photo: www.stolt-nielsen.com]

4.2.2 Secondary containment

Tanks are now being built with secondary containment. This can be a double-walled tank single-walled tank with either an open-top steel dike or a concrete vault. The secondary containment must be:

- Designed, installed and operated to prevent any migration of waste or accumulated liquid out of the system to the soil, groundwater or surface water at any time during the use of the tank system
- Capable of detecting and collecting releases and accumulated liquids until the collected material is removed

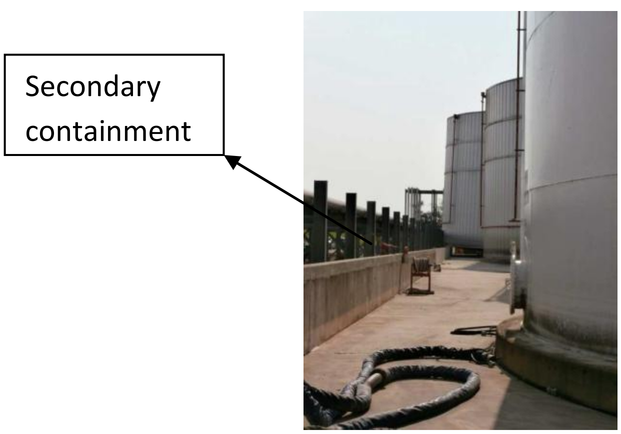


Figure 4-2 Secondary containment around storage tanks, Stolt-Nielsen terminal, New Orleans

4.3 Storage tanks

The main components of a storage tank are [6]:

- Cylindrical shell wall
  The shell wall can be made out of different types of steel. It is usually tapered, where the thickest shell course is at the bottom to take up the internal hydrostatic pressure. The wall is usually very thin. The minimum thickness of the wall is 6mm and depends on the diameter of the tank.

- Tank bottom
  Due to varying conditions, the tank bottom can have different shapes, such as a flat-bottom or a conical bottom. Corrosion is usually the most severe at the bottom of the tank and the design of the bottom can have a significant effect. Tank bottoms have thicknesses of around 10mm
- Tank roof
  This is described in the next part of the report
- Foundations
  There are different types of foundations, such as compacted soil, slab, crushed-stone ring wall, concrete ring wall and pile-supported. In this report slabs are taken into account.
- Pipeline connections

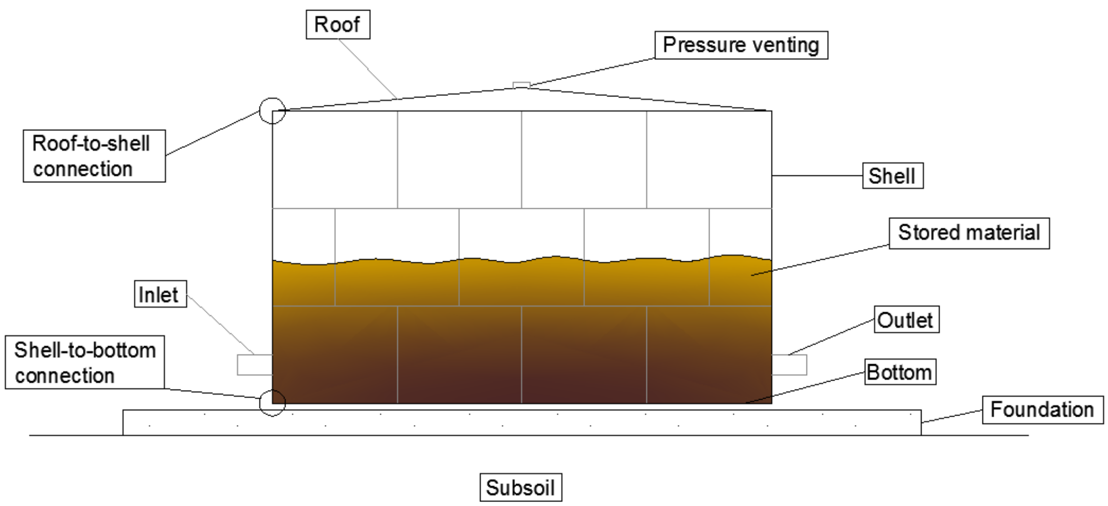


Figure 4-3 **Main components of a steel storage tank**

The primary classification of storage tanks is based on if they are above or below ground. [21] The focus in this thesis report will be on aboveground storage tanks.
Storage tanks that are constructed aboveground have the form of a metal cylindrical shell. They are made with a constant or tapered wall thickness, with or without reinforcing rings and a roof. In most cases the shape of the tank is established by the contents. The most broadly used method by codes, standards and regulations is the internal design pressure. Therefore the shape and type of tank are determined by the vapor pressure. The basic tank components are the general shape of the tank and the shape of the roof.

<u>Classification regarding internal pressure</u>
Tanks can be classified according to the level of pressure in cases where an internal pressure works on the tank during storage. The effect of the pressure depends on the size of the tank. Larger tanks have a more severe effect of pressure on the structure.

- Atmospheric tanks
  These are the most typical. They are used at an internal pressure level just above atmospheric pressure. These tanks are intended to operate between atmospheric pressure and 3.5kN/m^2 above atmospheric pressure. (1 atmosphere (standard) = 101.33kN/m^2)
- Low-pressure tanks
  These are designed for pressure levels higher than atmospheric tanks. These tanks are intended to operate between atmospheric pressure and 100kN/m^2 above atmospheric pressure.

- Pressure vessels (high-pressure tanks)
  These tanks are usually built underground

<u>Classification of tank roofs</u>
Tanks have a conical, dome or flat roof, while others have none. To avoid dangers of vapor accumulation in the upper part, floating roofs are usually provided inside the tank. Conical and flat roofs need a system of columns and rings to avoid large deflections of the roof shell, whereas usually a dome roof is self-supported. [22]
The following classification is made:

- Fixed-roof tanks
  - Cone-roof tanks
  - Umbrella-roof tanks
  - Dome-roof tanks
  - Aluminum geodesic dome-roof tanks
- Floating-roof tanks Internal floating roof (IFR)
  - External floating roof (EFR)
  - Internal floating roof (IFR)

A full description and pictures of the tank roofs can be found in Appendix II.

4.4 Design

The following section will be focused on the general design considerations of tanks. It mainly applies to atmospheric steel tanks with a flat-bottom, but many of the principles can be extended to other types of tanks.

4.4.1 General design considerations

<u>Standards</u>
The applicable code and standard is API Standard 650 (American Petroleum Institute), which also refers to other codes, such as ASCE (The American Society of Civil Engineers).

The API 650 does not provide calculation methods in case of external pressure by flooding. It only provides the following rules:

1. To prevent damage to the shell or bottom, an equivalent or higher level of liquid then the flood should be present inside the tank
2. If rule 1 is not possible, the tank and anchorage (if used) shall be designed to resist the pressure generated by the flood

<u>Site and process data</u>
Prior to detailed tank design a collection of relevant site specific data should be gathered, this includes:

- Existing geotechnical and meteorological data
- Loading conditions: wind, snow, rain and other conditions
- Physical properties of the stored liquids
- Flow ratio of liquids in and out of the tank
- Special hazards regarding the stored liquid
- Other process and local data

Materials

The two major factors in choosing what materials are acceptable for use in the tank are *corrosion* and *material compatibility*. On top of this, cost factors and establishment of the design life also play a role in choosing the required thickness.

Tank operations

The operation requirements of a tank contribute to the primary design of for example the tank capacity. Some examples are:

- It is essential to truly understand the operation process, because this can be somewhat unclear at times. A tank may be used for a certain stock at present, but can be used for another stock in the future. This change can have an effect on the corrosiveness or the specific gravity and therefore the storage capacity. It can also influence the required venting devices if changes are made in the filling and withdrawal rate, to prevent damage to tanks at unusual operations.
- Tank operations will also establish the requirements for access. For example ladders, platforms etc.
- Product purity is also an operation requirement which is less understandable. Some products tend to discolor when in contact with steel, for example glycol, which is clear and transparent in high-purity state. The tank should therefore be coated with a certain adhesive material.
- Related to the previous example, maximum cleanliness and the ability to clean the tank becomes an operation requirement and also design issue.

Liquid properties

Every stored liquid has its own problems related with its storage. Further research should be done in those specific problems. Some of those problems are:

- Many of liquid sulfur tanks have endured total collapse. This was due to molten sulfur, which caused the pressure valves to get stuck.
- Vapor pressure is also an important variable. This will lead to the required roof structure, fixed-roof or floating-roof. Flammability and flash point manage spacing and layout requirements and also determine the basic tank design.

Sizing considerations and tank proportioning

Determining the optimal size of a tank can be quite difficult. Normally, the design capacity must be equal to the maximum desired inventory plus the unusable volume that remains in the bottom of the tank. Inventory represents working capital plus operating expense which both reduce net profitability. Some analysis can be made in order to optimize the required design capacity. One of those is the consideration between one large tank and several tanks for the same kind of fluid. Even though it might appear that one large tank should be used, there are various reasons to consider several tanks:

- The size of one tank can limit the use of one tank. Even though the largest steel tanks are about 1 million barrel (almost 160,000 m^3), the limits for other types of tanks, like aluminum can be much smaller.
- A sufficient number of tanks on site can cover the inventory capacity of a tank which is out-of-service.
- Check tanks can be required to ensure quality control.
- Dedicated tanks can be needed
- Risk assessment can pin point that the probability of loss insists storage of a certain material in several separated location within the plant.

<u>Operational considerations:</u>

- *Nominal capacity*: the total volume of the shell to the very top, also called *gross capacity*
- *Operating capacity:* the usable volume of the tank, which is limited at the top by the safe oil height or maximum operating level and by the low pump-out height
- *Safe oil height/ maximum operating capacity:* the level at which the liquid is not allowed to rise. This level depends on the type of roof applied.
- *Unavailable inventory (minimum fill level):* reserve which cannot be pumped out of the tank without special changes to the operations of the tank. Low pump-out is often set by the nozzle elevations.

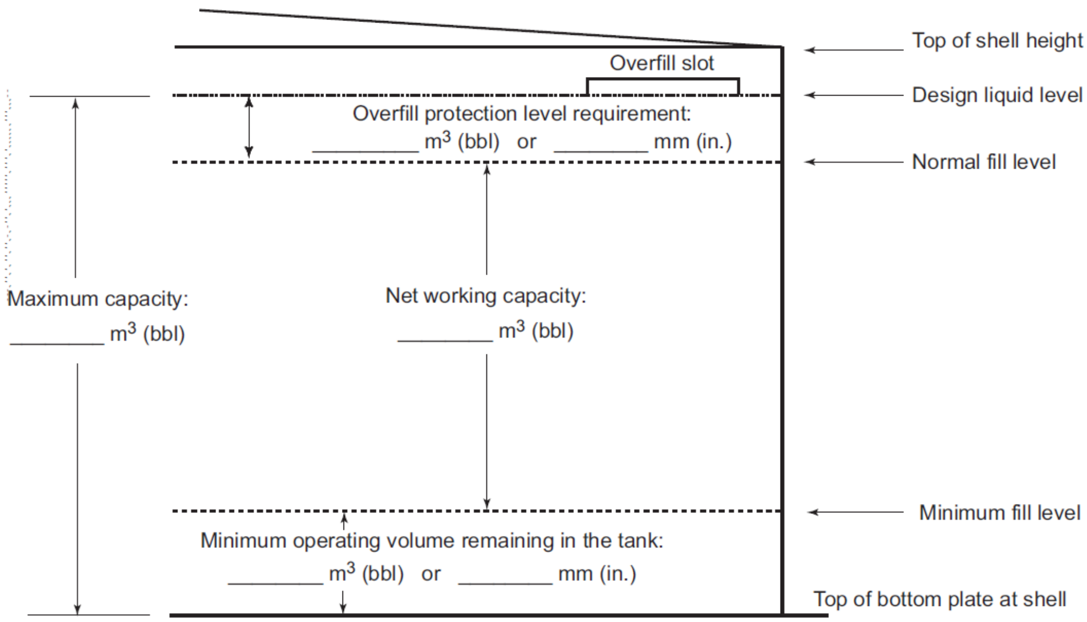


Figure 4-4 Storage tank with different fill levels (bbl = barrel) [23]

The height-to-diameter ratio (also called aspect ratio) is established by various factors. Some of them are:

- *Seismic considerations*: Since it is preferred to use unanchored tanks when possible, tank proportions in areas of high seismicity favor low height-to-diameter ratios.
- High land costs or limited space favors tanks with a greater height-to-diameter ratio.
- Process considerations often affect optimal height-to-diameter ratios. For example, for mixing, variation in the height can cause different levels of mixing and power requirements.
- A low height-to-diameter ratio tends to increase the percentage of unusable inventory.
- Low bearing capacity of the soil will limit the tank height.
- Use of gravity flow may determine the required height of the tank.
- Costs and material utilization are relevant.

<u>Internal pressure</u>

The internal pressure is the difference in pressure between the inside of a tank, or its vapor space, and the local barometric pressure or atmospheric pressure. The pressure is called vacuum if it is negative. The pressure is measured at the top of the liquid in the tank. The design minimum internal pressure is 1.2kN/m^2 (25psf).

Venting
Storage tanks with flammable liquids should be provided with venting. Tank breathing is the vaporization that occurs due to normal thermal changes.

Life span
Aboveground steel tanks are designed to last 20 to 30 years. This service life can be reduced if no proper care is taken to corrosion. The most sensitive part of the tank, which can bring the service life of a tank to only 5 years, is the tank bottom. If the bottom is not properly protected from corrosion, water and other reactive contents, it can accumulate in the tank bottom and corrugate in the steel plates from inside. Corrosives present in the soil can also attack the external service of the bottom. [24]

Hydrotest
A hydrotest, hydrostatic test, is done to test the strength of the storage tank and to detect leaks. This is done to every tank before putting it to use.

4.4.2 Structural design considerations

If a tank is loaded with an external or internal pressure, stresses will occur in the material.
This section describes some considerations to be taken into account at the design of a storage tank.

Internal pressure - edge disturbance
For this part of literature reference is made to [25] & [26].

A cylindrical tank consisting of liquid inside shall be subjected to a edge disturbance at the bottom of the plate. A cylindrical shell will tend to expand if an internal load or pressure is present, but cannot do this at the base, because the tank wall is connected to the bottom plate of the tank. Displacement at the bottom of the tank is zero and higher moments will occur in this area. The constraining causes an edge disturbance in the shell.

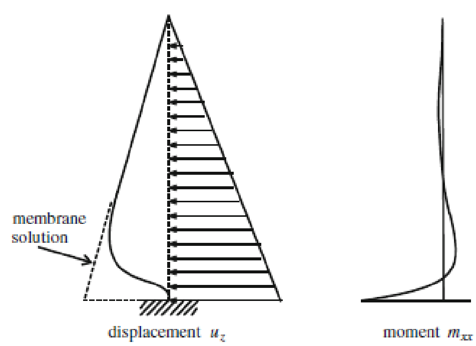


Figure 4-5 Edge disturbance at the bottom of the tank due to internal load

External pressure
External pressure on the tank can be generated by different forces, such as wind and hydrostatic pressure. If an external pressure is present on the shell, the shell wall will be in compression and will tend to buckle if the pressure is larger than the buckling strength of the structure.

- *Wind pressure*

If the wind pressure is larger than the buckling strength of the structure the tank wall can be provided with stiffeners.

Formulas for the determination of the buckling strength, required stiffeners etc. can be found in API Standard 650. These formulas are given in 0.

- *Hydrostatic pressure*

Apart from external pressure originating from wind, external hydrostatic pressure can also be present if the tank is surrounded by water. This pressure can also cause instability of the cylindrical shell, just like wind pressure. API standards 650 does not provide formulas to calculate the effects of hydrostatic pressure on the cylindrical shell of the tank.
The external hydrostatic pressure will also cause an edge disturbance in the lower part of the shell if the tank is empty.

4.5 Impact of the height/diameter (h/d) ratio of steel storage tanks on loads

This section describes the influences of the height-to-diameter (h/d) ratio on various aspects related to the storage tank. The relations between the weight of the structure, including the main contributing parts, and height-to-diameter ratios are presented. A link is also made between these ratios and the loads working on the structure.
In this section the tank is seen as 1 rigid structure and conclusions will be made with respect to the whole structure and not on different parts.
The aim of this section is to get an indication on the magnitude of the mentioned forces with regard to the weight of the tank and the h/d ratios.

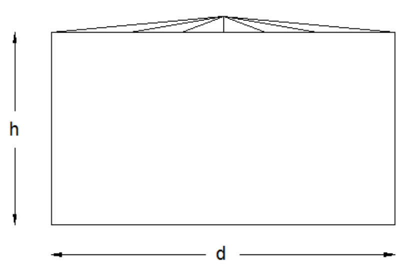


Figure 4-6 Diameter and height of a storage tank

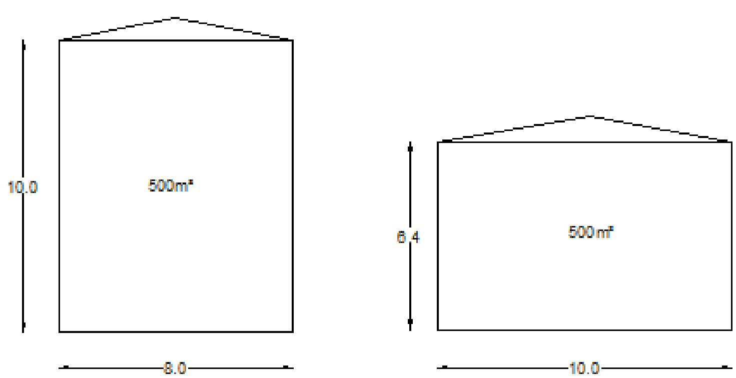


Figure 4-7 Example of 2 different h/d ratios for a 500m^3 storage tank

<u>Weight vs h/d ratios</u>

The volume of a tank depends on the capacity required at the certain storage facility. This is different in every situation, therefore no basic tank capacity can be appointed. To find the relationships the following 4 storage tank structures are used. These are existing tanks on the Stolt-Nielson terminal from which drawings and design specifications are known. The calculations are done assuming an empty tank.

In practice there is a minimum fill level of liquid inside the tank (see Figure 4-4). This level depends on the nozzle height.

		Drawing	Diameter (d) [m]	Height (h) [m]	h/d ratio	Volume [m3]
Tank 1	Cone roof tank	A50-5	33.2	9.8	0.3	7960
Tank 2	Cone roof tank	A15-1	18.3	9.8	0.5	650
Tank 3	Cone roof tank with floating roof inside	B50-9	27.7	14.6	0.5	7980
Tank 4	Umbrella roof tank	D6-4	11.9	12.1	1.0	1285

Table 4-1 Input data of 4 tanks

Note: The figure below presents the acting buoyancy load on the bottom plate of the structure. The weight of the shell and the roof acts as a line load around the circumference of the tank. This will have little to no contribution to the counterbalance of the buoyancy load. The resisting load is on the opposite side of the bottom plate, this is the weight of the bottom plate and the stored materials acting on the bottom plate.
Only in this part of the report the total weight of the structure is compared to the buoyancy load. In the remainder of the report only the bottom plate is taken into account.

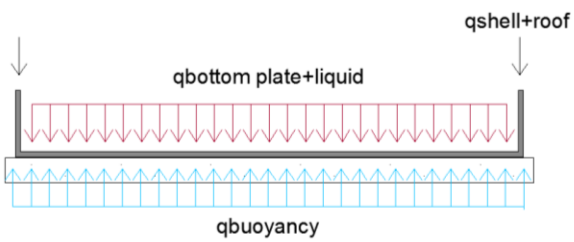


Figure 4-8 Buoyancy load and counterbalance acting on the bottom plate of the storage tank

In the following section calculations with large h/d ratios are also considered. This is only done to get an indication of how high the buoyancy load is with respect to the weight of the structure. In practice most storage tanks have a h/d ratio of approximately 1 or smaller.

Roofs

There are different types of roofs. If we take a look into the cone roof, the following can be concluded: The volume of steel of the roof and also the weight depends on the height of the roof and the radius, which is half of the diameter. This is a rough approximation, because the roof also contains stiffeners. The area of the cone is:

$$A_{roof} = \pi r . \sqrt{h^2 + r^2}$$

with:

h	= height of the roof	[m]
r	= radius of the tank	[m]

With a constant slope, the height of the roof varies if the diameter changes. The weight of the roof is independent of the height of the tank. The weight of the roof would stay the same if a constant diameter is held even though different heights are applied.

Therefore the weight of the roof cannot be related to a certain h/d ratio. The figure below shows the relation between the tank radius and the weight of the roof, assuming a constant slope and thickness of the roof for all radiuses.

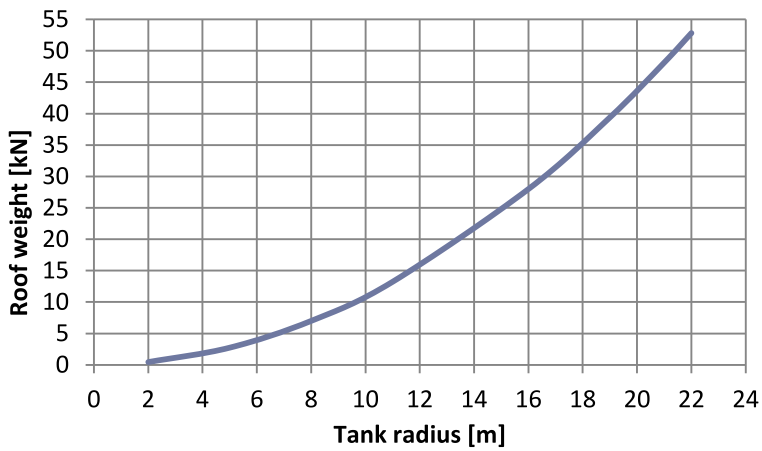


Figure 4-9 Relation between the radius of the tank and the weight of the roof

This also applies for the dome roof. The area of the dome is: $A_{roof} = 2\pi r h_{roof}$

Weight of the structure

The weight of the structure consists of the weight of the shell, roof and bottom plate. This weight depends on the size of the cylindrical shell and the roof. One specific h/d ratio cannot be related to one constant weight, because a certain h/d ratio can have different volume capacities. For example, h/d = 0.5 can have a height of 15m and a diameter of 30m or a height of 10m and a diameter of 20m. The latter has a smaller capacity and shall have a smaller weight.

For low h/d ratios, the weight primarily depends on the weight of the roof. On the other hand, for higher h/d ratios, the weight primarily depends on the weight of the shell. Tanks with different h/d ratios and different capacities, the weight can be illustrated as follows:

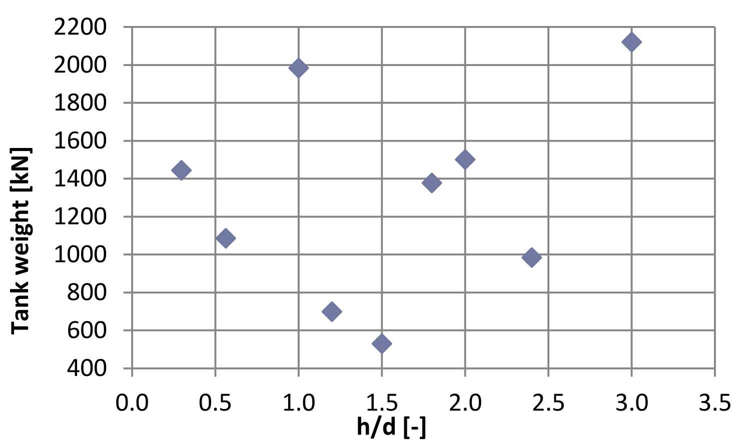


Figure 4-10 Weight of a tank at different h/d ratios and volumes

This section focuses on the relation between different h/d ratios and the weight of the steel storage tank. The relation of the different parts of the tank, contributing to the weight, is also studied.

To see if the same relation applies for other tanks, 4 different tanks are used. The calculations are done with available drawings and specifications of 4 existing tanks (Table 4-1).

For an impression on how the weight of a cylindrical steel storage tank behaves at different h/d ratios, the volume of the tank is held constant.

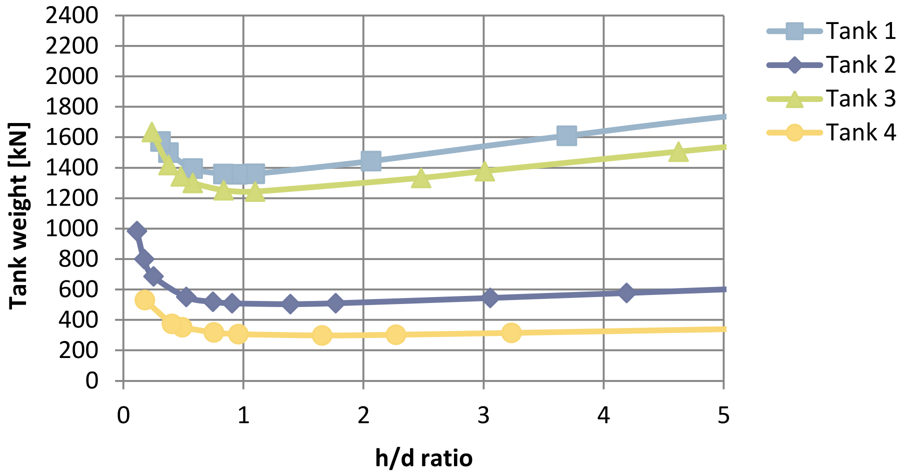


Figure 4-11 Relation between the tank weight and different h/d ratios of the 4 types of tanks.

All 4 tanks show the same relation between the ratios and the weight of the structure.
At low h/d ratios (between 0 and 1) the diameter is large and becomes smaller as the ratio increases. The total weight decreases, because the weight of the main components here (bottom and roof) depend on the diameter and will therefore decrease with increasing ratio. This can be seen in Figure 4-12. The calculated weight of the structure is compared with the actual weight specified on the tank drawings. The calculated weight is almost equal to the weight on the drawings, so the relations in the graphs are assumed to be correct.

The weight of the contributing parts (shell, bottom and roof) is calculated at varying h/d ratios. The figure below presents the weight of the different parts against the h/d ratios for Tank 2.

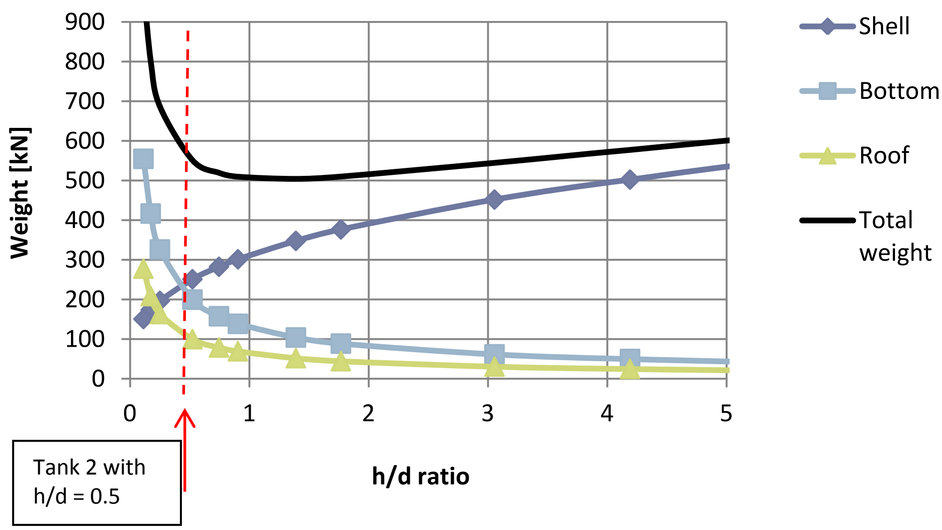


Figure 4-12 **Total weight of the tank with contributing parts of shell, bottom and roof for different h/d ratios (≤10) of Tank 2**

Not only tank 2, but the other tanks had the same relation as shown above. The weight of the shell relies on the diameter and height of the cylindrical shell, which is why this increases at higher h/d ratios. The bottom and the roof do not depend on the height of the cylindrical shell, therefore the weight of these parts decreases at higher h/d ratios.

Forces vs. h/d ratios

To get an indication of the influence of the diameter and the height of the tank on the working forces, a calculation is done where the volume is also held constant. Working forces are then determined with varying width of the tank.
All 4 tanks show the same relation to the h/d ratios, but with other values. The results of Tank 1 are presented below.
The graph below shows the wind and buoyancy force according to different h/d ratios for Tank 1. [see Appendix VII B for the equations of wind and buoyancy]

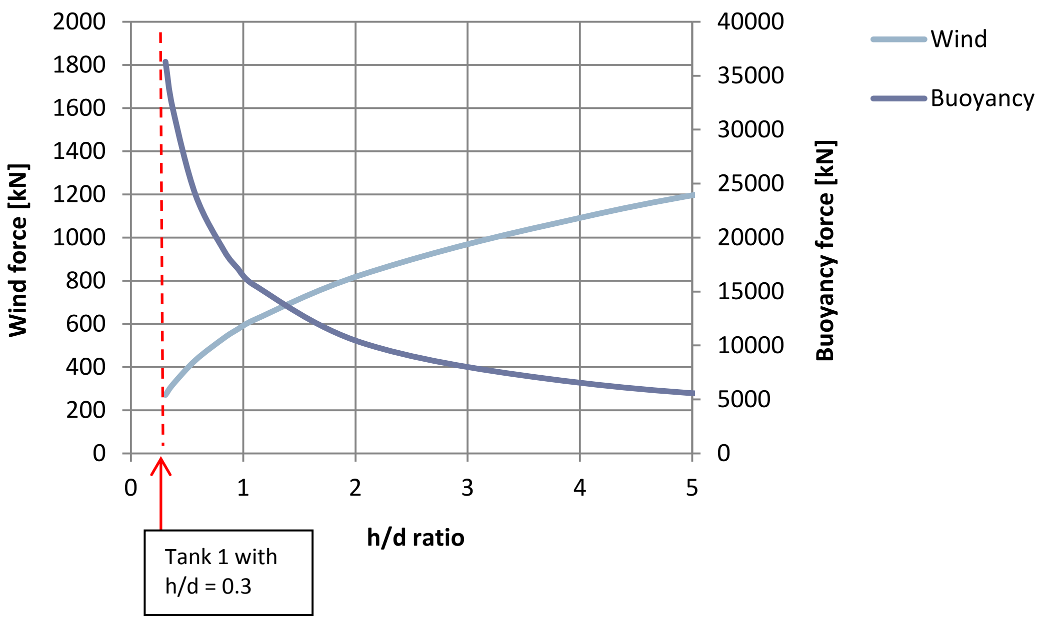


Figure 4-13 **Wind and buoyancy load for h/d variations of Tank 1**

Tank 1 has an h/d ratio of 0.3. According to the graph h/d ratio of 0.3 has a high buoyancy load and a low wind load. That is because a low h/d ratio has a large diameter and a small height.
The following figure displays the weight of the tank and the buoyancy force in one graph. If the tank weight is equal to the buoyancy load, the tank cannot be lifted off its foundation.

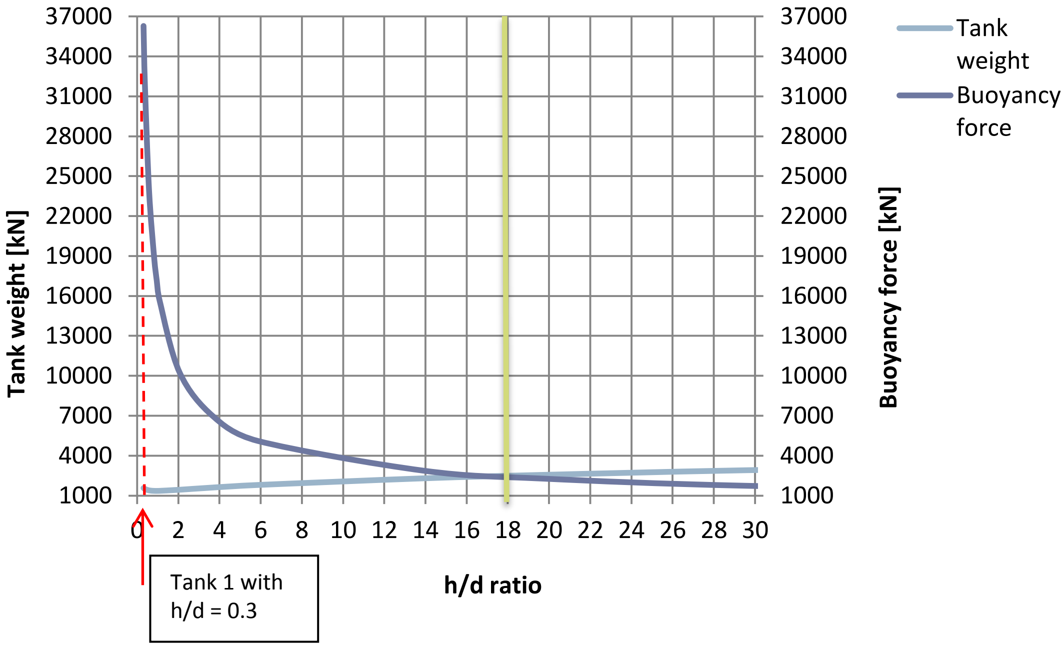


Figure 4-14 **Tank weight and buoyancy load for h/d variations of Tank 1. From the green line on to the left, the weight of the tank is larger than the buoyancy force and no additional weight is needed from this point on to prevent uplift.**

The next chart displays the weight difference between the buoyancy force and the weight of the tank. The values are presented in percentages of the tank weight.

For example: h/d = 4: Tank weight = 1641 kN
Weight difference in % of tank weight = 300% = 4914 kN

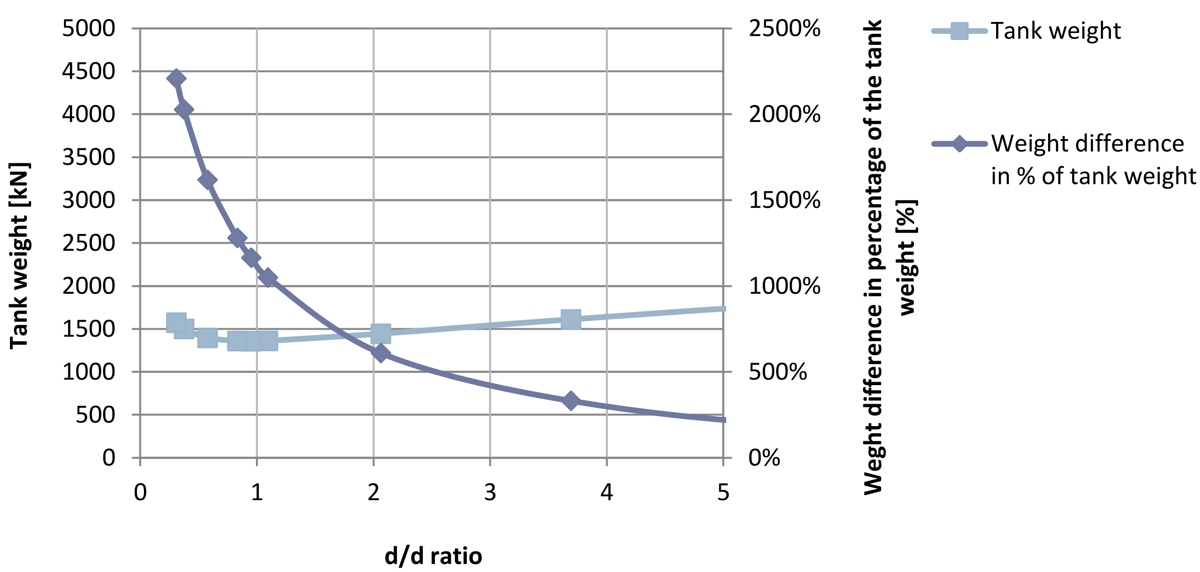


Figure 4-15 **Weight difference in percentages of the tank weight for h/d variations of Tank 1**

Conclusion

Figure 4-15 shows an enormous weight difference between the original weight of the tank and the weight of the tank as it should be for the prevention of uplift of the tank. For a tank with a volume of approximately 8000m^3 (like tank 1), the weight of the tank is only favorable for h/d ratios larger than 18 (As can be seen by the green line in Figure 4-14, the weight of the tank is equal to or greater than the buoyancy force for h/d ratios of approximately 18 or higher). For tank 2 (with a smaller volume) this value is higher, 24.
However, this means a tank with a h/d ratio of 18 can have a diameter of 2m and a height of 36m.
For such a slender structure the wind force will be very high and the pressure on the ground will be large as well.

4.6 Loads

The loads are based on the 1/100 year design water level E_{sw} of NAVD88+4.6m (taking into account a +1m ground elevation).
Apart from wind, the tanks will then be loaded with an extra external pressure from the flood, like hydrostatic pressure and wave pressure.

The table below presents a summary of the working loads on the structure. These loads are used in calculations in the remainder of the report.

Self weight	Self weight includes weight of: ▪ storage tank ▪ roof ▪ liquid
Internal loads	
Hydrostatic pressure (ref.: Appendix VII A)	Liquid inside a tank generates hydrostatic pressure against the shell wall and pressure on the bottom plate of the tank. The hoop or circumferential stress $\sigma_H$is the most important one, because it is the stress in the shell which resists the cracking of the shell due to the applied pressure inside the tank.
External loads	
Wind (ref.: AppendixVII B)	External pressure due to wind can be extremely damaging to a tank, because generally the surface area is large and develops large forces. The API Standard 650 states that for the design velocity of wind, a basic wind speed should be used which depends on the location of the project area. For Stolthaven, which is located in Louisiana, this is ≈ 259 km/h. (see Figure 3-5 in chapter 3.2.2). The corresponding wind pressure is: 1.6 kN/m2 [Note: For the design of storage tanks with external pressure (vacuum) as normal operating condition, the corresponding wind pressure is higher (see 0)]
Flood-induced loading (ref.: Appendix VII B)	The 1/100 year design flood depth $\mathbf{d}_s$is considered, which is approx 4.6m. Design flood velocity V is 4.6m/s. The hydrostatic load $\boldsymbol{F_{h,static}}$ against the tank depends on $\boldsymbol{d_s}$: → **For** $\boldsymbol{d_s = 4.6m}$: $\boldsymbol{F_{h,static} \approx 46\, {}^{kN}/_{m^2}}$ The buoyancy load ($\boldsymbol{F_{v,static}}$) depends on the displaced volume of the tank and varies with the weight and diameter of the tank. The hydrodynamic loads are the result of the flowing water against and around the structural element.

	The breaking wave load, where the maximum wave pressure against the tank is determined with a rule of thumb. The impact loads are loads resulting from any object transported by floodwater striking against buildings and other structures. (not taken into account for this case)

Table 4-2 Loads working on the tank

Load combinations

The load combinations given are in accordance with the allowable stress design (ASD method) (The coastal construction manual uses these combinations). The combinations are considered in Zone V and Coastal A Zone (Section 2.4.1 of ASCE 7-10). In the calculations, the load combination which has the most unfavorable effect will be considered and will be elaborated.
(see Appendix VII C for more information)

5 Problem statement: "How can storage tank damage and chemical spill best be prevented during a flood?"

The following section describes the problem statement and aim of the project

Stolt-Nielson, New Orleans and other chemical terminals have been victims of severe damage and chemical spills during Hurricane Katrina and Isaac. Overtopping/overflowing of the GoM Levee, which safety level is very low (see section 3.1) caused the terminal to flood.
Stolt-Nielson considers expanding the current terminal and is concerned that the previous disaster will repeat itself and will also affect the new terminal.
The current terminal is now being protected from floods with a floodwall.

A 1/100y flood height will cause approximately 5m water on the terminal. This means, the tanks will be subjected to flood loads. The main form of damage at the current Stolt-Nielson terminal was damage due to uplift and floating of storage tanks. The uplift and floatation were caused by the large buoyancy load at the bottom of the tank in combination with the light compensating weight. A flooded terminal together with large tank diameters and a lack of counter weight caused tanks to lift off their foundation at a small excess of water.
This uplift caused rupture to the connected pipelines and therefore spill of chemicals. Floating tanks can cause damage to other tanks and can also to themselves if run into other tanks or objects.
This event caused economical, emotional and ecological devastation.

Tanks are lifted off their foundation due to the lack of counter weight. Since the buoyancy acts on the area of the bottom plate, this area should have sufficient counter weight or should be able to transfer these high forces.

The problem can be formulated as: "How can storage tank damage and chemical spill best be prevented during a flood?"

In the remainder of the report the focus is on different alternative solutions which should be applicable for the still-to-be-built new terminal next to the present terminal. An alternative study is presented where then a selection is made of the alternatives that most likely seem to be the most beneficial ones. These are further worked out in detail.
Finally a simplified cost-benefit-analysis (CBA) is done to compare the chosen alternatives to a basic solution as the floodwall.

Phase 2 Alternative study and concept design

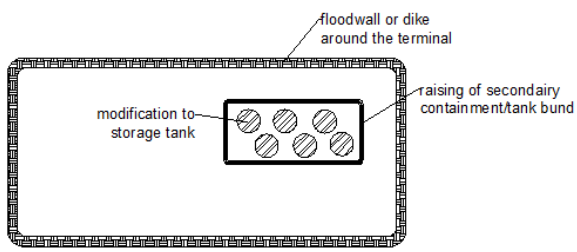
6 Alternatives for protecting the tank from flood loads

The following section is based on a description of different alternatives which are applicable for the still-to-be-built new terminal next to the present terminal.
These alternatives should provide a solution for the previously stated problem in Chapter 5. The aim is to evaluate the different solutions and finally make a selection according to certain selection criteria (more of this in the paragraph 6.2).

Currently, the present terminal in New-Orleans is being protected by the construction of a floodwall around the terminal. The construction of a flood wall was less expensive than a levee.
Due to the fact that the new terminal is still to be built, other site specific alternatives are also approached.

6.1 Alternatives

The problem can be approached in different ways:
- Alternative 0: Do nothing
- Alternative 1: Raise of Backlevee/constructing a new levee
- Alternative 2: Constructing a flood wall
- Alternative 3: Raising of the secondary containment bunds
- Alternative 4: Tank design modifications
- Alternative 5: Tank operations/management



Alternative 0: Do nothing

One approach is to do nothing and to accept the damage. At some cases it is considered to allow the damage to happen, because in those cases it is less expensive to repair the damage then to provide protection according to the conditions. In this case damage to the storage tanks causes spill of chemicals. This spill can be disastrous to the environment and toxic to humans and animals. This damage, the environmental damage, cannot be replaced and is therefore not acceptable. For this reason alternative 0 cannot be applied in this case.

Alternative 1: Raise of the Backlevee/ constructing a new levee

Chapter 3.1 describes the existing levee system around the terminal. The GoM levee (Back levee) stretches all the way behind the new terminal, a solution can be to raise this levee. Unfortunately this raise should be over the entire length of the GoM levee (see blue line in **Error! Reference source not found.**) and will be very expensive.

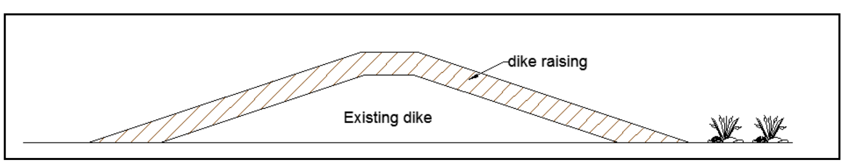


Figure 6-1 Raising of the dike

Another option would be to construct a new levee just around the terminal site. The risk of flooding will be reduced and therefore also the risk of damage to the tanks. The levee should have a safety level of about 1/100, which is the current safety level of the Mississippi levee. Usually levees are built

out of clay or sand or a combination of the two. To resist wave attack, the levee is provided with a bed protection of geotextile and rocks.
To provide adequate stability, the levee requires a wide base and therefore a lot of space. The slopes of the dike should be around 1:2 or 1:3. The flood wall around the present terminal has a height of 8m. For a levee, the same design height would be required. If a levee has to be 8m high and has a slope of 1:3, the width of the base should be around 50m. This will occupy a great quantity of land. Sometimes land has to be acquired from landowners to facilitate for the width of the dike. In this case that would not be a problem, because the land is still deserted.
According to the bore logs of Stolthaven, the quality of clay present in the area is low and is not suitable for a levee. This can also be assumed for the areas surrounding Stolthaven. Clay has to be excavated from another area and transported to the construction site or other area for the riping-process. Due to the large settlements of clay, a large amount of clay is needed. Another possibility would also be to perform soil excavation. If the levee is built out of sand, it has to be transported to location, because the area of Stolthaven does not consist of sand. An offsite location should be found where sand can be excavated and transported to the terminal site.

Advantages	**Disadvantages**
Easy to construct	Wide base; space consuming
Easy to adjust/enlarge profile	Low quality of the material nearby
	Slow construction
	Frequent inspection
	Expensive, large amount of clay needed

Alternative 2: Constructing a flood wall

The construction of the flood wall will also reduce the risk of flooding and damage to the tanks. A study is done by Royal HaskoningDHV for the construction of a flood wall for the existing terminal. This method can also be applied to the expanded new terminal.
Flood walls are made to protect the hinterland from heavy wave attack and flooding. They are mostly constructed of concrete (such as the T-wall), steel sheet piles or timber cribs. The T-wall exists of a concrete horizontal slab and concrete vertical wall with a rigid connection. The steel sheet piles are interlocking, thin sheet piles driven deeply into the ground. Most of the time, the sheet pile is backfilled and anchored. The sheet piles should be sufficiently long to provide for seepage control in the underlying layers of the flood wall.

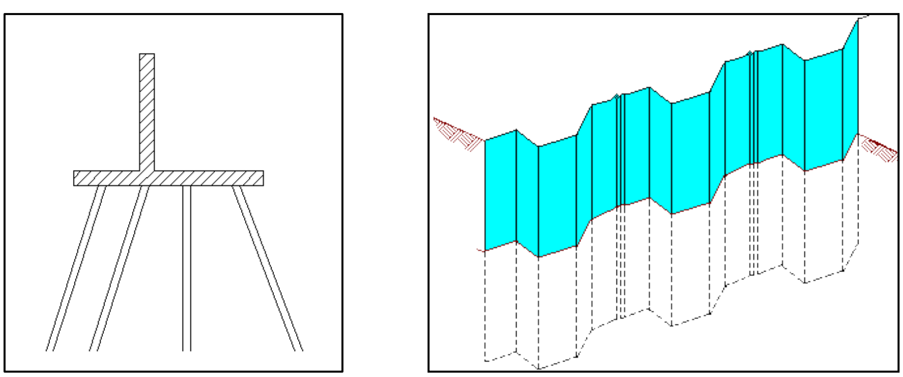


Figure 6-2 T-wall and Steel sheet piles

The floodwall should be provided with large gates, to have a connection between the terminal and the outside. In this way the chemicals can be transported from and to the terminal. When there is a high risk of flooding, these doors should be closed down.

Figure 6-3 Example of Caernarvon Flood gate [[Report: M903703_site_investigation_stolt_nielsen]

Advantages	**Disadvantages**
Small base	Expensive (less than a dike, reference is made to the current terminal)
Less construction time due to prefabrication	Foundation height depends on subsoil characteristics
Usually prefabricated	

In contrary to a levee, a flood wall does not need a wide base and therefore does not require that much space, because it is usually constructed with concrete or steel. The flood wall is usually prefabricated in segments and transported to the construction site, where it will be installed. The costs for material, transportation and installation will be high. The construction time of the flood wall would be less than the levee, because of prefabrication methods. The segments of the wall can be built in advance and does not rely on the installation time. Floodwalls also require a routine inspection to make sure there are no signs of cracking, spalling or scour.

Alternative 3: Raising of the secondary containment bunds

Usually the secondary containment is placed around a couple of tanks to protect the outer area from chemicals is case a spill would occur (see chapter4.2.2). These containments can be heightened to reach the certain safety level in case a 1/100y flood height occurs. This will be like the dike or floodwall alternative, but instead, one containment would be protecting a couple of tanks and not the whole area. One terminal would have a number of these. This alternative is not efficient for the terminal operations; pipelines etc. would be laid across the wall, almost similar to the current situation at the MR Levee (see Figure 3-9). Also storage tank maintenance would be very difficult to carry out

Advantages	**Disadvantages**
Tanks are contained in one area and damage can be reduced	Expensive
Chemical spill is contained inside one containment bund	A lot of space required
	Not efficient for the terminal operations and maintenance

Alternative 4: Tank design modifications

The risk of damage to storage tanks can also be reduced by taking measures on the terminal site. Since the terminal is not yet built, structural modifications to the tanks can be an option. A structural improvement to the storage tank design can lead to stability of the tank and a reduction of damage. For example, an improvement can be made to the storage tanks with regard to the resistance against flooding.

Advantages	**Disadvantages**
Little space required	"innovation risk"
Improvement to tank design	
Construction time	
Flexibility	

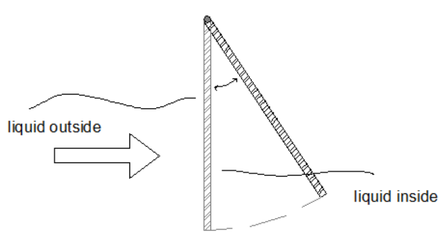
Measures on the terminal site will not require an extensive amount of space, since this is the opposite for a dike. They can be improvements made to the storage tanks to resist the forces which are generated by a hurricane and the flood. Because this solution is for the new reclaimed land, improvements can be made in the design of the tank. The building method can therefore also be improved, by building the new tanks not all at once, but in series of a number of periods. The costs for this improvement would be an addition to the original costs of a tank. It is however not familiar if such a technology has already been implemented. Here, the "innovation risk" plays a role with respect to proven technology.

Alternative 5: Tank operations/management

As mentioned in paragraph 3.5, one of the pre-storm mitigating proposals that were presented after the damage of Hurricane Katrina and Isaac was the filling of tanks with a product or water to resist the upward force.

For a 1/100y flood, the design water level is 5.6m NAVD88. Assuming a ground elevation of +1m and a specific weight of the product of lower than 10KN/m^3, the height of liquid inside the tank should be higher than 4.6m to provide enough weight. The height of the tanks varies somewhere between 5m and 15m. The average height of a storage tank can be set at approximately 10m. This means that 50% of every tank should be filled with stored material. Therefore a large inventory should be available, at least in the hurricane period.

A valve can be constructed in the tank wall to let water inside in case the tank is surrounded by water. The valve opens if the water outside the tank is high enough. If the tank is filled with water, this can be sea or river water, it can contaminate the product which is already inside the tank.



Advantages	**Disadvantages**
Cost effective	Unpredictability of available required inventory
	Contamination of chemicals if filled with water
	Unreliable

6.2 Selection

The selected alternative(s) should be applicable to the new terminal.
Looking closely into the given alternatives, 2 groups can be distinguished. Group 1 (alternatives 1 thru 3) focuses on the protection of the storage tanks on a major scale, the whole terminal or a couple of tanks altogether. Whereas the group 2 (alternatives 4 and 5) focuses on the protection of each storage tank separately.

To make a selection these two groups are considered. Protect the entire (or parts of) the terminal against flooding or allow the terminal to get flooded and make the storage tanks flood proof.
A selection based on the costs is in this phase difficult to make, because the costs for implementing alternative 4 (Tank design modifications) are unknown.

Group 2 (alternatives 4 and 5) allows the site to be completely open to the outside. No floodgates are required and pipelines do not have to cross levees or walls. The tanks can be built in several periods for example 40 tanks can be built in 5 years, with 8 tanks per year. Alternatives 4 and 5 can lead to a stable & strong enough structure, to prevent damage during a hurricane and flood. The tank is already flood proof and the terminal does not have to be protected. Tanks can be put into operation after construction. For these reasons group 2 with alternative 4: *Tank design modifications* and alternative 5: *Tank operations/management* will be further elaborated.

7 Possible storage tank solutions

For the purpose of this thesis, only storage tanks with a conical roof, without internal columns will be further elaborated.

This chapter presents some methods that can provide a solution to storage tanks. They are improvements which are applied to a standard tank design or the area where the tanks have to be built. They can lead to a recommendation for an optimal solution that can be applied to the new Stolthaven terminal. These methods have to be analyzed to see if they satisfy the conditions.

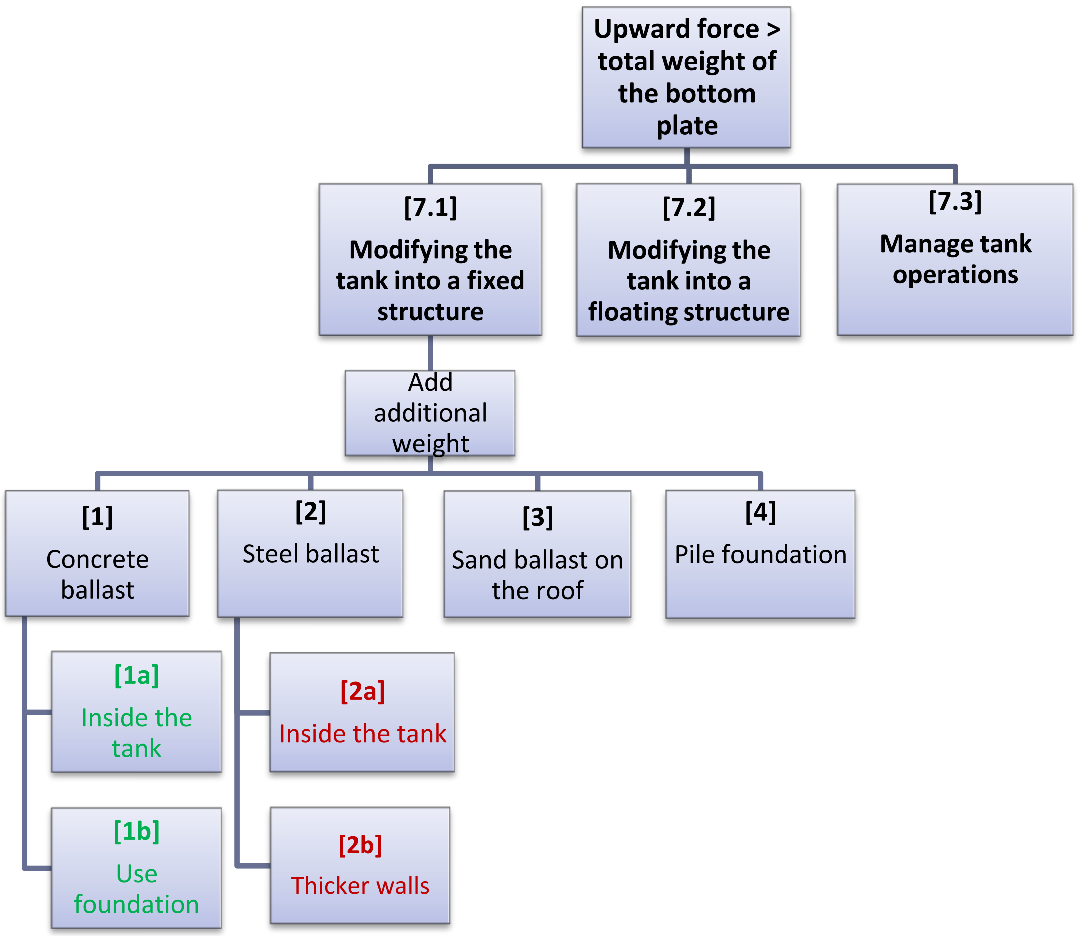


Figure 7-1 Scheme of possible solutions in case the upward force is larger than the weight of the tank

If the terminal is subject to flooding, the following happens to the tank:

- An upward force is present at the bottom of the tank and an external hydrostatic pressure is present against the tank wall.
- Due to this upward force the following can be distinguished:
    - The tank has enough weight to compensate for the upward force. In this case the tank stays on its place. Due to the upward force, the bottom of the tank can still be damaged if the counter weight of the liquid inside is not sufficient. This why the weight of the bottom plate should be considered and not the total weight of the structure. (Assuming a normal bottom thickness of approximately 8mm).

- The tank has not enough weight to compensate for the upward force. In this case the tank will float. The bottom of the tank can here also be damaged from the upward force.

In the following section a detailed explanation is given of the scheme above.
The 1/100y flood height is used, this is NAVD88 +5.6m. Using the ground elevation of 1m. The flood height on the terminal is assumed to be 4.6m.

7.1 Modifications for a fixed structure

Add sufficient weight to the structure to counteract the upward load. The structure stays on its position if the downward force ≥ upward force. This additional weight should be applied on the bottom plate of the structure. The weight of the shell and the roof do not contribute to the counterbalance. This is mentioned in Figure 4-8.

For example:

Tank 1 (Table 4-1): Diameter (d) = 33.2m
Height (H) = 9.8m
h/d = 0.3
h_w (water height outside the tank) = 4.6m
Volume = 7960kN
Empty tank weight ≈ 1600kN
Weight of the bottom plate = 680kN
Buoyancy load = ${}^{1}\!/_{4} \cdot \pi \cdot 33.2^2 \cdot 4.6 \cdot 10 \approx 39800\text{kN}$
Additional weight to be applied = 39120kN
≈ 2500% of the empty weight of the structure

The principle here is to apply additional ballast material to make the tank heavy enough for it to be unable to float when an excess of water is present at the terminal. The weight can be applied in one of (but not limited to) the following manners:

[1] Concrete ballast

An option is to apply concrete as ballast weight to gain sufficient weight. Two solutions can be applied here. Use concrete inside the tank for ballast or use the concrete foundation as ballast.

[1a] Inside the tank

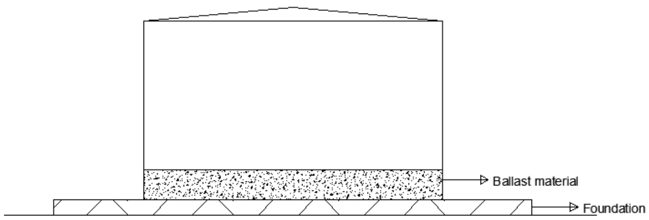


Figure 7-2 Concrete as ballast material used inside the tank

Interesting here to know is the amount of concrete needed inside the tank. How much is this going to affect the capacity of the tank? Furthermore the connection between the steel bottom plate and the concrete ballast should be sufficient to guarantee the collaboration between the steel and the concrete. The concrete ballast should be provided with reinforcement to take on the tensile forces. The interface between the concrete layer and the liquid inside should be sealed with a liner or coating or chemical resistant concrete should be used, to avoid seepage through the concrete and the potential of corrosion of the reinforcement. An option is to apply a steel plate on top of the concrete.

Required amount of concrete

The first step here is to get an indication of the required amount of concrete needed. This amount is calculated for tank 1 (see previous page). With a concrete specific weight of 25kN/m^3 the additional weight corresponds to a concrete height of approximately :

$$\text{Concrete height} = \frac{39120}{1/4 \cdot \pi \cdot 33.2^2 \cdot 25} \approx 1.8\text{m} \quad \text{(see previous page for values)}$$

The concrete can be applied in different ways inside the tank:

I. Concrete slab

The concrete ballast can be provided by pouring concrete in the tank. Because of the large thickness, reinforcement is also required.

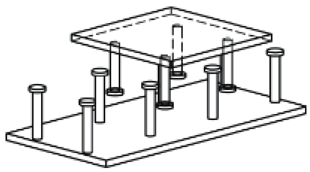
II. Combination steel and concrete slab

The concrete ballast can also be provided with a certain structural element. A structure that can be used is the steel-concrete-steel (SCS) sandwich slab[4]. It is a special form of sandwich structure and exists of steel face plates and a concrete core, which are connected by shear connectors.

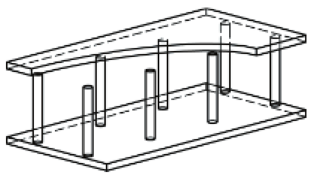
[4] An alternative design of steel-concrete-steal sandwich beam [paper]

This system was at first designed for the use in submerged tube tunnels, but is also applicable to nuclear containment, gas and liquid retaining structures and blast resistant shelters[5].

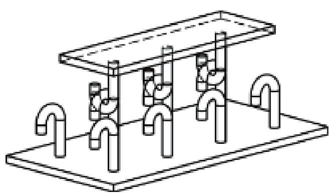
The purpose is that the steel plates and the concrete resist the external load with the use of shear connectors. The latest construction forms of SCS sandwich structures are given below. The difference between them is the layout of the shear connectors



(1) Double-skin sandwich construction (DSC)



(2) Bi-Steel sandwich construction (Bi-Steel)



(3) Alternative sandwich construction

Figure 7-3 Construction layouts of the SCS sandwich structure

<u>Basic structural elements</u>

The sandwich system basically consists of two welded steel plates, between 6 and 15mm thickness that form two parallel skins of the system. At constant distances from each other shear connectors (studs) are welded to the plates and connects the steel plates to the concrete core. The sandwich system behaves as a combined structure, where the steel plates carry tension or compressive forces, the concrete core some compression and the studs transferring horizontal shear between the layers and vertical shear across the section. What makes this type of construction interesting is the absence of formwork, a reduction in costs of reinforcement due to the absence of bending and fixing of steel[6].

[5] The design of double skin composite elements [paper]
[6] An experimental investigation into the behavior of double-skin sandwich beams [paper]

[1b] Use foundation

Instead of adding concrete ballast inside the tank, the foundation of the tank can be used as ballast to resist the upward force. The foundation consists of a concrete slab on which the tank rests. This slab can be connected to the tank to increase the counter weight of the total structure. The slab should have enough weight which is equal to the ballast weight required if concrete ballast is put inside the tank. Foundation slabs are generally about 0.3m thick.

For example:
Tank 1: Diameter (d) = 33.2m
Foundation: Diameter = 33.2+1 = 34.2m
Weight of the bottom plate = 680kN
Buoyancy load = $^1/_4 \cdot \pi \cdot 34.2^2 \cdot 4.6 \cdot 10 \approx 43002\text{kN}$
Additional weight to be applied = 42322kN
Concrete specific weight = 25 kN/m^3

Foundation height = $\frac{42322}{1/4 \cdot \pi \cdot 34.2^2 \cdot 25} \approx 1.8\text{m}$

The foundation should be sufficiently attached to the foundation. This can be done in different ways:

I. Anchoring from the sides

Anchoring can be provided around the tank from the shell to the foundation, where the anchors prevent the tank from floating up.

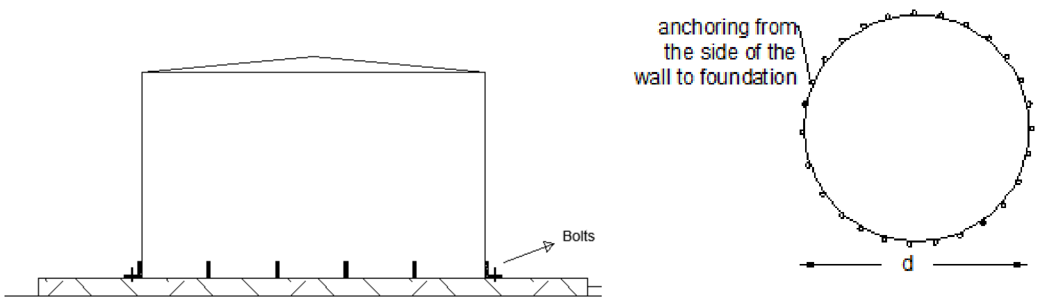


Figure 7-4 Anchor bolts for uplift resistant

Anchor bolts
Steel tanks are anchored to restrain the tank in its position. The movement of the tank can normally be summed up in the following groups:

- Uplift of the tank
- Sliding of the tank (horizontal movement)
- Overturning of the tank

  At uplift and overturning, the anchors are loaded in tension. When sliding appears the anchors are loaded in shear.

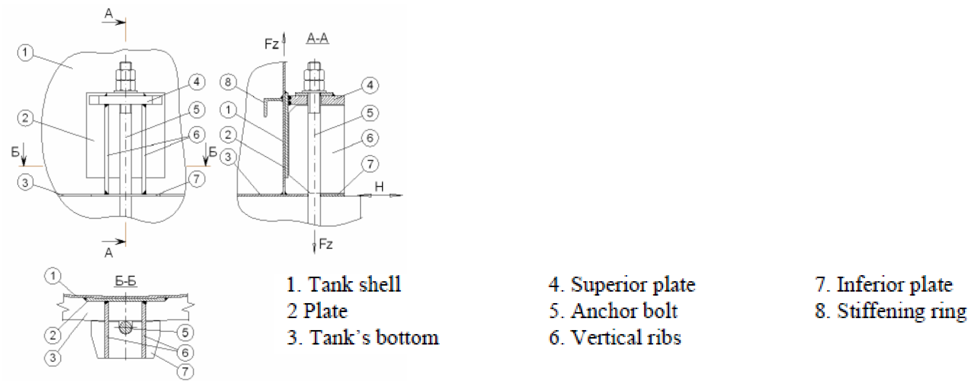


Figure 7-5 **Details of anchor bolts** [7]

For example:	
A tank with:	Diameter (d) = 23m
	Buoyancy load = $^1/_4 \cdot \pi \cdot 23^2 \cdot 4.6 \cdot 10 \approx 19000\text{kN}$
	Load/meter circumference= $\frac{19000}{\pi \cdot 23} = 263\text{kN/m}$
	5 anchors/m = 53kN/anchor

Bottom plate

Since the bolts are on the perimeter of the tank and are keeping the structure on its place, water can accumulate between the tank bottom and the foundation. This means, the bottom plate becomes subject to buoyancy forces in case of flooding and can deform freely. Large moments will be present in the plate, which can lead to large deformations. This can cause rupture of the welds between the bottom plate and the cylindrical shell and becomes critical for liquid spill.

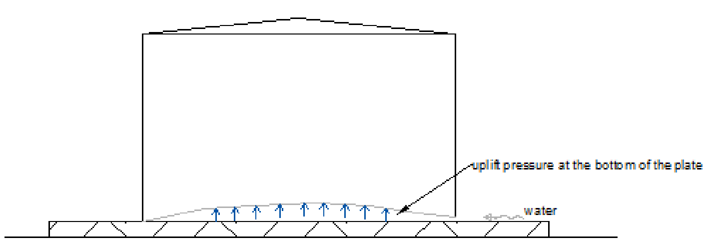


Figure 7-6 **Upward pressure against the bottom of the tank**

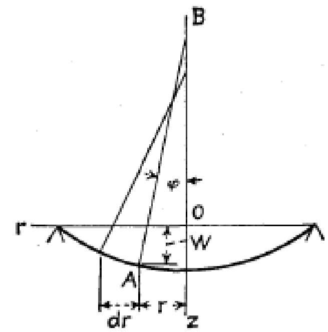
Uniformly loaded circular plates[8]

The general equation for the deflection of a uniformly distributed circular plate can be written as:

$$\omega = \frac{qr^4}{64D} + \frac{C_1 r^2}{4} + C_2 \log\frac{r}{a} + C_3$$

[7] Design problems of anchoring of above ground steel tanks; Lyubomir Zdravkov
[8] Theory of plates and shells (S. Timoshenko)

Where:



$$D = \text{flexural rigidity of plates} = \frac{Eh^3}{12(1-\upsilon^2)}$$

q = Uniformly distributed load, which is the permanent load of soil
C_1, C_2, C_3 = constant values
r = radial direction
a = radius = 1/2 diameter

$$\upsilon = \text{average value of the Poisson's ratio for steel} = 0.3$$

For simply supported edges the following applies:

The deflection at mid-span:

$$\omega_{max} = \frac{(5+\nu)qa^4}{64(1+\nu)D}$$

The maximum bending moment at the center of the plate:

$$M_{max} = \frac{3+\nu}{16}qa^2$$

This is valid if the tank is empty.

For example (see previous example):

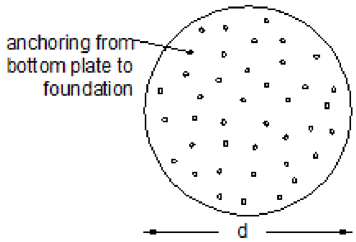
Buoyancy load = $^1/_4 \cdot \pi \cdot 23^2 \cdot 4.6 \cdot 10 \approx 19000\text{kN}$
Buoyancy pressure $= 46\ ^{\text{kN}}/_{\text{m}^2}$
a = 11.5m
$\text{M}_{\text{max}} = \frac{3+0.3}{16} \cdot 46 \cdot 11.5^2 = 1255\text{kNm}$

The following can be done:

- To minimize the deformation, the bottom plate should consist of stiffening beams to increase the strength. The transfer of the net uplift load through the wall of the tank should be checked.
- The area around the tank between the shell and the foundation (shown with a circle in Figure 7-7) should be made watertight to prevent water from seeping under the tank.

II. <u>Anchoring from the bottom</u>

The bottom plate of the tank can be anchored to the foundation by means of studs. The studs can be placed at equal distances from each other.



Here the foundation can also contribute to the counter weight against the uplift force. In this case the large deformation shown in Figure 7-6 will become smaller, due to the smaller span length of the plate between the studs.

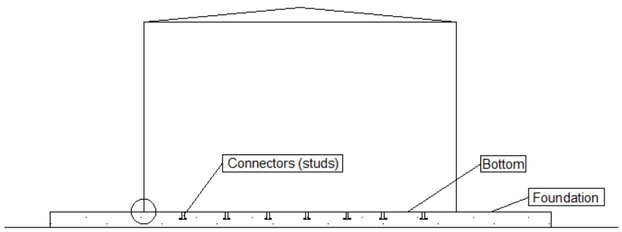


Figure 7-7 Foundation used as ballast material

[2] Steel ballast

Just like concrete ballast, also steel ballast can be applied. This can be done by adding steel as ballast material inside the tank or by making the structure itself heavier. This means the layout of the structure stays the same, but more material is added to the original design to make the structure heavier. For example: thicker walls.

[2a] Inside the tank

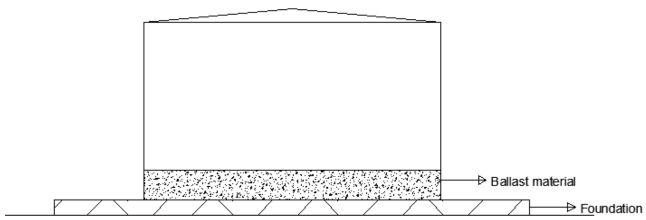


Figure 7-8 Steel as ballast material used inside the tank

Just like in [1a] the first step here is to get an indication of the required amount of steel needed. This amount is calculated for tank 1. The amount of steel required as ballast is less than concrete, because of its specific weight. Looking at the example on page 7-60, a steel ballast of 500m^3 is needed with a contributing height in the tank of approximately 0.6m. The steel ballast takes up less capacity than concrete ballast. But then, the costs of steel are higher than concrete, which makes steel less interesting.

[2b] Thicker walls

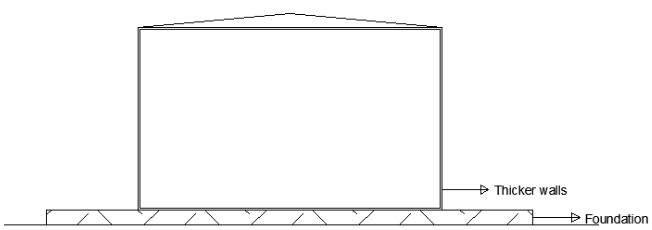


Figure 7-9 Thicker steel walls as ballast material

Tank	Required ballast weight [kN]	Steel [m^3]	Current wall thickness (approx.) [mm]	Extra thickness [mm]	Total [mm]	Ratio
2	39481	502,9	8	497	505	6214%

Table 7-1 Results for the required steel volume/wall thickness

The table above shows the required steel thickness for tank 2. With a current wall thickness of 8mm and a required extra thickness of almost 500mm, the walls would become too thick. A structure of a more than 0.5m thick wall would be economically not feasible. It can be concluded that this is not a solution to prevent uplift of the tank and no further details will be stated.

[2c] Sand ballast on the roof

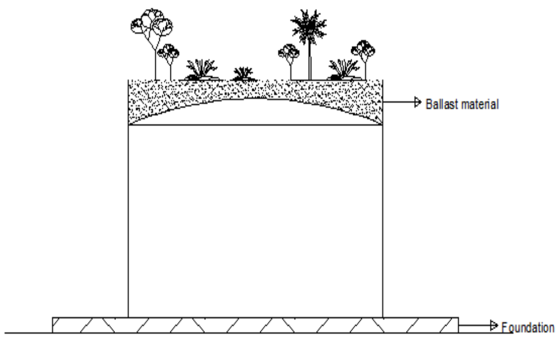


Figure 7-10 Sand on top of the tank as ballast material

The sand on top of the tank adds weight to the structure, however the bottom plate should still be provided with stiffeners, due to the acting buoyancy on that part of the tank. For the example given on page 7-60, a sand ballast of approx. 2180m^3 is needed with a contributing height of 2.5m. This option, with sand and plants on top of the roof would be an environmental friendly option.
The required amount of ballast needed on top of the tank will be too heavy for the tank to carry. The roof would have large deformations and would require large supporting structure. The walls of the tank would require stiffening rings to carry the load.

[2d] Pile foundation

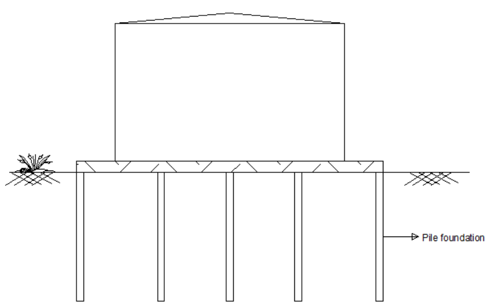


Figure 7-11 Tank on pile foundation

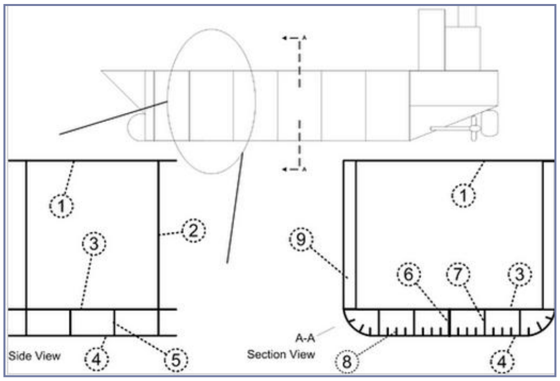
A tank on a pile foundation will be subjected to tension forces if an upward force is present on the tanks. The pile subjected to the tension force will cause shear stresses along the shaft. The ground around the piles should take up the tensile forces. The tip will not be loaded and tip resistance will therefore not be taken into account in the strength calculation. For this application it is important to assume that the connection between the tank and the foundation is sufficiently anchored to transfer the uplift forces from the tank structure to the foundation.
This variant seemed interesting at first, but will not be considered due to the following:

- Like Option ***1bI*** (see page 7-63), sufficient anchors should be provided to transfer the upward load
- Like Option ***1bI***, the upward pressure against the bottom plate is also a problem here

This option will be cost inefficient, because of the purchase and installation of the piles. Also the same problems occur here as at some of the other variants.

7.2 Modifying the tank into a floating structure

A floating tank can be related to a ship. A ship, which floats in water, can be generally divided into a hull structure and a superstructure (deckhouse). The hull structure is the hollow lower part of the ship, floating and partially submerged and supports the remainder of the ship. Like a ship, the floating tank also has a submerged lower part (draught). Therefore, the design of the bottom such a tank is similar to that of a ship.



1. Deck plating
2. Transverse bulkhead
3. Inner bottom shell plating
4. Hull bottom shell plating
5. Transverse frame (1 of 2)
6. Keel frame
7. Keelson (Longitudinal girder) (1of 4)
8. Longitudinal stiffener (1 of 18)
9. Hull side beam

Figure 7-12 Example of the structural elements of a ship's main hull

On a transverse section of the ship, the following loads apply: (a) hydrostatic pressure due to surrounding water, (b) internal load due to self-weight and cargo weight. These loads are not always equal to each other at every point; consequently loads working on transverse members will produce transverse distortion.

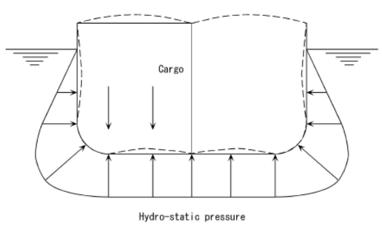


Figure 7-13 Example of deformation due to transverse loads

In the case of the storage tank the following holds:
For the given example on example on page 7-60, the draught of an empty tank is:

$$h_{draught} = \frac{1600}{1/4 \cdot \pi \cdot 33.2^2 \cdot 10} = 0.18m$$

The draught of the tank is small due to the light weight of the tank. In this case a draught of 0.18m will not require a lot of stiffeners.
The tank can be provided with column to guide the tank up and down when the terminal is flooded.

For a tank filled with chemical liquids, the weight of the liquid will counterbalance the acting buoyancy force. If the liquid height inside the tank is not sufficient for the counterbalance, the tank will start floating and will have a larger draught. In both cases the net moment working on the bottom plate will be small due to the liquids inside the tank.

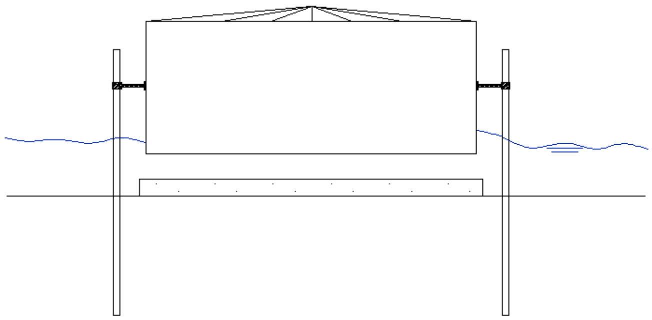


Figure 7-14 Floating tank

An issue with this alternative is the accumulation of debris under the tank if the tank is floating. If the flood height is lowering, the tank is also lowering with the water level. If there is debris under the tank, the tank cannot return onto its original position on the foundation.

7.3 Manage tank operations

Current mitigating proposals (2012) (paragraph 3.5)

After the severe damage caused by Katrina and Isaac, the existing "pre-storm preparation procedures" were improved by "damage mitigation proposals" at Stolt-Nielson. These mitigation proposals are specified to tanks, equipment preservation, personnel preservation, asset preservation and information preservation. Based on the forecasted upcoming storm and its predicted threats, the following measures can be taken to storage tanks [19]:

1. Emptying of the contents of the tanks into trucks, barges, railcars or a ship → cleaning of the tank → tank should be left empty with the manways open during the storm →this can lead to physical damage to the tank, but no environmental damage.
2. Filing of the tanks, also non-bolted tanks with (river) water or product to a level which will resist the flood level inside the terminal → this can lead to contamination of the product.
3. Addition of ballast, anchors to the bottom of the tanks → this should prevent uplift, release of product and floor damage.

Filling of an oil tank is a measure that has already been implemented in Hawaii and a number of plants in the United States, but has not yet been adopted along the Gulf Coast. [7]

Disadvantages of the proposals

Some disadvantages are given below, regarding each statement made above.

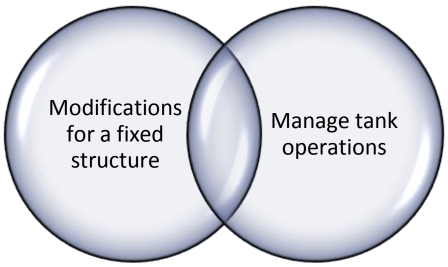
1. During hurricane Katrina not only tanks but also railcars and trucks were damaged, this apparently also lead to chemical spill. This means that this option can be applied as a mitigating solution in the hurricane period, but these cars are also subject to the influences of the hurricane and can also be damaged.
2. Tanks that are empty can be filled with a product or water to resist the upward force. For a 1/100y flood, the design water level is 5.6m NAVD88. Assuming a ground elevation of +1m and a specific weight of the product of lower than 10KN/m^3, the height of liquid inside the tank should be higher than 4.6m to provide enough weight. The height of the tanks varies somewhere between 5m and 15m. The average height of a storage tank can be set at approximately 10m.

This means that 50% of the tank should be filled with stored material. Therefore a large inventory should be available, at least in the hurricane period.
If the tank is filled with water, this can be sea or river water, it can contaminate the product which is already inside the tank.

3. Strapping of the tank to the foundation can prevent the structure from moving, but due to the large upward force on the bottom plate of the tank, floor damage can still occur, which can lead to spill.

7.4 Combination

To provide for additional weight a combination can be made between a fixed structure and tank operations. A maintained liquid height inside the tank will also provide for additional weight.



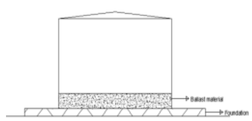
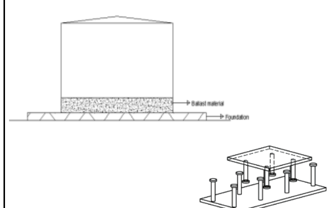
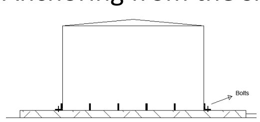
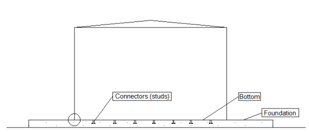
Therefore a combination can be made between for example the concrete ballast and tank operations. Less ballast can be used inside the tank, if a specific level of liquid is available during the hurricane period. In this case the amount of ballast will decrease and on the other hand the required inventory level of liquid will not be as high as 50% of the tank.

7.5 Alternative selection

For further elaboration, a selection of alternatives is made. From the above mentioned alternatives (see Figure 7-1) a selection is made based on a Multi Criteria Analysis (MCA). First, a summary is given of some advantages and disadvantages of each alternative.

<u>Summary of advantages and disadvantages</u>

The table below gives a brief presentation of some advantages and disadvantages of the alternatives.

		advantages	disadvantages
[1a]	Concrete slab 	– Adds weight	– Reinforcement required – Coating between concrete and chemical liquid – Thick concrete slab required
[1b]	Combination Steel and concrete; making use of a composite slab 	– No reinforcement required – Upper steel plate (of SCS slab) forms a layer between concrete and chemical liquid	– Thick concrete slab required
[2a]	Concrete ballast with the use of the foundation Anchoring from the sides 	– Adds weight – Uses foundation as additional weight	– the area between the bottom edge of the tank and the foundation should be made watertight , to prevent accumulation of water between the bottom plate and the foundation – side anchors are obstacles if the bottom edge should be sealed (mentioned above)
[2b]	Concrete ballast with the use of the foundation Anchoring from the bottom 	– Adds weight – Uses foundation as additional weight	– the area between the bottom edge of the tank and the foundation should be made watertight , to prevent accumulation of water between the bottom plate and the foundation

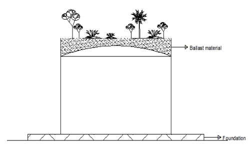
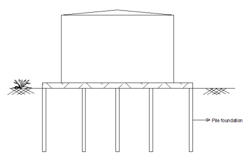
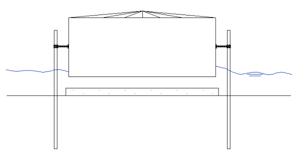
[3]	Sand ballast on the roof 	– Adds weight – contributes to the ecology	– due to heavy weight of the soil on the roof, a heavy steel structure is needed as roof – thicker walls are required to carry the extra weight
[4]	Pile foundation 	– Adds weight	– the piles are a cost inefficient option – this alternatives does not prevent the tank from floating if the connection between the foundation and the tank is not sufficient
	Floating structure 	– No additional weight required	– a heavier profiled bottom plate is needed, due to floating of the structure – accumulation of debris under the tank

Table 0-1 Some advantages and disadvantages of each alternative

7.6 Multi Criteria Analysis

To make a decision which alternatives are the best solutions for the storage tanks, a multi criteria analysis (MCA) is executed. At first the criteria and variables are determined, then weight factors are applied and at last the scores of each alternative are given.

Determination of criteria

The table below gives an overview of the criteria with their description on which the alternatives are compared.

Criterion	Description
Functionality	Satisfaction of the function/uncertainties
Innovation risk	Risk of technical difficulties during construction
Execution	Modularity: production/construction in modules
	Logistics: execution in number of steps
	Construction time: length of the construction period
Maintenance	Ease to repair

Table 0-2 Criteria with description

Weight factors of criteria

A weight should be appointed to each criterion to show the importance of each one with respect to the other. This is necessary for the MCA. If a criterion in the row is more important than the criterion in the column, the value '1' is given. If not, the value '0' is given. The last column gives the summation of all values for each criterion. The most important criterion has the highest total and the least important the lowest.

	Functionality	Innovation risk	Execution	Maintenance	Total	Weight
Functionality	x	1	1	1	3	4
Innovation risk	0	x	0	1	1	2
Execution	0	1	x	1	2	3
Maintenance	0	0	0	x	0	1

Table 0-3 Weight determination

The storage tank should at least serve its function of storing chemicals, which is the reason that functionality has the highest points.

The weight factors can be determined:

	Weight	Weight factor
Functionality	4	4/10=0.4
Execution	3	3/10=0.3
Innovation risk	2	2/10=0.2
Maintenance	1	1/10=0.1
	10	1

Table 0-4 Weight factors of criteria

Scores of the alternatives and results

Each alternative is given a score between 1-5 on the variables. This score is multiplied by its weight factor. Then, the total of this is summed up and the total score for each alternative is determined. Note: Innovation risk has a negative impact on the alternatives and is for this reason scored with a negative value.

Alternatives	Functionality [0.4]	Execution [0.3]	Innovation risk [0.2]	Maintenance [0.1]	Total score
Concrete ballast inside the tank					
Pouring of concrete	4	2	(-)3	3	1.9
Combination steel and concrete	4	3	(-)3	3	2.2
Concrete ballast with the use of the foundation					
Anchoring from the sides	3	3	(-)2	2	1.9
Anchoring from the bottom	4	3	(-)3	2	2.1
Sand ballast on the roof	3	3	(-)4	1	1.4
Pile foundation	2	2	(-)4	3	0.9
Floating structure	4	2	(-)2	4	2.2

Table 0-5 Scores of alternatives and results

The 3 alternatives with the highest values are further elaborated. These are:

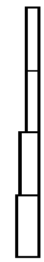
1. Concrete ballast inside the tank: Combination steel and concrete
   From now on referred to as: Steel-concrete-steel composite slab (SCS slab)
2. Concrete ballast with the use of the foundation: Anchoring from the bottom
   From now on referred to as: Foundation anchoring from the bottom plate
3. Floating structure

8 Elaboration concept tank design

In this part of the report, the 3 different methods are analyzed to come to an improved storage tank design for the expansion site. This design is able to resist hurricane forces, to prevent damage and spill of storage tanks during hurricane season.
The following section describes a more detailed description of the 3 chosen alternatives.
For each chosen alternative a more detailed description and calculations is given.

Tank example
For further elaboration of the alternatives, the following is assumed:
Cone roof storage tank:

Diameter (d): 30m
Volume ≈ 8000m^3
Wall thickness: stepped wall of 4 courses, first 2 courses: 6.5mm,
3rd course: 7mm, 4th course: 12mm
Height (H_{total}): variable and depends on the alternatives
(course height = Height/4)
Specific weight of stored liquid: 0.85-1.6 g/ml



Stepped wall with 4 courses of different thicknesses

8.1 Steel-concrete-steel composite slab (SCS slab)

This tank alternative is called **Tank A** in the remainder of the report.

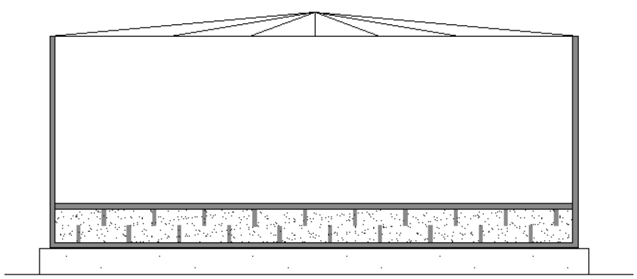


Figure 8-1 Example of Tank A (shear studs are not to scale)

General information and stress distribution of the SCS slab is given in Appendix IX.
The following section includes the following:

- Concept design
  - Selection of SCS type
  - Steel plates of the slab
  - Required amount of concrete
  - Studs
  - Stability checks
- Building procedure
- Combination with tank operations
- Elaboration on details

8.1.1 Concept design

This part of the report gives a concept design of the SCS slab for the implementation of tank A.

The SCS slab is mainly provided as ballast material inside the tank. This slab has further no other constructive function. During a 1/100y flooding, the net moment on the slab will be zero.

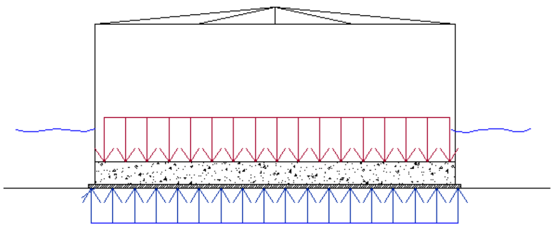
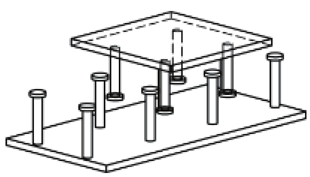


Figure 8-2 Net moment on the bottom plate is zero

<u>Selection of SCS type</u>

Figure 7-3 shows typical construction layouts for the SCS slab. For the storage tanks it will be useful to use the double-skin sandwich construction (DSC). This type has separate shear studs welded on both steel plates. The shear studs are not connected to both plates, which makes the building process easier, because the studs can be welded on each plate separately and then be put together on site, where the concreting can take place.



<u>Steel plates of the slab</u>

If the steel plates have similar thickness and strength, the SCS sandwich beam can be considered as an under reinforced beam. The beam can deflect extensively and wide cracks can occur in the final loading. The moment resistance of the sandwich section will therefore be less than if the plates are not of equal thickness. A more detailed explanation is given in Appendix IX.
For the calculations it is therefore chosen to have different thicknesses for the plates. The thickness of the upper plate (t_1) is chosen to be 6mm and the thickness of the bottom plate (t_2) is 8mm.

<u>Required amount of concrete</u>

If the weight of the entire structure is equal to the buoyancy load, it prevents the structure from lifting off its foundation. But this will not prevent the bottom plate from having large deformations, due to the large unstiffened area of the bottom plate.
So the required amount of additional weight, in this case concrete, required for preventing large deformation and rupture of the tank depends on the weight of the bottom plate.

The weight of the 2 steel plates will be negligible compared to the concrete core, so only the weight of the concrete is considered for the weight of the SCS slab.
Because the weight of the SCS slab should be the same as the buoyancy force generated at a 1/100y design still water depth, the following relation applies:

Weight SCS slab = Buoyancy load

$\rightarrow \frac{1}{4}\pi d^2 \cdot t_{slab} \cdot \gamma_c = \frac{1}{4}\pi d^2 \cdot h_w \cdot \gamma_w$

$\rightarrow t_{slab} \cdot \gamma_c = h_w \cdot \gamma_w$

$$\rightarrow t_{slab} = \frac{h_w \cdot \gamma_w}{\gamma_c}$$

where:

d	= tank diameter	[m]
t_{slab}	= thickness of the SCS slab	[m]
h_w	= flood height on terminal = design still water depth (d_s) (see 0)	[m]
γ_c	= specific weight of concrete	[kN/m^3]
γ_w	= specific weight of concrete	[kN/m^3]

The loads all work on the same area (tank area) and the values for h_w, γ_c, γ_w are constant, so t_{slab} will have the same value for all tank diameters:

$$\rightarrow t_{slab} = \frac{1.5 \cdot h_w \cdot \gamma_w}{\gamma_c} = \frac{1.5 \cdot 4.6 \cdot 10}{25} \approx 2.76m$$

[Note: A safety factor of 1.5 is used for the buoyancy load]

The value of t_{slab} is independent of the height of the tank, but the SCS slab is placed inside the tank, so the height of the tank should be increased for a certain capacity of the tank.

To maintain a volume of 8000m^3, the height of the tank is:

$$H_{total} = \frac{Volume}{tank\ area} + t_{slab} = 11.32 + 2.76 = 14.08m$$

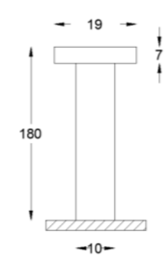
<u>Shear studs</u>

Shear studs are welded on both plates.

With the design guidelines given in Appendix IX, the following is chosen for the shear stud connectors:

Stud connectors[9]:

- D = 10mm
- L = 180mm
- s (spacing between the stud connectors) = 250mm
  It is chosen to apply the same spacing for the upper and lower studs. Therefore the plate thickness with the smallest value is used (6mm).



The result for the spacing is: 276mm → 275mm is chosen

For a tank with a diameter of 30 m, this results in approx. 22620 stud connectors for both steel plates.

[Note: A more detailed calculation should be done if the application of longer studs is possible in this case]

[9] http://www.chinabolt.biz/html_products/34-x-4-78-weld-shear-connector-studs--for-steel-structural-with-ce-and-iso-66.html

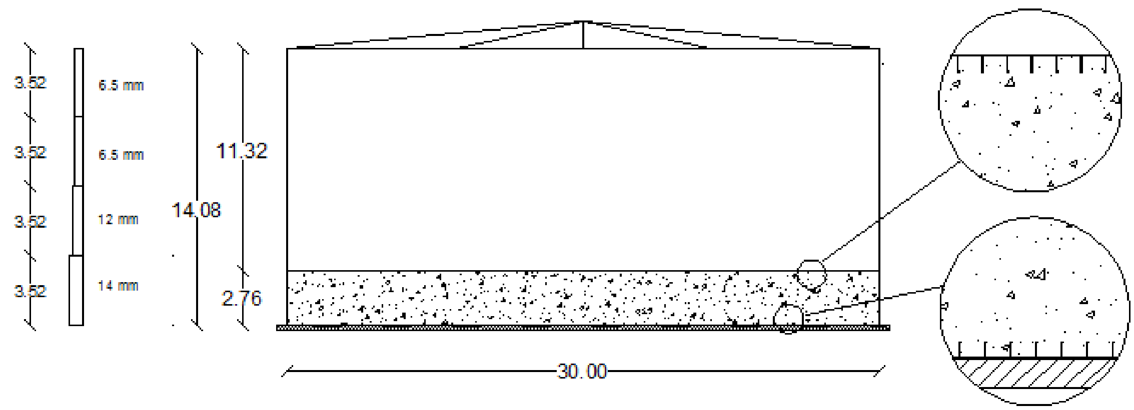


Figure 8-3 **Overall dimensions of Tank A in meters (thickness is in mm and not to scale)**

Stability checks

The most unfavorable load combinations are given below.

Vertical stability: bearing capacity of soil

For the vertical stability the weight of the entire structure should be considered, including the weight of stored liquid.
Most unfavorable load combination: Dead weight + liquid weight (fully filled) + wind, no flood is present. The presence of flooding has a favorable effect on the vertical effective soil pressure and is therefore not taken into account.
The calculation is done with the use of Appendix X B and is given on page 11-151.

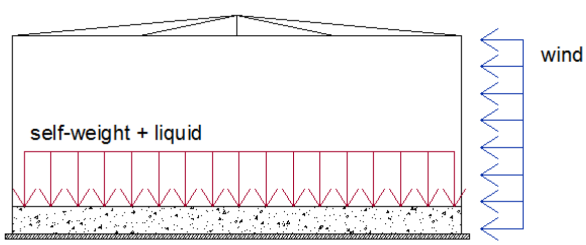


Figure 8-4 Load combination for the bearing capacity of soil

For Tank A, the bearing capacity of the soil is not sufficient to carry the weight of the structure if this is filled with liquid. Measures should be taken to improve the stability of the soil.

Methods for inadequate support of the subsoil [6]

When the conditions of the subsoil are not adequate enough to support the tank without extreme settlements, some of the following methods have been used:

- Take out inadequate material and replace with proper compacted material
- Built up sufficient backfill to preload the area
- Reduce the bearing pressures by designing the foundation with extended foundation areas and/or piles
- Subsoil stabilization with the use of vibrocompaction or dynamic compaction methods

Horizontal stability
For the horizontal stability, the structure is verified for the resistance against sliding. For the friction coefficient *f* between steel and concrete a value of 0.4 is used.
Most unfavorable load combination: Dead weight + buoyancy load+ hydrodynamic load + wave load + wind, flood is present.

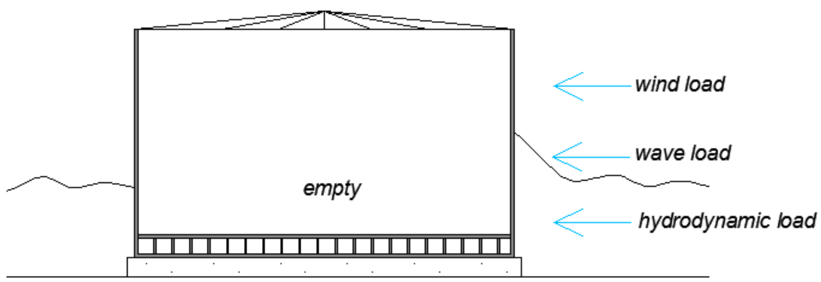


Figure 8-5 Load combination for the horizontal stability

The structure is stable in horizontal direction. The calculation is done with the use of Appendix X B and is given on page 11-153.

Rotational stability
According to API Standard 650 the safety against overturning with regard to the wind load is checked on the basis of two criteria. Both criteria involve the working moments of the active loads on the structure.

The structure is stable in rotational direction. The calculation is done with the use of Appendix X C and is given on page 11-153.

8.1.2 Building procedure

A proposition is made with regard to the building of this type of storage tank. The building procedure can be summed up in steps:

- Welding of steel plates together to form 2 circular plates for the upper and bottom part of the SCS slab. The bottom plate should be larger than the upper plate, for the connection of the shell.
- Welding of the shear studs on both plates according to the required spacing between the studs.
- On site, the bottom plate of the SCS slab can be placed on the foundation.
- The plates for the lower course of the shell can be welded to the bottom plate of the SCS slab. This can be done on both sides of the wall structure.
- The upper plate of the SCS slab can be positioned on top of the bottom plate with the use of steel or concrete separators to maintain the height of the concrete core.
- Some openings can be made in the bottom plate for the pouring of concrete between the steel plates for the concrete core.
- Welding of the remaining part of the shell structure
- Installation of the roof

8.1.3 Combination with tank operations

Here, it is proposed to maintain a certain level of liquid inside the storage tank, in order to increase the weight and therefore decrease the required materials.
The liquid inside the tank should have a minimum operating requirement level, which should be managed especially during hurricane season when the risks of flooding are high.

Or, liquid should be able to flow inside the tank when a certain water level is reached outside. The specific weight of the stored liquid varies between 0.85 and 1.6 g/ml.
[Note: A minimum liquid level is currently already maintained, but this level only depends on the height of the valves of the tank and is not taken into account in the elaboration of the solutions in Chapter 0.]

Liquid level
The weight of the liquid and the weight of the bottom plate will together counteract the buoyancy load. During hurricane season the tank should have a minimum liquid level inside that is obtained by the 1/100y flood height, the thickness of the SCS slab and the specific weight of the stored liquid.

The following applies:
Weight SCS slab + Weight liquid = Buoyancy load
$\rightarrow \frac{1}{4}\pi d^2 \cdot (t_{slab} \cdot \gamma_c + h_l \cdot \gamma_w) = \frac{1}{4}\pi d^2 \cdot h_w \cdot \gamma_w$
$\rightarrow t_{slab} \cdot \gamma_c + h_l \cdot \gamma_w = h_w \cdot \gamma_w$

$$\rightarrow t_{slab} = \frac{(h_w - h_l) \cdot \gamma_w}{\gamma_c}$$

where:

d	= tank diameter	[m]
t_{slab}	= thickness of the SCS slab	[m]
h_w	= flood height on terminal	[m]
h_l	= liquid level inside the tank	[m]
γ_c	= specific weight of concrete	[kN/m^3]
γ_w	= specific weight of concrete	[kN/m^3]

Figure 8-6 shows the required thickness of the SCS slab for the case of a maintained liquid level inside the tank.

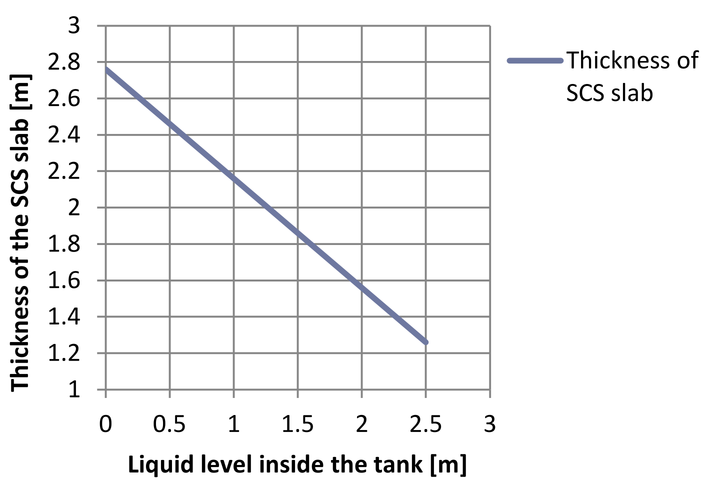


Figure 8-6 Relation between stored liquid level and thickness of the SCS slab

Table 8-1 shows an indication on the effects of the liquid level inside the tank on the reduction of concrete.

Stored liquid height [m]	Reduction of concrete [%]	Reduction operational capacity [m3]
0	0%	0%
0,5	11%	4%
1	22%	9%
1,5	33%	13%
2	43%	18%
2,5	54%	22%

Table 8-1 Reduction of concrete and operational capacity

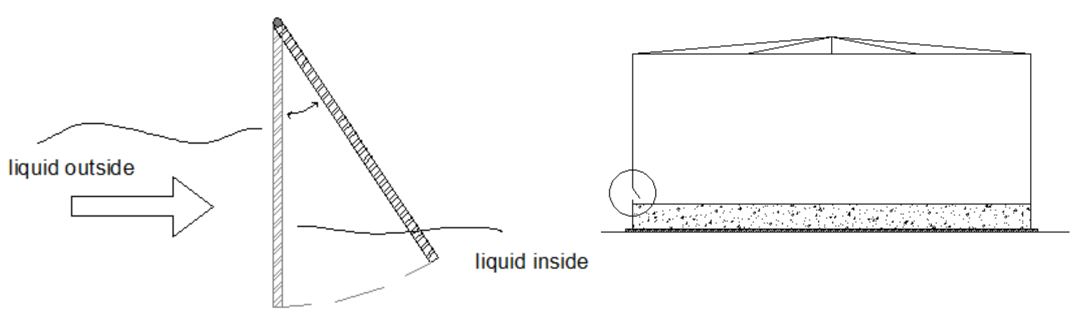
A maintained liquid height of 1m inside the tank can cause a concrete reduction of about 22%. This will also lead to a 22% reduction in material costs for the SCS slab. A 1m maintained liquid height, which is also called: "unavailable inventory", decreases the operational capacity of 8000m^3 with 9%.
Another advantage of a maintained liquid level is the internal pressure of the liquid against the cylindrical shell. When the 1/100y flood height occurs during a storm the storage tank will be standing in a water level of about 4.6m. This generates large hydrostatic pressure against the cylindrical shell. The liquid inside the tank causes a counter pressure and the resulting hydrostatic pressure against the cylindrical shell is decreased.

How can it be achieved
The combination of applying the SCS slab and maintaining a certain liquid level can be achieved by:

- Maintaining an "unavailable inventory" of liquid in the tank (especially) during hurricane season. This can be done by having a certain liquid height in the tank. If this cannot be achieved, because there is not enough liquid available, water can be filled into the tank.

  Disadvantages
  - The "unavailable inventory" causes a reduction in operational capacity of the tank
  - This certain height should at all time (especially in hurricane season) be available
  - If water is filled into the tank, the liquid inside will be contaminated and should be cleaned afterwards.
  - Decontamination costs

- Constructing a valve in the cylindrical wall of the tank. This valve will be able to open (e.g. with sensors) in one direction only, to the inside of the tank. When a certain pressure is reached outside the tank, the valve opens and lets water flow inside the tank. The valve will not open if the weight of the SCS slab can still counteract the buoyancy load.



[*Note*: The construction of a valve can also be applied when the SCS slab is not present for additional weight. The valve can be opened if the pressure outside the tank (at the position of the valve) is greater than the pressure inside the tank. So, if the external pressure due to the flood level outside the tank is greater than the internal pressure due to the liquid height inside the tank, the valve is opened and water can flow inside the tank.]

Disadvantages

- If water is filled into the tank, the liquid inside will be contaminated and should be cleaned afterwards.
- Decontamination costs

8.1.4 Elaboration on details

This section focuses on some critical conditions that the storage tank can encounter under normal and natural disasters. What are the effects of these conditions for Tank A?

Shell-to-bottom connection

Generally the tank-to-bottom connection is executed by welds at the bottom plate on both sides of the shell.

Due to the SCS slab inside the tank, another configuration of the connection should be applied. This depends on the loads working on the welds and also on the building procedure of the slab in the tank. The following connection between the SCS slab and the shell is proposed:

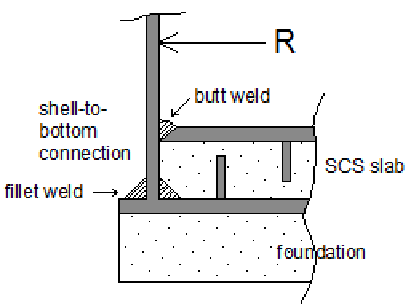


Figure 8-7 Shell-to-bottom connection of tank A (shear studs are not drawn to scale)

The new tank design, also called tank A, has a bottom plate of steel and concrete, where the concrete core of the slab has a thickness of approximately 1.85m. This is a very thick plate and it can be assumed that the slab has a high stiffness. The wall and bottom plate are welded on 3 locations, two fillet welds at the bottom on both sides of the wall and one butt weld on one side of the wall (see Figure 8-7).

The butt weld is made of similar thickness as the mother material (plate material). Here, the weld is not governing for the strength of the connection, but the plate material (mother material) is the governing element. If the welding is performed properly, the minimum strength of the weld material is higher than the plate material, so weld calculations are not required. [27]

Design check for the plate material:

The hoop stress generated by the internal hydrostatic pressure should be smaller than the yield stress of the plate.

For a steel grade of S355 (yield strength = 355 N/mm^2) and the provided information given on page 8-77 and Figure 8-3, the following is concluded:

$$\sigma_H = \frac{p \times d}{2t} \le \frac{f_y}{2}$$

where:

σ_H	= the hoop or circumferential stress	[kN/m² or N/mm²]
p	= the pressure in the cylinder (if filled with water)	[kN/m²]
d	= the diameter of the cylinder	[m]
t	= the thickness of the cylindrical wall	[m]
f_y	= the yield strength	[N/mm²]

$$\rightarrow \frac{(14.08 \cdot 10) \times 30}{2 \cdot 0.012} \le \frac{355}{2}$$

$$\rightarrow 176000\,{}^{kN}/_{m^2} = 176\,{}^{N}/_{mm^2} \le 177.5\,{}^{N}/_{mm^2} \text{ OK}$$

Edge disturbance

Due to the hydrostatic pressure inside the tank, the tank shell wants to expand to a radius of R+ΔR. The shell is connected to the bottom plate and the deformation in this point is equal to zero. This causes an edge disturbance.

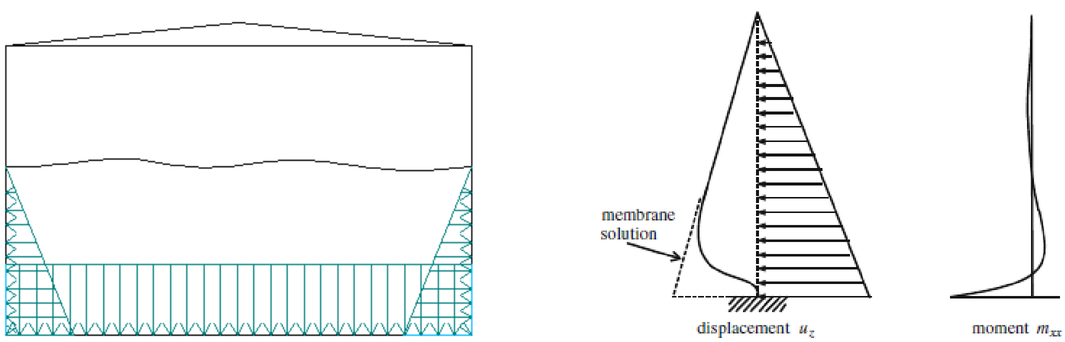


Figure 8-8 Internal hydrostatic pressure and displacement of the wall

Due to this connection, internal forces are generated to compensate for the displacements in that point. The force distribution in the wall and the slab are affected and a local disturbance occurs.

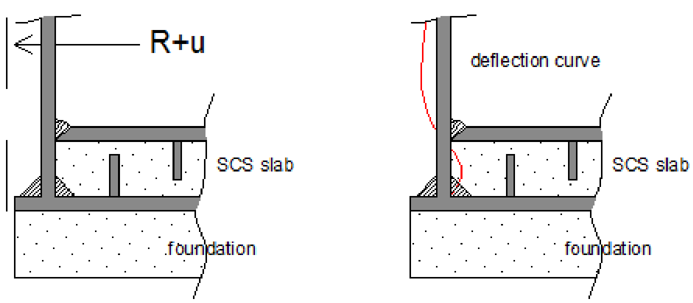
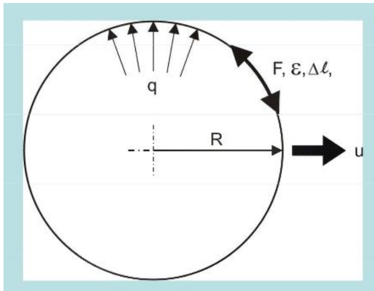


Figure 8-9 Radial displacement of a cylindrical shell with no bottom restraint (left), Deflection curve of cylindrical shell connected to a bottom plate (right)

The drawing on the right indicates the deflection of the wall due to the internal hydrostatic pressure. Off course the drawing is a bit overrated, but it gives a view of how the wall will deflect.
The radial displacement at the bottom connection of Tank A is as follows:
Tank A: diameter: 30m, liquid height: 11.35, thickness: 8mm, fully filled: water.

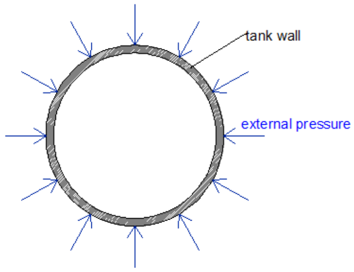


$$2\pi R \rightarrow 2\pi(R+u)$$
$$\text{elongation} = \Delta\text{l} = 2\pi R\varepsilon)$$
$$\text{F} = \text{q}R$$
$$\varepsilon = \frac{\text{F}}{Et}$$
$$\Delta\text{l} = 2\pi\frac{\text{q}R^2}{Et}$$
$$\Delta\text{l} = 2\pi \cdot u$$
$$radial\ displacement: u = \frac{\text{q}R^2}{Et}$$
$$Tank\ A: u = \frac{(11.35 * 10)\cdot 15^2}{210\cdot 10^6 \cdot 0.008} = 0.0152m = 15.2mm$$

The shear force and moment generated to prevent this radial displacement has to be taken by the welds, in this case the single upper weld of the slab.

External pressure
The external pressures working on the storage tank are wind and hydrostatic pressure.
The effects of the external pressure are examined by 2 methods, the design criteria stated in the API Standard 650 [23] and the weighted smeared method which is stated in an article studying the external pressure of stepped walled cylindrical shells [28].

Both methods consider the same loading conditions, a uniformly distributed external pressure. This is shown in the figure below.



a. Wind
The external wind pressure is considered uniformly distributed. With the information given on page 8-77 and Figure 8-3, a calculation check is done for Tank A to see if the wall thicknesses are sufficient for the external wind pressure.
The basic wind speed of 259 km/h corresponds to a maximum wind pressure of $2.8\frac{kN}{m^2}$ (section 3.4.2).

- The weighted smeared method
  In the article [28], calculations are given for the determination of the critical buckling pressure of cylindrical shells with varying thickness subjected to a uniform external pressure. Here, the different thicknesses are converted to an equivalent thickness.

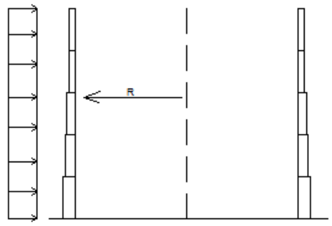


Figure 8-10 Uniform external pressure on a stepped wall cylindrical shell

The non-uniform wind pressure on the tank can be substituted by an equivalent uniform wind pressure [29]. A full description can be found in 0.

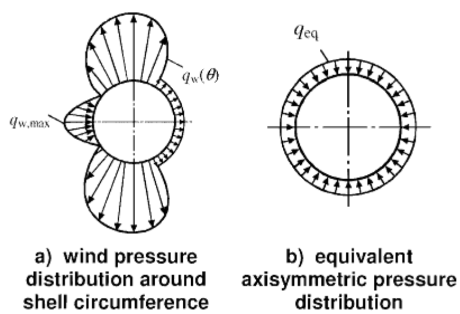


Figure 8-11 Transformation of typical wind external pressure load distribution [29]

Evaluation
The critical buckling pressure for this tank is the lowest value of the 3 results given in the first column of Table 8-2 (the full description is given in Table 11-7), which is 1.12kN/m^2. This value is far less than the maximum wind pressure of $2.8\frac{kN}{m^2}$. This means that circumferential stiffeners should be applied.
Another option is to design the tank with a smaller diameter. The critical buckling pressure also depends on the radius of the storage tank. If the radius is small, the critical buckling pressure is higher. For example: If the same thicknesses are used for a tank with a diameter which is half of the current one, so 7500mm, the results for the critical buckling pressure would be the following:

	$P_{cr,s}$ [kN/m^2]	
	r = 15000mm	r = 7500mm
1	1.61	4.55
2	1.12	3.16
3	1.18	3.34

Table 8-2 Critical buckling pressures for r = 15000mm and r = 7500mm

The results for a tank with half the diameter are almost 300% higher.

- API Standard 650
  In the standard requirements are given for the design external pressure of a tank. Due to the varying thickness the shell height is here converted into a transformed shell height H_{TS}.
  The wind pressure here is uniform over the theoretical buckling height of the tank shell. It is stated that a storage tank which is subjected to a higher external pressure than the allowable should be provided with stiffeners.

The minimum required thickness is calculated for Tank A, subject to the maximum wind pressure. This can be found in Appendix XIV A.

Evaluation
If only the wind is present, the stability factor $\psi = 1$.

H_{ts} [m]	ψ	Ps [kN/m^2]	t_{smin} [mm]	H_{safe} [m]	Nr. of stiffeners
10.1	1	2.8	10.2	3.2	2

The required minimum course thickness is 10.2mm. The thinnest shell course of Tank A is 6.5mm, which is smaller than the minimum required thickness, so 2 stiffeners are required at the thinnest top course with a spacing of not larger than 3.2m. The amount of stiffeners reduces as the thickness becomes larger at the lower courses.

b. Hydrostatic pressure

The full effect of the external hydrostatic pressure on the tank is only possible if the tank stays on its position (does not lift off its foundation due to buoyancy load) and if the tank is not filled with liquid (no internal pressure present). The resulting pressure on the cylindrical shell is then equal to the external hydrostatic pressure.
As mentioned in Chapter 4.5, Figure 4-14, the weight of the tank is in general far less than the buoyancy load generated at a 1/100y flood height (approx. 4.6m).

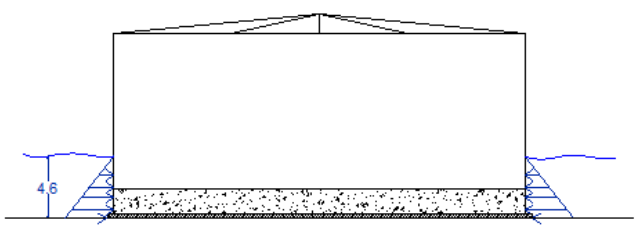


Figure 8-12 External hydrostatic pressure on Tank A

The 2 methods described above are intended for a cylindrical shell subjected to a uniform external pressure. However, for an indication of the buckling pressure, these methods are also used for the hydrostatic pressure.

Note:
One paper was acquired: "Buckling under the external pressure of cylindrical shells with variable thickness" where also the hydrostatic pressure was tested on a cylindrical shell. Due to the complexity of this paper, it is highly recommended to make use of it for more detailed calculations of the buckling of a stepped wall cylindrical shell.

The pressure increases with the depth and has the highest value at ground level. Due to the 2.76m thick SCS slab inside the tank, shell is stiffened over this height. Therefore, the remaining critical pressure on the tank is (4.6 - 2.76) x 10kN/m^2 = 18.4kN/m^2 (not taking the foundation height into account).

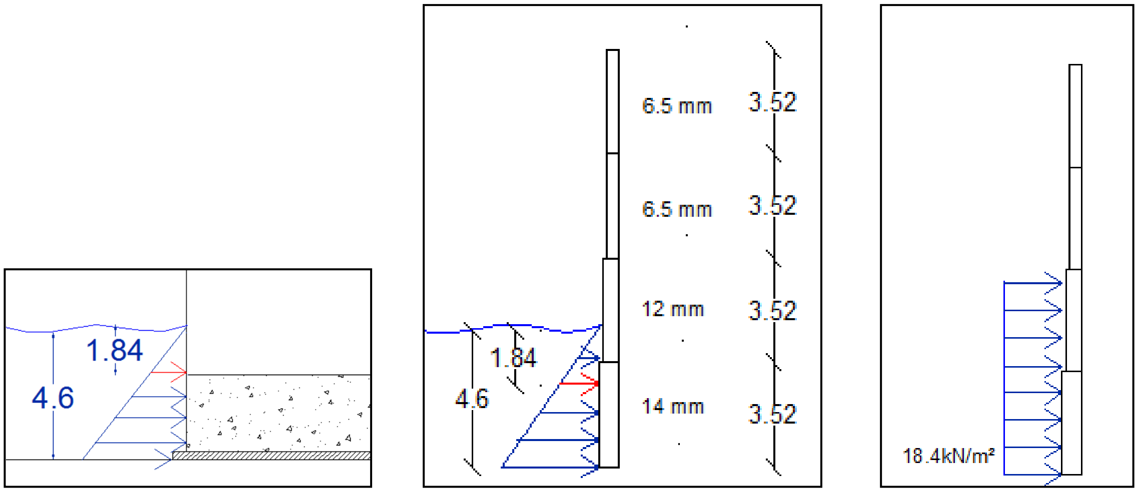


Figure 8-13 External hydrostatic pressure on Tank A transformed into uniform pressure

The external hydrostatic pressure acts in this case only on the 3rd and 4th shell course. For the calculations it is assumed that only these 2 courses are present, so the eventual use of stiffeners can be applied to the region which is affected by the hydrostatic pressure. The hydrostatic pressure of 18.4kN/m^2 is assumed to be uniformly distributed over the height of the 3rd and 4th shell course.

For an indication on the effects of the external hydrostatic pressure, the same 2 methods used for the wind pressure are applied:

- The weighted smeared method

	t_i [mm]	h_i [mm]	l [mm]	Contributing h_i [mm]	t_{eq}	$p_{cr,s}$ [kN/m^2]
3	12	3520	3520	3520	12	14.9
4	12	3520	7040	3520	13.1	9.2
	14	7040	7040	7040		

Table 8-3 Results for Tank A regarding the weighted smeared method for t3=12mm and t4=14mm

The critical buckling pressure is 9.2kN/m^2. (the full description is given in Table 11-7). This value is far less than the pressure of 18.4kN/m^2.

- API Standard 650

  The hydrostatic pressure is a lot larger than the wind pressure, therefore the stability factor $\psi = 3$. For the 2 bottom courses the following is found:

	h_n [m]	t_n [mm]	H_{ts} [m]	ψ	Ps [kN/m^2]	t_{smin} [mm]	H_{safe} [m]	Nr. of stiffeners
3	3.52	7	4.2	3	27.5	27.8	0.13	31
4	3.52	12						

Table 8-4 Results for Tank A regarding API for t3=7mm and t4=12mm

The table above shows that a minimum thickness of 27.8mm is required for a pressure of 18.4kN/m^2.

This thickness is a lot larger than the thinnest shell course, which is why stiffeners can be applied. The number of stiffeners to be applied to the 2 bottom shell courses is 31, where the spacing between these stiffeners should not be larger than 0.13m. Since the stiffener spacing of 0.13m is very small, it is recommended to apply thicker steel plates at these courses.
For a thickness of 12mm and 14mm for respectively the 3rd and 4th course, the results are:

	h_n [m]	t_n [mm]	H_{ts} [m]	ψ	Ps [kN/m^2]	t_{smin} [mm]	H_{safe} [m]	Nr. of stiffeners
3	3.52	12	5.5	3	27.5	31.2	0.51	10
4	3.52	14						

Table 8-5 Results for Tank A regarding API for t3=12mm and t4=14mm

The table above states that for the changed thickness the number of stiffeners is reduced to 10 with a stiffener spacing of 0.51m.

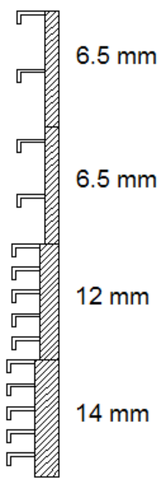


Figure 8-14 Wall of Tank A with changed thicknesses for the bottom 2 courses and provided with stiffeners
[Note: This is a concept calculation and presents rough approximations]

The bottom 2 courses can be stiffened in other ways, such as applying the SCS- slab as part of the shell wall. [This can lead to a reduction of the weight of the SCS slab inside the tank.]

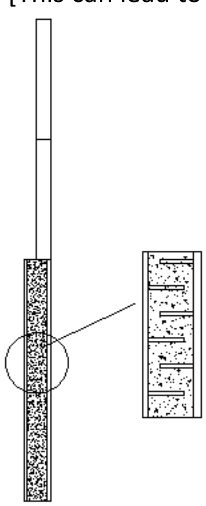


Figure 8-15 SCS sandwich wall as stiffener

8.2 Foundation anchoring from the bottom plate

The foundation is anchored to the bottom plate of the storage tank. Shear studs are welded under the bottom plate. The bottom plate, foundation and the shear studs can work as a composite structure. This tank alternative is called **Tank B** in the remainder of the report.

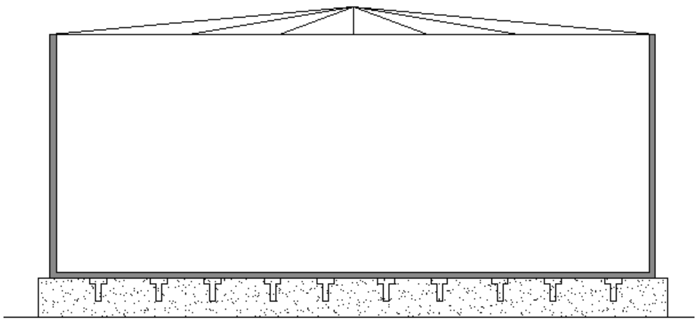


Figure 8-16 Example of Tank B (shear studs are not to scale)

This section includes the following:

- Slab design
  - Required amount of concrete
  - Studs
  - Stability checks
- Building procedure
- Combination with tank operations
- Elaboration on details

8.2.1 Concept design

Steel plates of the slab

The thickness of the steel plate is chosen to be 10mm.

Required amount of concrete

The amount of concrete needed for the foundation is the same as for the SCS slab. (see section 8.1.1)
$\rightarrow t_{slab} = 2.76m$

The height of the tank does not depend on the height of the foundation slab.
To maintain a volume of 8000m^3, the height of the tank is:

$$H_{total} = \frac{Volume}{tank\ area} = 11.32m$$

Shear stud connectors

Shear studs are welded on both plates.
To take on the working shear force, 88 stud connectors (19mm diameter) are needed for the whole area of the bottom plate (shown in Appendix IX).
With the design guidelines given in Appendix IX, the following is chosen for the shear stud connectors:

Stud connectors[10]:
- D = 19mm
- L = 190mm
- s (spacing between the stud connectors) = 450mm

For a tank with a diameter of 30 m, this results in approx. 3491 stud connectors.

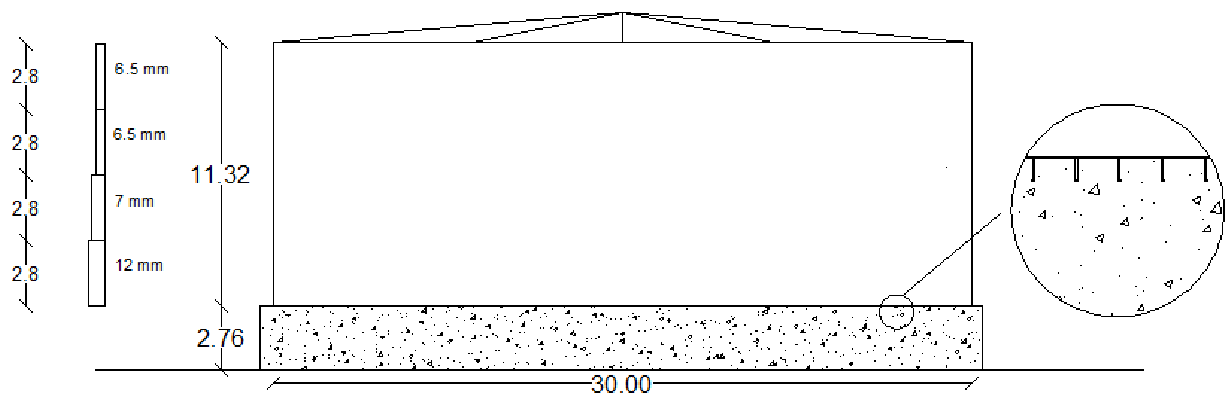


Figure 8-17 Overall dimensions of Tank B in meters (thickness is in mm and not to scale)

If the tank is fully filled and no buoyancy load is present, the bottom of the foundation will be in tension. Therefore, the foundation should be provided with reinforcement (minimum reinforcement should suffice) to take up the tension stresses.

Stability checks

The vertical, horizontal and rotational stability are checked. Since the overall structure is similar to Tank A, the results are identical to that of Tank A (see 8.1.1).

8.2.2 Building procedure

A proposition is made with regard to the building of this type of storage tank. The building procedure can be summed up in steps:
- Installation of the formwork for the foundation slab
- Installation of the reinforcement, which is located on the bottom of the foundation slab
- Concrete blocks should be placed at several locations to hold the bottom plate of the tank on top of the freshly poured concrete
- Casting of the concrete of the foundation slab
- Simultaneously with the above, the bottom plate of the tank is welded together
- The studs are welded on the bottom plate
- After pouring the concrete into the formwork, the bottom plate is placed on top of the concrete and is resting on the concrete blocks

[10] http://www.chinabolt.biz/html_products/34-x-4-78-weld-shear-connector-studs--for-steel-structural-with-ce-and-iso-66.html

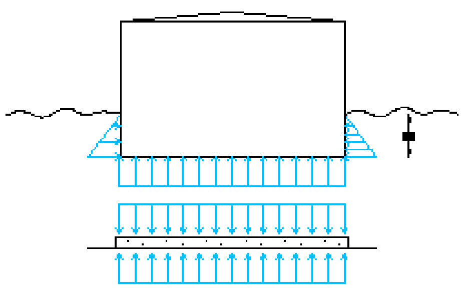
8.2.3 Combination with tank operations

For the concept design of anchoring the foundation to the bottom of the tank, also requires a concrete thickness of the foundation of approximately 2.76m. This amount can also be reduced if a certain level of liquid inside the tank can be maintained.
The weight of the liquid and the weight of the bottom plate will together counteract the buoyancy load. The same applies here as for Tank A, this is described in section 8.1.3**Error! Reference source not found.**

8.2.4 Elaboration on details

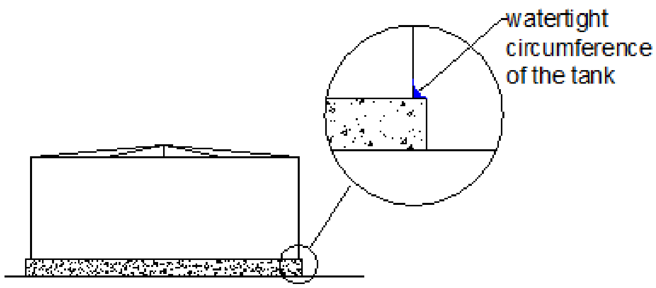
This section focuses on some critical conditions that the storage tank can encounter under normal and natural disasters. What are the effects of these conditions for Tank A?

<u>Shell-to-bottom connection</u>
If the terminal is flooded, water can accumulate between the tank bottom and the foundation. This will cause an upward on the bottom of the tank, which can dislocate the tank from the foundation.



For the area around the perimeter of the tank between the shell and the foundation, the following can be done:

- Prevent water from seeping through by making the perimeter of the tank between the shell and bottom watertight



This can be done by:

- Spraying a coating system around the tank, which produces a seamless and impermeable membrane [30]. For example:

  The Rhino linings coating system is sprayed on and can therefore take any shape and size. The typically minimum thickness is 1.5mm and can have up to an unlimited thickness. According to the company this coating can take up flood loads and can be used to prevent water from seeping under the bottom tank.

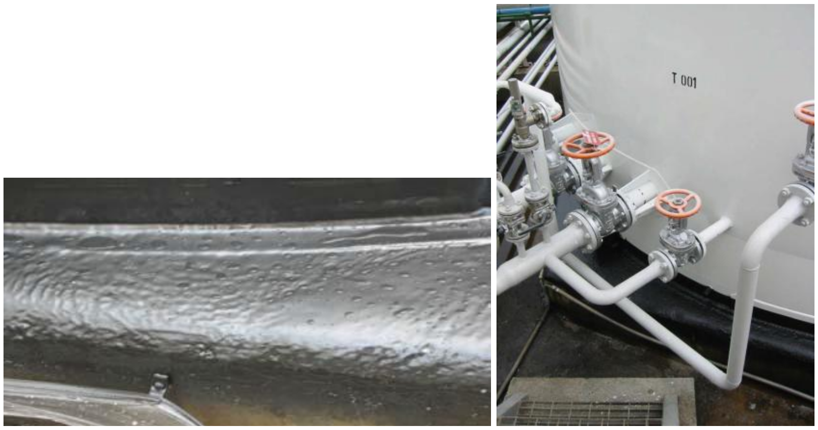
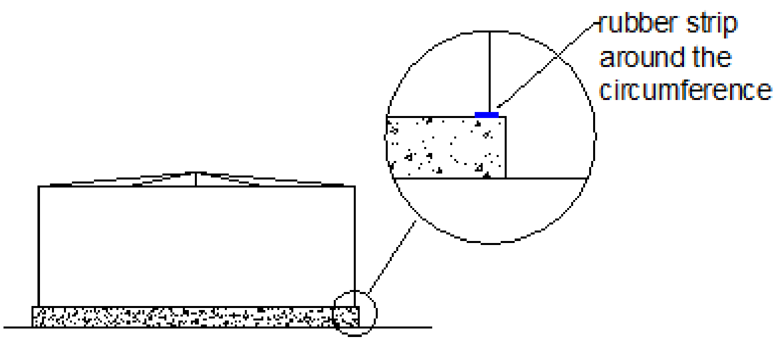


Figure 8-18 Example of the Rhino Linings coating system [11]

- A rubber strip can be put between the shell bottom and the foundation around the perimeter of the tank.
  The shear studs between the bottom plate and the foundation are providing compression on the strip.



- Take no action. In this case the bottom shell and the shear studs will be under the influence of a buoyancy force of approximately 46kN/m^2. The plate will deform upward which will cause tension to the shear studs. The plate should be designed to take on those forces. The number of studs can be increased to decrease the span length of the plate between the studs.

[11] http://www.rhinoliningsindustrial.com/gfGy6S44anlz/1272566194Oil%20Tank%20Footings.pdf

8.3 Floating structure

The tank is guided by piles when its floating.
This tank alternative is called **Tank C** in the remainder of the report.

This section includes the following:

- Slab design
    - Required amount of concrete
    - Studs
    - Number of piles
- Building procedure
- Elaboration on details

8.3.1 Concept design

Design of the bottom plate

The bottom plate should be able to transfer the moment distribution in the bottom plate when the tank is floating. For the stiffening of the bottom plate, the same SCS sandwich slab is used.
The design value of the maximum occurring moment is determined. Also the allowable moment of the SCS sandwich slab is determined with the plastic moment capacity.

As first assumptions, the following is given:

- Use of the SCS sandwich slab for the transfer of the bending moment in the bottom plate. Steel plate thickness of 6mm (t_1) and 8mm (t_2). Concrete height (h_c) of 60mm.

Most unfavorable load combination: Dead weight + buoyancy load, flood is present. A safety factor of 1.5 is used for the buoyancy load.

The calculation can be found in Appendix XIII D.

A bottom plate consisting of:

- 2 steel plates of 6 and 8mm
- concrete base of 60mm

seems sufficient for the bending moment resistance.

Shear studs

Shear studs are required to connect the steel plates to the concrete core. These studs are welded on both plates. With the design guidelines given in Appendix IX, the following is chosen for the shear stud connectors[12]:

- D = 10mm
- L = 40mm
- s (spacing between the stud connectors) = 250mm

It is chosen to apply the same spacing for the upper and lower studs. Therefore the plate thickness with the smallest value is used (6mm). The result for the spacing is: 276mm → 275mm is chosen

For a tank with a diameter of 30 m, this results in approx. 9350 stud connectors.

[12] http://www.chinabolt.biz/html_products/34-x-4-78-weld-shear-connector-studs--for-steel-structural-with-ce-and-iso-66.html

[Note: The required thickness to incorporate the SCS sandwich slab in the bottom plate is in total 74mm (2 steel plates + concrete core). This thickness may not be practical, especially for the construction of this thin slab in a tank of 30m in diameter. An alternative in this case can be the application of steel stiffeners on the original bottom plate. In this way the moments can be taken up by the stiffeners and the pouring of 74mm concrete can be avoided.]

To maintain a volume of 8000m^3, the height of the tank is:

$$H_{total} = \frac{Volume}{tank\ area} + t_{slab} = 11.32 + 0.06m = 11.38m$$

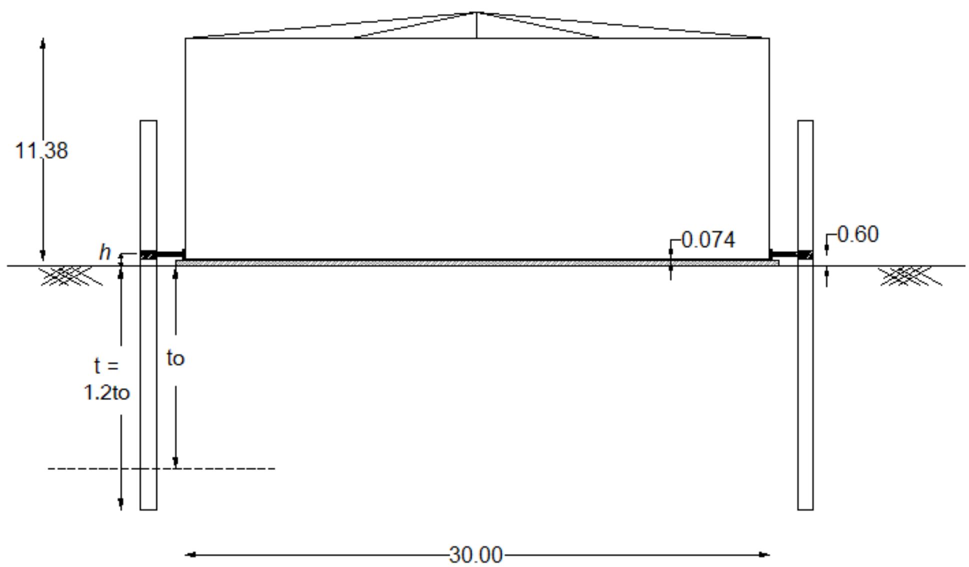


Figure 8-19 Overall dimensions of the tank and the bottom plate of the tank.

<u>Number of piles</u>

The piles are assumed to be of circular steel hollow sections.

The assumption for a thickness of the hollow sections is based on the following rules of thumb:

- t > 1/100D to avoid piling problems
- t > 1/80D to avoid buckling problems

The number of piles is determined with the theory of Blum, where the pile diameter and number of piles are chosen according to the balance of moments.

The following assumptions/notes are made beforehand and refers to Figure 8-19:

- The height *h* at which the tank is connected to the piles depends on the floating height of the tank (in reference with the ground level). The tank is assumed to be connected at 0.3m above the bottom of the tank. In this way the connection is close to the stiffened bottom plate. As can be seen in the previous section, the thickness of the bottom plate is 74mm (taking the steel plates of 6 and 8mm into account). So if the tank is still on the ground the height *h* is equal to 0.3 + 0.3 = 0.6m. This can be seen in Figure 8-19.
- Assuming 10 circular hollow sections with a diameter of 800mm.
- Thickness > 1/80 diameter = 10mm. A thickness of 20mm is chosen.
- t_o is the depth where the moment of the ideal load is zero and is at first assumed to be 5m.

The piles have to withstand the forces working on the tank and on the piles themselves. The loads are presented in Figure 8-20.

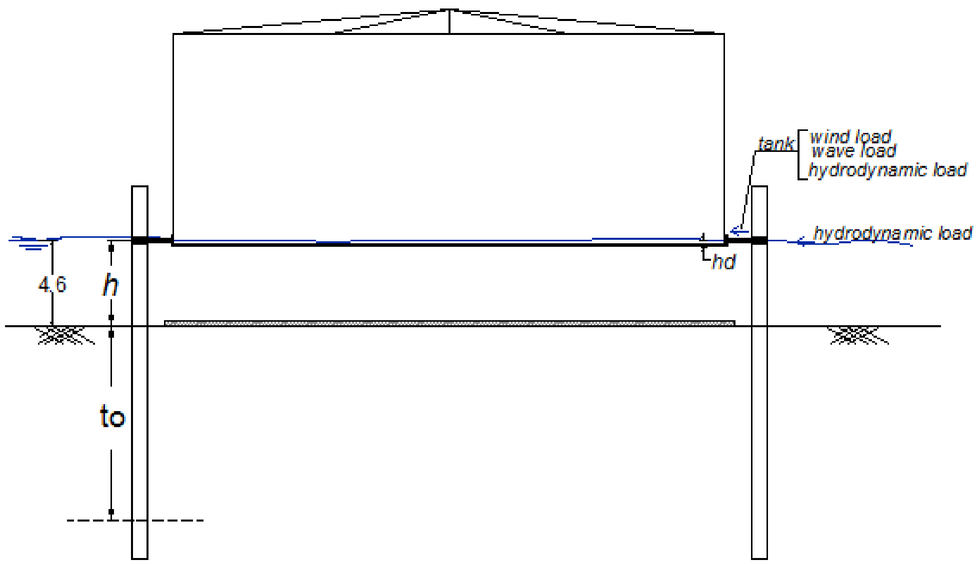


Figure 8-20 Loads working on the tank and the pile

According to the example calculation, piles with a diameter of 800mm and an embedded length of 6m (to*5m) are not sufficient enough to transfer the forces to the ground. An iterative calculation is done for the calculation of the balance of moments.
The depth t_o which is required for the moment equilibrium is determined for the diameters 0.8m and 1.0m and the selected number of piles *n* of 8 and 4.
With these 4 diameter/number of piles combinations, the thickness of the pile is also checked. This is done by checking the momentcapacity of the pile. Here it is assumed that the piles have a steel grade of S355 (the representative yield strength = 355N/mm^2). A unity check is done with the formulas given in Appendix XI on page 11-148.
The assumed thickness of 20mm is for non of the above combinations sufficient to withstand the working moment, therefore the required thickness is determined. The results and an example calculation are presented in appendix XIII E on page 11-148.

Based on the number of piles and required thickness, the following pile dimensions are chosen:
- Diameter: d = 1m (1000mm)
- Theoretical embedded depth: t_o = 9.5m
- Number of piles: n = 4
- Thickness of the hollow section: d = 48mm

This results in:
- The embedded depth: t = 1.2* t_o = 11.4m
- Length of the pile (from the bottom of the pile to the connection point of the tank): t + h = 11.3 + (4.6-0.38+0.3) = 15.92m
- Total length of the pile (+ extra safety height of 0.5m): 15.92 + 0.5 = 16.42m

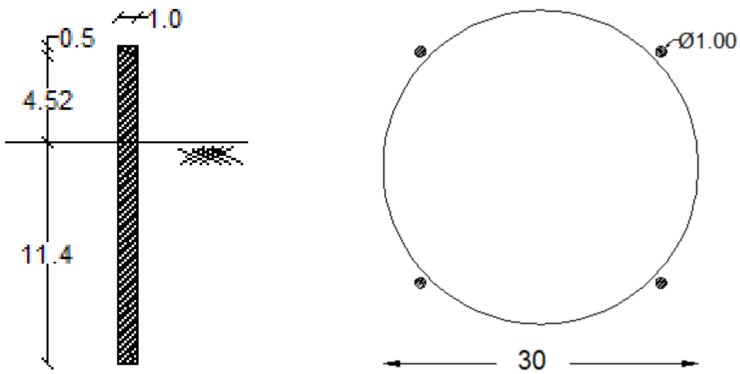


Figure 8-21 Overall pile dimensions & number of guiding piles

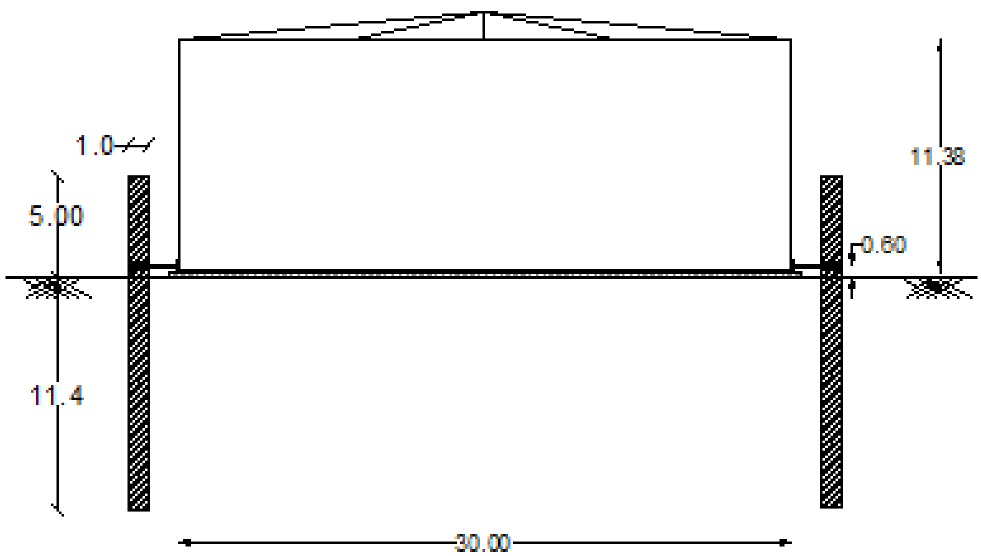


Figure 8-22 Overall dimensions of Tank C in meters

This tank alternative is called **Tank C** in the remainder of the report.

8.3.2 Building procedure

A proposition is made with regard to the building of this type of storage tank. The building procedure can be summed up in steps:

- Welding of steel plates together to form 2 circular plates for the upper and bottom part of the SCS slab. The bottom plate should be larger than the upper plate, for the connection of the shell.
- Welding of the shear studs on both plates according to the required spacing between the studs.
- On site, the bottom plate of the SCS slab can be placed on the foundation.
- The plates for the lower course of the shell can be welded to the bottom plate of the SCS slab. This can be done on both sides of the wall structure.
- The upper plate of the SCS slab can be positioned on top of the bottom plate with the use of steel or concrete separators to maintain the height of the concrete core.
- Some openings can be made in the bottom plate for the pouring of concrete between the steel plates for the concrete core.

- Welding of the remaining part of the shell structure
- Installation of the roof
- Installation of piles for the guidance of the tank
- Connecting the piles to the tank
- Connecting of pipelines to the tank

8.3.3 Elaboration on details

Pipeline & connections

Pipelines should be modified to cope with the floating of the tank. To prevent spill from pipelines during uplift of the tank, the following can be done:

- The pipelines should be able to follow the tank as it is floating; this can only be done if the pipeline is made flexible and/or compressible. The pipeline should be able to follow the tank if it is floating.

Figure 8-23 [left & mid] Example of flexible pipelines of tanker loading systems [13]; [right] Example of flexible and compressible pipelines[14]

- The pipes can exist of a safety mechanism, where the pipeline breaks at a specified point if the tank has reached a certain height. The pipeline can be closed off at either side of the breaking point.

Tank-column connections

The tank-column connection should not be made rigid. It can be fastened to the tank and provided with a circular section with rollers for it to be able to guide the tank as it floats.

[13] http://issuu.com/permartin/docs/0045_pusn_br_loading_syst?e=1737322/4206562#
[14] http://news.thomasnet.com/fullstory/Polyurethane-Hose-is-offered-with-thermally-molded-soft-cuff-819431

Figure 8-24 Example of the connection of floating jettie to piles

The part of the connection that glides up and down on the pile can be made in such a way that the concentrated force on the pile can be reduced. An option is to increase the height of the circular part around the pile. This way the forces are transmitted over a larger area to the pile, which in turn increases its allowable load capacity.

The connection on the tank is positioned at the lower part of the shell wall. This part is stiffer compared to the upper part of the shell, because of the stiffened bottom plate. This is done to withstand the concentrated force from the pile. If needed, the shell can be made a bit thicker to prevent buckling in the lower part of the shell.

<u>Issue when coming down</u>

An issue with Tank C is the accumulation of debris under the tank if the tank is floating. If there is debris under the tank, the tank cannot return onto its original position on the foundation.
An option would be to stretch a wire mesh around the columns (on the outside). This can be done in the lower part of the column and prevents the debris from getting under the tank. This is however just an idea and further research should be done to know if this method will hold the debris outside.

Phase 3 Concluding analysis

9 Simplified Cost Benefit Analysis (CBA)

This chapter leads to a conclusion on the feasibility of the implementation of the 3 alternatives for the tank design. A simple cost benefit analysis has been made involving the investments and benefits of these alternatives. An introduction is given describing requirements for the CBA. A layout proposal is given to indicate the number of tanks for the new terminal. After this the costs of the tanks, flood wall and levee are calculated. Before the CBA is done an assumption is made on the damage, probability of failure before and after investing in the storage tank. The CBA is done with 2 scenarios, construction of a levee around the terminal with the current tank design or the construction of flood proof storage tanks. The results are compared and a conclusion is drawn on the feasibility of the flood proof storage tanks.

9.1 Introduction

A simple cost benefit analysis is done to get an indication of the relation between the investments (costs) and the benefits of the alternatives. A simplified CBA is done where the following holds: costs < benefits.

Simplified:

- Costs = investment
- Benefits = risk reduction

Assume

- Investment: I
- Probability of failure before investment: $P_{f,0}$
- Probability of failure after investment: $P_{f,N}$
- Damage in case of failure: D
- Probability of failure after investment: $P_{f,N}$
- Damage in case of failure: D

CBA criterion:

$I < (P_{f,0} - P_{f,N})\, D$

The CBA for the new terminal is done for 2 scenarios with the given 3 variants. The following scenarios are elaborated:

- <u>Scenario 1</u>
  - Floodwall construction in year 1
  - Tank construction (not flood proof) of 20 tanks (see chapter 9.2). Construction of 4 tanks per year gives a total of 5 years.
- <u>Scenario 2</u>
  - Tank construction (flood proof) of 20 tanks (see chapter 9.2). Construction of 4 tanks per year gives a total of 5 years.

9.2 New terminal layout proposal

Before a comparison can be made, a layout of the new terminal is proposed, with the following assumptions:

1. The area of the new terminal is approximately the same as the current one, which is approximately 18600 m^2 (estimated from Figure 3-3 &Figure 3-7).
2. The tanks are all of the same size, with a diameter of 30m.
3. Space between the tanks is 5m.

4. The total required storage capacity of the terminal is the same as the current one, which is 308636 m^3

With the assumptions stated above, the number of tanks that can be situated on the terminal is 20.

9.3 Tank costs

9.3.1 Current tank costs

The costs of a tank with a size (diameter=33m, height=10m) similar to that used for the concept design in Chapter 8 are approx. $ 1.5 million[15]. These costs are including painting, lining, insulation and calibration and hydro testing.

9.3.2 Modified tank costs

The costs calculated for the flood proof tanks are costs of materials which are required on top of the current tank design to make the tanks flood proof. The subtotal of the costs is summed up with the costs of the current tank design to get the total costs of each tank variant.

[Note: Soil modification is not taken into account.]

[15] Repairs estimate: Stolt-Nielson

Variant 1: Tank A (with SCS sandwich slab inside the tank)

Materials	**Amount**		**Unit rate**		**total**
Concrete slab (thickness=2.76m)	1951	m3	€ 300.00	m3	€ 585,278.71
Foundation (thickness=0.25m)	189	m3	€ 300.00	m3	€ 56,607.57
steel plates16 (thickness=6+8mm)	0.78	ton	€ 3,000.00	ton	€ 2,330.51
shear connectors17	22620	piece	€ 1.00	piece	€ 22,620.00
stiffeners	289	ton	€ 3,000.00	ton	€ 865,618.73
				Subtotal in €	€ 1,532,455.53
			further detailing	30%	
			engineering	4%	
			indirect costs	15%	
			legal charges	2%	
			unforeseen	10%	
				61%	€ 934,797.87
				Total in €	€ 2,467,253.40
				Conversion € to $	$ 3,289,671.20
			+ $ 1.5 million	Total in $	$ 4,789,671.20

Table 9-1 Rough approximation of the extra costs for Tank A

The extra costs for the implementation of tank A are $2,875,423.93. Adding up the basic costs of the current tank design ($ 1.5 million), the total costs are $ 4,375,423.93.

If tank A is combined with the tank operations, the costs are calculated for a given example:
Assume that a concrete slab of half the required thickness is used, this is equal to 0.5 x 2.76m = 1.38m. In Figure 8-6, the required amount of stored liquid related to this concrete thickness can be read. This is equal to approx. 2.25m of stored liquid per storage tank. For 20 tanks the volume is equal to 31800m^3. This amount is approx. 10% of the total storage capacity (see point 4 in section 9.2).

With 50% less concrete thickness, the subtotal costs of the storage tank becomes approx. $2,456,624.50. This is 15% lower than if only concrete is used.

[16] http://www.alibaba.com/product-detail/ASTM-Standard-A36-Grade-304-Stainless_1697809444.html?s=p
[17] http://www.alibaba.com/product-detail/Shear-Stud-Connectors_1851998889.html

Variant 2: Tank B (with foundation anchored to the bottom of the tank)

Materials	Amount		Unit rate		total
Concrete slab/ foundation (thickness=2.76m)	1951	m^3	€ 300.00	m^3	€ 585,278.71
steel plates (thickness=8mm)	0.44	ton	€ 3,000.00	ton	€ 1,331.72
shear connectors	3491	piece	€ 1.00	piece	€ 3,491.00
stiffeners	289	ton	€ 3,000.00	ton	€ 865,618.73
				Subtotal in €	€ 1,455,720.16
			further detailing	30%	
			engineering	4%	
			indirect costs	15%	
			legal charges	2%	
			unforeseen	10%	
				61%	€ 887,989.30
				Total in €	€ 2,343,709.46
				Conversion € to $	$ 3,124,945.95
			+ $ 1.5 million	Total in $	$ 4,624,945.95

Table 9-2 Rough approximation of the extra costs for Tank B

The extra costs for the implementation of tank B are $ 2,710,698.69. Adding up the basic costs of the current tank design ($ 1.5 million), the total costs are $ 4,210,698.69.

Like Tank A if tank B is combined with the tank operations, it would also require 2.25m of stored liquid.

Variant 3: Tank C (floating tank)

Materials	Amount		Unit rate		total
Concrete slab (thickness=60mm)	42	m^3	€ 300.00	m^3	€ 12,723.45
Foundation (thickness=0.25m)	189	m^3	€ 300.00	m^3	€ 56,607.57
steel plates thickness=6+8mm)	0.78	ton	€ 3,000.00	ton	€ 2,330.51
shear connectors	22620	piece	€ 1.00	piece	€ 22,620.00
piles [16.42m x 4]	65.68	m	€ 1,500.00	m	€ 98,520.00
Pile-driving installation			€ 5,000.00		€ 5,000.00
Pile driving [16.42m]	4	piles	€ 2,000.00	8piles/ day	€ 1,000.00
				Subtotal in €	€ 198,801.53
			Modified connections/ pipelines	5%	
			further detailing	30%	
			engineering	4%	
			indirect costs	15%	
			legal charges	2%	
			unforeseen	10%	
				66%	€ 131,209.01
				Total in €	€ 330,010.55
				Conversion € to $	$ 440,014.06
			+ $ 1.5 million	Total in $	$ 1,940,014.06

Table 3 Raw approximation of the extra costs for Tank C

9.3.1 Flood wall

According to the study conducted by Royal HaskoningDHV the construction of a 8m high, 1.5km floodwall around the current terminal at Stolthaven will cost about €22,50 - €30 million (≈ $35 - 40 million [1$ ≈ €0.75][18])

9.3.2 Levee

To give a rough approximation of the costs a levee the same height and length as the flood wall are assumed. For the calculation, the following input is used:

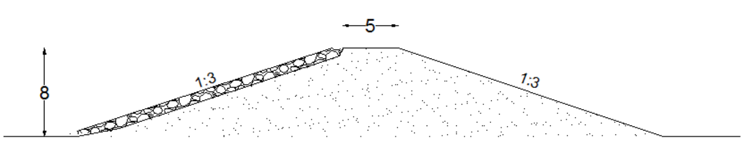


Figure 9-1 Assumption for levee layout

[18] http://www.bloomberg.com/markets/currencies/currency-converter/

Levee	
height	8m
length	1500m
slope	1:3
base	5m
settlements	2m
rock revetment	0.5m
	200kg

Table 9-3 Input for cost calculation of a levee

Materials	Amount		Unit rate		total
soil[19]	525000	m^3	€ 15.00	m^3	€ 7,875,000.00
dry excavation, including slope leveling and transport[20]	525000	m^3	€ 4.00	m^3	€ 2,100,000.00
Rock revetment	47434	ton	€ 75.00	ton	€ 3,557,562.37
Placing of revetment[21]	23717	m^3	€ 50.00	m^3	€ 1,185,854.12
Geotextile[22]	47434	m^2	€ 3.00	m^2	€ 142,302.49
				Subtotal in €	€ 14,860,718.98
			further detailing	30%	
			engineering	4%	
			indirect costs	15%	
			legal charges	2%	
			unforeseen	10%	
				61%	€ 9,065,038.58
				Total in €	€ 23,925,757.57
				Conversion € to $	$ 31,901,010.09

Table 9-4 Rough approximation of the costs of a levee

9.4 Benefits

To calculate the benefits the probability of failure before ($P_{f,0}$) and the probability of failure after ($P_{f,N}$) the modifications on the tank design should be known. This probability difference multiplied with the damage costs in case of failure, is the (expected) benefit.

A ten-year study indicates that the main cause of accidents on tank farms was storage tank failure. Other causes of accidents were operator error, valve failure, pump failure and bolt fitting failure. If we take a look at storage tanks, there are a number of causes leading to failure.

[19] Leslie Mooyaart
[20] CIE4170: Case study: Overview costs
[21] http://www.loranet.org/faqrevet.htm#How Much Does A Revetment Cost
[22] CIE4170: Case study: Overview costs

Some of them are human error, poor maintenance, vapor ignition, differential settlement, earthquake, lightening strike, hurricane, flood damage and over-pressurization. Since this thesis mainly focuses on hurricane and flood events, other failure causes are left aside.

The probability of failure of storage tanks due to flooding should be determined by setting up a failure tree with all possible failure modes regarding the presence of flooding. Of course the failure due to flooding is one of many failure modes of a storage tank. Now only flooding is brought into perspective.

The probability of failure of storage tanks due to flooding depends on the following modes:
[Note: This list is not limited to these modes, therefore a thorough analysis should be done]

- The flood intensity on the terminal site. This depends on:
  - The overtopping rate of the flood wall
  - The velocity of the flood water
  - The height of the flood water
- The structural capacity against a certain flood intensity. The structure is able to withstand a certain degree of water height and velocity and is defined by:
  - The diameter of the tank
  - Fill level of stored material

Since the calculations of the CBA will give estimated values from which an indefinite conclusion will be drawn on the feasibility of the variants, the probabilities are assumed and not calculated.

9.4.1 Probability of failure before ($P_{f,0}$)

The current probability of failure of the storage tank is assumed, given the occurrence of a flood event (per year). In this case, the probability of failure is equal to the safety level of the GoM Levee (Back Levee) (see **Error! Reference source not found.**). Here, it is assumed that the storage tank fails when the GoM Levee gets overtopped/overflowed and the terminal gets flooded. Section 3.4.2, shows that storage tanks can already lift off their foundation if a small excess of water is present on the terminal. The probability of failure of a storage tank (current design) on the new terminal is therefore set equal to the existing terminal, which is the same as the safety level of the GoM Levee. $P_{f,0}$ = 1/5 (see section 3.1).

9.4.2 Probability of failure after ($P_{f,N}$)

Catastrophic failure of storage tanks related to accidents of a certain nature, such as an explosion, are unlikely. The probability of occurrence of such accidents is in the order of 5.10^{-6} per year. [23]
Since the determination of the failure probability of the flood proof storage tank will be time consuming, it is assumed that the probability of failure of the flood proof storage tank is also in the order of 5.10^{-6} per year. Considering the case that the tank is now flood proof and that the failure of the tank due to flood is now unlikely to occur.

9.4.3 Damage (D)

Chemical spill is caused if the tank is damaged. This can already happen if the structural damage to the tank is small. These damage costs related to the structure will be small compared to a total collapse of the tank. Like mentioned in section 2.5, due to environmental damage the cleanup costs will be high. For now, the damage costs for one storage tank is assumed. The damage costs include:

- Cut down of tank in residue
- Haul pieces to onsite cleaning station

[23] Review of failures, causes & consequences in the bulk storage industry

- Transport for offsite disposal
- Cleaning/decontaminate environment/ground
- Tank purchase
- Tank/ terminal downtime

The damage costs, structural and environmental, of the current terminal was approximately $300 million. The damage costs in case of failure and chemical spill for one storage tank is assumed to be $6 million (4 times the current tank costs).

9.5 Analysis

The CBA analysis is done for the 3 variants of the flood proof tank. The cost-benefit analysis is done for 20 years.
It should be noted that the benefits are yearly costs, whereas the investments are once-only costs. The yearly costs do not have the same value as the once-only costs. The yearly costs are therefore discounted to the present value by means of the net present value (NVP). To do this a discount factor is needed. This is assumed to be 2.5% per year. The NPV is determined with the following equation:

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

where:

T	= Total considered time	[years]
C_y	= Yearly costs	[$/€]
r	= discount rate	[%]
t	= considered year	[year]

The CBA is done with 2 scenarios, where the 3 variants of the flood proof tank are implemented in scenario 2. The scenarios are:

- Scenario 1
  - Floodwall construction in year 1
  - Tank construction (not flood proof): 20 tanks => 4 tanks per year => 5 years

  For the first scenario the new terminal is protected against flooding with the construction of a floodwall. The construction is assumed to take place in 1 year. The tanks on the terminal are from the current design (not flood proof). Approximately 20 tanks with a diameter of 30m can be situated on the terminal site. With a construction of 4 tanks per year, the total construction time is 5 years.

- Scenario 2
  - Tank construction (flood proof): 20 tanks => 4 tanks per year => 5 years

  For the second scenario the tanks on the terminal are flood proof. This means, the terminal does not have to be protected against flooding. Here, the construction of 20 tanks will also take place in 5 years, with the construction of 4 tanks each year.

The results for the CBA for both scenarios and the tank variants is given in the next table. The NPV is calculated from the net costs of that certain year. The net costs is found by subtracting the investment costs from the benefits. The Excel sheet with the calculation is given in Appendix XVI.

Basic assumptions	**Tank A**	**Tank B**	**Tank C**
Costs tank (without flood protection)	$ 1,500,000.00	$ 1,500,000.00	$ 1,500,000.00
Costs flood wall	$ 40,000,000.00	$ 40,000,000.00	$ 40,000,000.00
Extra costs tank (with flood protection)	$ 3,289,671.20	$ 3,124,945.95	$ 2,174,382.06
Costs tank (with flood protection) = Costs tank (without flood protection) + Extra costs tank	$ 4,789,671.20	$ 4,624,945.95	$ 3,674,382.06
Damage costs per tank	$ 6,000,000.00	$ 6,000,000.00	$ 6,000,000.00
$P_{f,0}$	1/5	1/5	1/5
$P_{f,N}$	5.10^{6}	5.10^{6}	5.10^{6}
$P_{f,0}$ - $P_{f,N}$	0.199995	0.199995	0.199995
Discount rate	2.5%	2.5%	2.5%
NPV Scenario 1 of 20 years	€ 261,531,371	€ 261,531,371	€ 261,531,371
Flood wall first year			
Tanks without flood protection, 4 every year. NPV for 20 years.			
NPV Scenario 2 of20 years	€ 239,422,768	€ 242,483,909	€ 292,378,841
Tanks with flood protection, 4 every year. NPV for 20 years.			

Table 9-5 Basic assumptions and results for scenario 1 and scenario 2 for all 3 variants of flood proof tanks

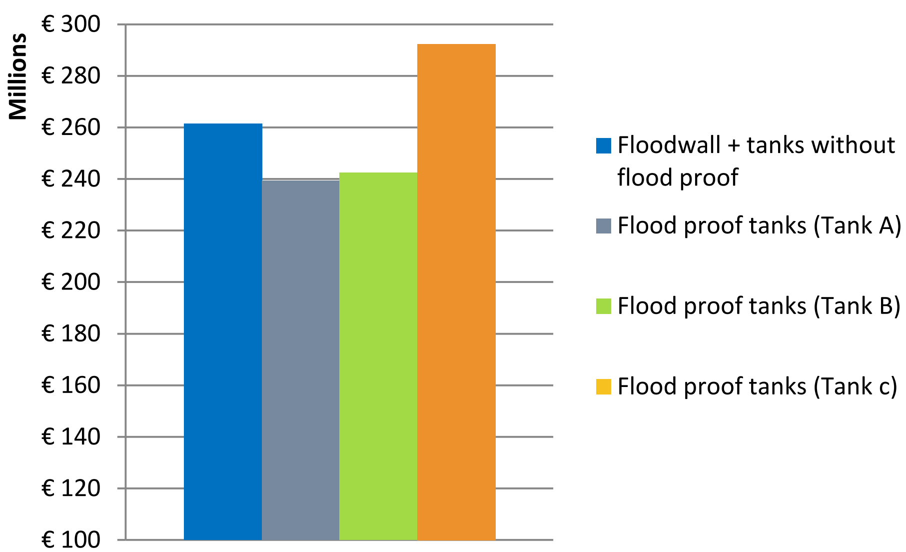


Figure 9-2 Results of the net present value of 20 years

The results show that the net present value of Tank C (floating tank) has the highest value. With this highest NPV, this alternative seems to be the most promising for implementation at the new terminal.

Keeping in mind that Tank A and Tank B are not ruled out, because the NPV of all scenarios are of the same order of magnitude and lie between €230 million and €300 million for a period of 20 years.
The flood proof storage tank is compared to the basic solution as the flood wall a flexible alternative, because the tank does not need to be protected against floods.

10 Conclusions & recommendations

This thesis is about the "Design of a flood proof storage tank". In this last chapter of the report the findings and recommendations of the thesis are presented.

10.1 Conclusions

- ***API Standard 650 provides insufficient information regarding flood loads***
  After having a number of tank failures on different tank terminals involving floods it can be concluded that the API standard 650 does not seem to provide sufficient information regarding the failure modes and especially uplift of storage tanks during a flood event whereas design requirement in the event of earthquakes are intensively provided. The given information on the required fill level is not sufficient, due to the fact that the amount of required inventory can be unreliable at the time of a flood.
- ***Light self-weight results in sensitivity to floating***
  The structure of the storage tank is made out of thin steel plates and therefore has a light weight. This makes the structure sensitive to floating in the event of a flood.
- ***Additional weight or guiding piles are the main requirements for the flood proof tank***
  Tank A (storage tank with SCS slab) and Tank B (storage tank with foundation as ballast) require an additional concrete height of 2.76m. For Tank C (floating tank) the required length of the piles is approx. 16.4m for a number of 4 piles around the tank.
- ***Implementation of flood proof storage tank may require soil modification***
  Tank A (storage tank with SCS slab) and Tank B (storage tank with foundation as ballast) are due to the large amount of additional weight too heavy for the subsoil. Soil modifications may be required for the new terminal when implementing one of these tanks.
- ***Circumferential stiffeners are required to prevent buckling (regarding wind pressure)***
  The storage tanks should be designed with the basic wind speed which is valid for the area where the terminal is located. In this case this is 259 km/h and corresponds to a maximum wind pressure of $2.8\frac{kN}{m^2}$. With the chosen thinnest shell course of approx. 6mm, the critical buckling pressure is lower. Due to this stiffeners are needed around the circumference of the tank to prevent buckling of the storage tank or thicker shell walls should be applied.
- ***Circumferential stiffeners are required to prevent buckling (regarding hydrostatic pressure)***
  The hydrostatic load originated from a 1/100y flood event is in this case approximately $46kN/m^2$. This pressure can be fatal to the tank structure, if the tank is empty. Like mentioned in the previous conclusion, stiffeners are in this case also required or thicker shell walls to prevent buckling.
- ***Smaller tank diameters have a greater buckling resistance***
  According to the weighted smeared method, the critical buckling pressure of a storage tank increases with almost 300% if the diameter of the tank is made 50% smaller. So tanks with smaller diameters have a greater resistance against buckling.
- ***Combination between tank operations and Tank A and Tank B***
  With the use of a maintained inventory level of chemicals inside the tank, the concrete thickness required for Tank A and C can be reduced. With a 2.5m liquid height inside the tank the amount of concrete reduces with approx. 50%.
  Filling of tanks to prevent uplift appeared to be unreliable in previous cases.
- ***Approx. 30% higher costs for Tank C and more than 200% for Tank A and Tank B***
  The rough approximation of the costs of the variants show that the floating tank (Tank C) can be 30% more expensive than the costs of the current storage tank. For the other 2 alternatives this is more than 200%.

- ***The floating tank (Tank C) is more beneficial***
  Implementing the flood proof tank on this site seems to be more beneficial than the other 2 variants and the construction of the basic solution with the flood wall and tanks with the current tank design. It has a higher NPV than the other 3 scenarios.
- ***NPV's of all 4 scenarios are of the same order of magnitude***
  Tank C, the floating tank has a higher Net Present Value, but the NPV of all 4 scenarios are of the same order of magnitude and lies between €230 million and €300 million for a period of 20 years. Due to this, the implementation of the other scenarios is not ruled out. The flood proof tank is more flexible than the construction of the flood wall.

10.2 Recommendations

- ***Adjustment of API Standard 650 regarding flood loads***
  Specific design requirements for storage tanks in events of hurricanes and floods should added to the API Standard 650. This should include the working loads, safety factors, load combinations, specific structural checks.
- ***Thorough soil investigation***
  A soil investigation is required for the new terminal. If needed, soil improvements should be made in order to implement Tank A or Tank B.
- ***Combination between tank operations and Tank A and Tank B***
  Filling of tanks to prevent uplift appeared to be unreliable in previous cases. A study should be done to increase the reliability of this method and ways to properly fill the tank. Due to this, the combination of this method with the alternatives A or C can be made favorable.
- ***Determination of the probability of failure of the flood proof tank***
  The probability of failure of the flood proof storage tank (all 3 alternatives) is assumed to be 5.10^{6} per year. This is done to get a rough approximation for the CBA. It is recommended to do a thorough calculation of the failure probability of the improved tanks taking into account all failure modes. These tanks are modified in a manner to withstand flood loads, but it is possible that this influences other failure modes.
- ***A more accurate determination of the investment and damage costs***
  The CBA is done with assumed investment and damage costs of the storage tanks. The investment costs should be more accurately determined and the damage costs, especially environmental damage caused by spill of chemicals should be predicted more precisely.
- ***Testing before implementing the flood proof storage tank***
  Small scale tests are required before implementing the flood proof storage tank in practice. This is an innovative solution, which is why tests should be done to determine all failure modes.

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11 Appendices

Appendix I Water level and waves

Water level

Near the levee in the Gulf of Mexico (GoM) and at the Mississippi River (MR) water level statistics are gained from a Joint Coastal Surge Study for Louisiana. It was conducted by the US Army Corps of Engineers after Hurricane Katrina. The study focuses on return periods between 1/50 – 1/500 years and presents results of surge levels and wave statistics of a combination of detailed surge modeling and a probabilistic analysis. [15] With a regression analysis a relation between the water level and its frequency is found. Numbers for lower and higher return periods have been found via extrapolation. [17]

Return period	Water level near GoM	Water level near GoM (Weibull-distribution k=1.56, λ=6.87)
[year]	[m+NAVD88]	[m+NAVD88]
5		2.8*
10		3.6*
50	4.6	5.0
100	5.6	5.6
200	6.2	6.1
500	6.9	6.7
1,000	7.3	7.2
2,000	7.6	7.7*
10,000		8.7*

*These numbers are extrapolated outside the limits of the modelling results and the accuracy of these numbers is much less.

Figure 11-1 Water level statistics for the Gulf of Mexico (JCS study and Weibull-distribution)

For water levels with low return periods (1/5 and 1/10 years) the back levee (2.6m +NAVD88) will partially hinder the inflow into the small Braithwaite polder. For this matter the surge level statistics at the terminal will differ from the statistics at the Gulf of Mexico. For surge levels with return periods higher than 50 years, the back levee will be overpowered by hurricane surge. Therefore the surge level near the terminal will be the same as the surge statistics at the Gulf of Mexico.
The water level (flood level) with return period of 100years is used in the remainder of the thesis. This is equal to NAVD88 +5.6m.

Return period	Water level near MR	Water level near MR (Weibull k=1.04, λ=3.39)
[year]	[m+NAVD88]	[m+NAVD88]
10		2.3*
50	3.7	3.8
100	4.8	4.5
200		5.1
500	5.8	6.0
1,000		6.6*
2,000		7.3*
10,000		8.7*

*These numbers are extrapolated outside the limits of the modelling results and the accuracy of these numbers is much less.

Figure 11-2 Water level statistics in Mississippi River due to storm surge only (no river flooding included)

Waves

The waves in front of the GoM Levee are predicted by Royal HaskoningDHV using the 1D SWAN Model (Simulating WAves Nearshore).

The findings of this study are presented below.

The wave heights are calculated for one transection which starts at the Gulf of Mexico runs over the wetlands and the back levee into the Braithwaite polder and ends behind the Mississippi river. The two shallow swamp pools, Lake Lery and the Big Mar in front of the GoM Levee are also included. These swamp pools will enhance the wave growth in the wetlands.

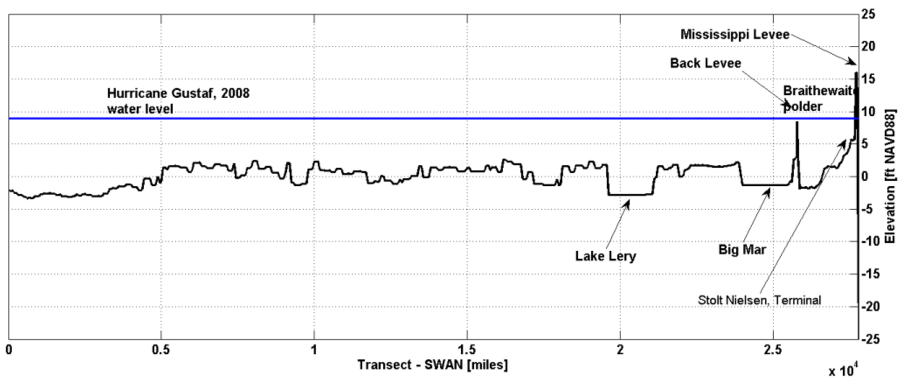


Figure 11-3 Representation of the cross-section of the 1d SWAN model

The calculations are based on the following parameters:

– Bathymetry

– Bottom friction.

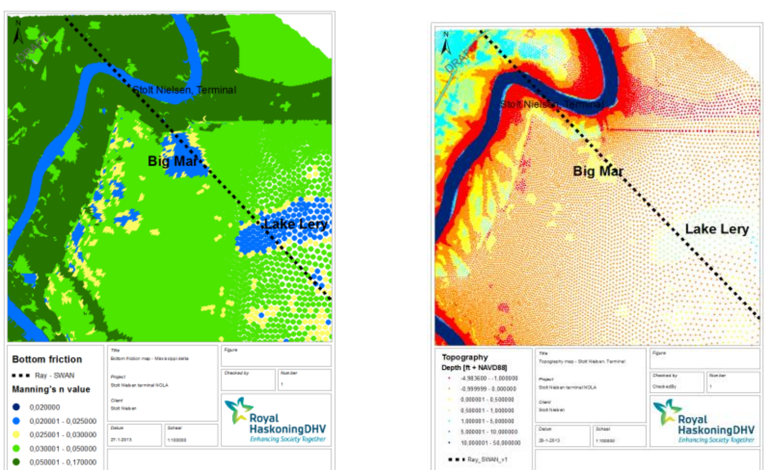


Figure 11-4 Bathymetry and bottom friction of the project area

– Wind speeds and water levels. These are based on return periods of 1/10, 1/50, 1/100, 1/500 and 1/1000. The wind speeds are gained from the design report of HSDRRS (Hurricane Storm and Damage Risk Reduction system) (2007) and the water levels are gained from the technical report of LACPR (Louisiana Coastal Protection and Restoration) (2011).

Return period [1/yr.]	Water level [NAVD 88 m+]	Wind speed [km/h]
10	3.7	75.6
50	4.6	112.7
100	5.6	124.0
500	6.4	140
1000	7.3	160.9

Figure 11-5 Return periods, water levels and wind speeds of the Louisiana coast according to HSDSSR

For all 25 possible combinations of wind speed and water level, the wave height is simulated. It is considered that the return periods of the wind speed and the water level are about the same, these are shown in bolt in the table below. The table below shows the results of the wave study.

WAVE HEIGHT	Water level [NAVD88m+]	3.7	4.6	5.5	6.4	7.3
Wind speed [km/h]	Return period [1/yr.]	10	50	100	500	1000
75.6	10	0,18	0,21	0,27	0,27	0,43
112.7	50	0,27	0,27	0,34	0,37	0,58
124.0	100	0,30	0,34	0,34	0,40	0,61
140	500	0,37	0,37	0,40	0,43	0,67
160.9	1000	0,43	0,46	0,46	0,46	0,76

Figure 11-6 Simulated wave heights for all combinations of the water levels and wind speeds

The wave height with return period of 100years is used in the remainder of the thesis. This is equal to NAVD88 +0.34m.

Appendix II **Bore log**

The ground profile of one bore log of the existing terminal is given below.

Sample No.	SAMPLE Depth in Feet From	SAMPLE Depth in Feet To	STRATUM Depth in Feet	VISUAL CLASSIFICATION	* Blows per Foot	Symbol Log	Scale (feet)	UNCONFINED COMPRESSION (Qu) (lbs./sq.ft.)	WATER CONTENT (percent)	UNIT WEIGHT (lbs./cu.ft.) DRY	UNIT WEIGHT (lbs./cu.ft.) WET	ATTERBERG LIMITS L.L.	ATTERBERG LIMITS P.L.	ATTERBERG LIMITS P.I.
1	.0	.5	.0 – 1.0	GRAVEL, SHELL, BRICK & SOME CLAY (FILL)			0							
2	2.5	3.0	1.0 – 7.0	STIFF TAN & GRAY CLAY W/ SILT				3480	26.2	85.2	107.5			
3	5.5	6.0					5	2625	47.7	71.2	105.2			
4	8.5	9.0	7.0 – 9.5	SOFT GRAY & TAN CLAY W/ SILT				885	41.6	75.1	106.3			
5	11.5	12.0	9.5 – 22.0	VERY SOFT TO SOFT GRAY CLAY W/ SPECKS WOOD & ORGANIC			10	490	43.3	76.5	109.6			
6	14.5	15.0					15	675	55.8	65.1	101.5	70	25	45
7	19.5	20.0					20							
8	24.5	25.0	22.0 – 27.5	SOFT BROWN ORGANIC CLAY W/ WOOD			25	965	250.3	19.3	67.5	225	126	99
9	29.5	30.0	27.5 – 34.5	SOFT GRAY CLAY W/ SILT LENSES			30	840	94.6	46.2	89.9	116	51	65
10	33.5	34.0												
11	34.5	36.0	34.5 – 46.0	MEDIUM DENSE GRAY SILTY FINE SAND	13		35		22.5				(31)	
12	38.5	40.0			19		40							
13	43.5	45.0			10		45		22.2					
14	49.5	50.0	46.0 – 77.5	SOFT TO MEDIUM STIFF GRAY CLAY W/ SOME SILT			50	815	48.6	69.2	102.9			
15	54.5	55.0					55	905	43.0	73.9	105.7			
16	59.5	60.0					60	1125	61.6	60.8	98.2			
17	64.5	65.0					65	790	51.0	66.1	99.8	60	20	40
18	69.5	70.0					70							
19	74.5	75.0					75	765	59.2	60.6	96 5			
20	79.5	80.0	77.5 – 80.0	MEDIUM STIFF GRAY CLAY W/ SAND LENSES			80	1150	56.8	61.8	96.9			

Appendix III Tank roofs

The following classification is made:

- Fixed-roof tanks

The aboveground tanks mostly have cylindrical shapes on the part that contains fluid. This method is cost-effective and has an easy shape to fabricate for pressure containment. An essential aspect of such cylindrical tanks is that the top end must be closed. The relatively flat roofs do not lend themselves to much internal pressure. While internal pressure increases, the tank designers use domes or spherical caps.

- Cone-roof tanks

  These are the most frequently used tanks for storage of bulk fluids. The top is made in the form of a shallow cone and the bottom is generally flat. In large-diameters tanks the cone-roof tanks typically have roof rafters and support columns.


Figure 11-7 Cone-roof tank (left) and a cone-roof tank with column supports (right)

- Umbrella-roof tanks

  These are like the cone-roof tanks, but the roofs look like an umbrella. The umbrella-roof tank does not have support columns to the bottom of the tank and can therefore be seen as a self-supporting structure.

- Dome-roof tanks

  These have the same shape as the umbrella type, except that the dome comes closer to a spherical surface than the segmented sections of an umbrella-roof.

Figure 11-8 Dome-roof tank

- Aluminum geodesic dome-roof tanks
  Most tanks are made of steel, but some fixed-roof tanks have an aluminum geodesic dome-roof. Compared to steel roofs, these roofs have a higher resistance against corrosion. These roofs do not require internal support and can be built to basically any required diameter.

Figure 11-9 Aluminum geodesic dome-roof tank [31]

– Floating-roof tanks Internal floating roof (IFR)
  These tanks have a screen that floods on the surface of the liquid. The floating roof, as it is called, has enough buoyancy to guarantee that the roof will float under all expected conditions, even if leaks occur in the roof. The floating roof reduces the surface area of liquid that is open to the atmosphere and therefore reduces air pollution and evaporation losses.
  - External floating roof (EFR)
    These tanks are open on top.

Figure 11-10 EFR tank [32]

- Internal floating roof (IFR)
  These tanks have a fixed roof that ceils the floating roof.

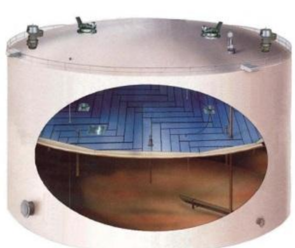


Figure 11-11 IFR tank [33]

Appendix IV General design considerations

The following section will be focused on the design considerations of tanks. It mainly applies to atmospheric steel tanks with a flat-bottom, but many of the principles can be extended to other types of tanks.
For the design considerations is referred to [6] & [23].

Shell

Thickness

The shell thickness shall not be less than the thickness given in the table below.

Nominal Tank Diameter		Nominal Plate Thickness	
(m)	(ft)	(mm)	(in.)
< 15	< 50	5	3/16
15 to < 36	50 to < 120	6	1/4
36 to 60	120 to 200	8	5/16
> 60	> 200	10	3/8

The thickness of the shell can be calculated by the 1-Foot method. This method calculates the required thickness at design points 0.3m (1ft.) above the bottom of each shell course. This method should not be used for tanks with a diameter larger than 61m (200ft.)
The minimum thickness of shell plates shall be the greater value of these 2 equations:

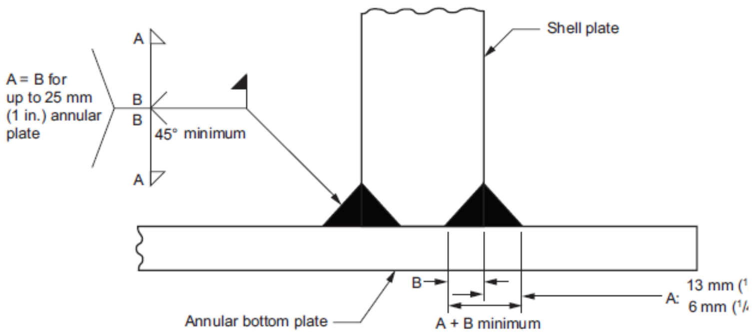
1. Thickness according to the stored liquid and corrosion allowance:
   $$t_d = \frac{4.9D(H-0.3)G}{S_d} + CA$$

2. Thickness according to the hydrostatic test:
   $$t_t = \frac{4.9D(H-0.3)}{S_t}$$

where:

t_d	= the design shell thickness	[mm]
t_t	= the hydrostatic shell thickness	[mm]
D	= the nominal tank diameter	[m]
H	= the design liquid level	[m]
	= bottom of the considered course to the top of the shell (liquid level)	
G	= the design specific gravity of the liquid to be stored	[kN/m^3]
CA	= the corrosion allowance	[mm]
S_d	= the allowable stress for the design condition	[kN/m^2]
	= the lesser value of 2/3 of the yield strength or 2/5 of the tensile strength	
S_t	= the allowable stress for the hydrostatic stress condition	[kN/m^2]
	= the lesser value of 3/4 of the yield strength or 3/7 of the tensile strength	

Shell-to-bottom connection



NOTE 1 A = Fillet weld size limited to 13 mm (1/2 in.) maximum.

NOTE 2 A + B = Thinner of shell or annular bottom plate thickness.

NOTE 3 Groove weld B may exceed fillet size A only when annular plate is thicker than 25 mm (1 in.).

Figure 11-12 Typical shell-to-bottom connection

Appendix V Surface roughness

Classification of the exposed area [surface roughness]

For every evaluated wind direction, an exposure category will be determined for the area that indicates the characteristics of ground roughness and surface irregularities in that area as much as possible. The surface roughness will be determined within a 45-degree region for a distance upwind of the site.
Due to the appearance of flat open land and the occurrences of hurricanes in the project area, the area of Stolthaven can be placed in the surface roughness category C.

Surface roughness categories	Characteristics
B	Urban and suburban regions, wooded regions or other areas with several closely spaced obstructions having the size of single-family residences or larger.
C	Open areas with spread obstructions with heights mainly less than 9.1m (31 ft.), including grasslands, flat open land and all water surfaces in hurricane-sensitive areas.
D	Flat, open areas and water surfaces outside hurricane-sensitive areas, including smooth mud and salt wasteland and unbroken ice.

Figure 11-13 Surface roughness categories

Appendix VI General on shells [34]

Tanks can be schematized as a vertical cylindrical shell. To get insight into the deflection op the shell of a fully filled tank, a tank can be related to a horizontal cylindrical shell, as shown in

Figure 11-14. From the equations of equilibrium of forces, as shown in Figure 11-15, the deformations of the shell can be determined. Here it is assumed that the deformations are symmetrical, the thickness of the shell is constant and that there is no change in curvature in the circumferential direction.

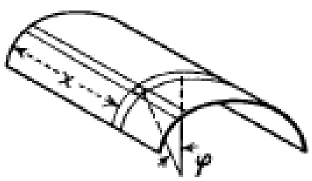


Figure 11-14 Horizontal cylindrical shell parallel to the x axis

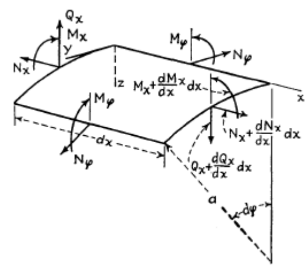


Figure 11-15 Forces acting on the sides of the element

The deformations of a cylindrical shell can be written as the following differential equation:

$$\frac{d^4w}{dx^4} + 4\beta^4 w = \frac{Z}{D} \qquad \text{with: } \beta^4 = \frac{Eh}{4a^2D} = \frac{3(1-v^2)}{a^2h^2}$$

with:

w	= deformation of the shell	[]
x	= length of the cylinder	[]
a	= radius of the cylinder	[]
D	= flexural rigidity of the shell = $\frac{Eh^3}{12(1-v^3)}$	[]
Z	= load intensity	[]
E	= modulus of elasticity	[]

The general solution to the equation above is:
$w = e^{\beta x}(C_1 \cos\beta x + C_2 \sin\beta x) + e^{-\beta x}(C_3 \cos\beta x + C_4 \sin\beta x) + f(x)$, in which f(x) is the particular solution and C1,…, C4 are the constants of integration.

If a tank is filled with liquid and therefore subjected to the action of liquid pressure, the stresses in the wall can be analyzed by the following:

$$Z = -\gamma_l(h_l - x)$$

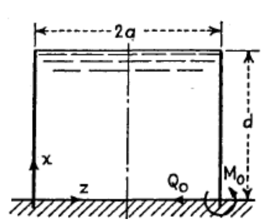


Figure 11-16 Tank subjected to internal liquid pressure

With:

Z	= load intensity	[]
γ_l	= specific weight of the liquid	[]
h_l	= height of the liquid inside the tank	[]
x	= height of the tank	[]

By combining the two equations above, the following is obtained:

$$\frac{d^4w}{dx^4} + 4\beta^4 w = -\frac{\gamma_l(h_l - x)}{D} \qquad 1$$

For a cylindrical shell with free edges under the action of hoop stresses, the equation below represents the radial expansion and is also the particular solution of the equation above.

$$w_1 = -\frac{\gamma_l(h_l - x)}{4\beta^4 D} = -\frac{\gamma_l(h_l - x)a^2}{Eh} \qquad 2$$

Now the general solution can be written as:

$$w = e^{\beta x}(C_1 \cos \beta x + C_2 \sin \beta x) + e^{-\beta x}(C_3 \cos \beta x + C_4 \sin \beta x) - \frac{\gamma_l(h_l - x)a^2}{Eh}$$

For most tanks the following can be stated:

– the thickness is small compared to the radius and the height

– the shell can be considered as infinitely long

This means that the constants C_1 and C_2 are zero.

Appendix VII **Loads**

This section focuses on the working loads on the tanks, such as internal pressure, loads present during hurricanes (wind, flood loads).
The loads are based on the 1/100 year design water level E_{sw} of NAVD88+4.6m (taking into account a +1m ground elevation).
Apart from wind, the tanks will then be loaded with an extra external pressure from the flood, like hydrostatic pressure and wave pressure.

VII A Internal loads

Hydrostatic pressure
Liquid inside a tank generates hydrostatic pressure against the shell wall and pressure on the bottom plate of the tank. Due to this pressure the shell wall is in tension.

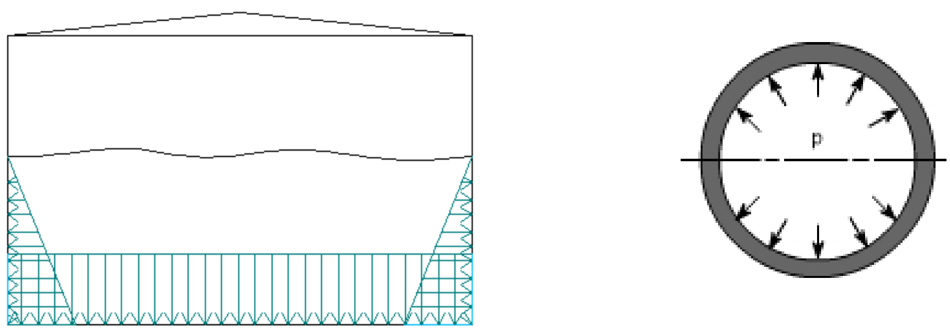


Figure 11-17 **Internal hydrostatic pressure *p* against the cylindrical shell**

The steel tank can be related to a cylinder which is internally loaded by a uniform pressure *p.*

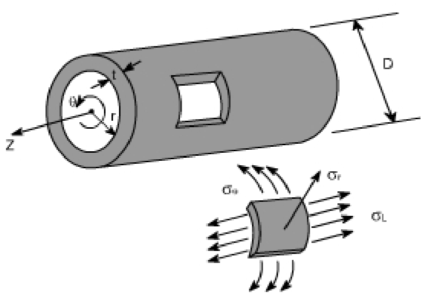
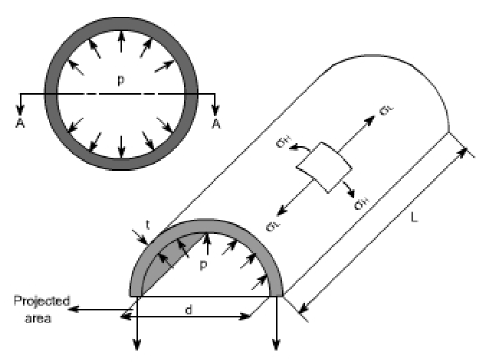


Figure 11-18 **Stresses in different directions**



A thin-walled cylinder which is subjected to internal pressure, the following stresses occur in the material[24]:

- Hoop or circumferential stress σ_H
  This is the stress in the shell which resists the cracking of the shell due to the applied pressure inside the tank. The following applies (see Figure 11-18):

[24] http://www.nptel.ac.in/courses/Webcourse-contents/IIT-ROORKEE/strength%20of%20materials/lects%20&%20picts/image/lect15/lecture15.htm

Force due to internal pressure = $p \times d \times L$

Force due to resisting hoop stresses = $2 \times \sigma_H \times L \times t$

Equilibrium of the cylinder:

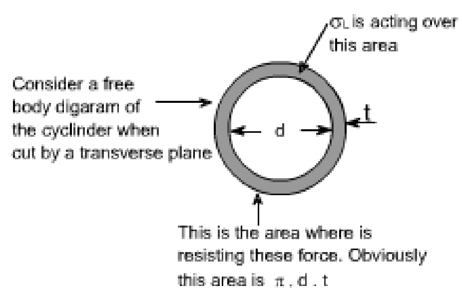
$$p \times d \times L = 2 \times \sigma_H \times L \times t \;\; \rightarrow \;\; \sigma_H = \frac{p \times d}{2t}$$

where:

p	= the pressure in the cylinder	[kN/m^2 or N/mm^2]
d	= the diameter of the cylinder	[m or mm]
L	= the length of the cylinder	[m or mm]
t	= the thickness of the cylindrical wall	[m or mm]
σ_H	= the hoop or circumferential stress	[kN/m^2 or N/mm^2]

For the case of a storage tank, the pressure *p* is not constant. The liquid inside the tank causes a hydrostatic pressure. This means that the pressure on the walls will increase with the depth of the liquid.

- Radial stress σ_R
  This stress acts normal to the curved plane and is negligible compared to the other two stresses, particularly if (thickness/internal radius) < 1/20
- Longitudinal stress σ_L
  Assuming that the cylinder has closed ends, the walls of the cylinder will then also be exposed to a longitudinal stress.



Force due to internal pressure at the ends = $p \times \frac{1}{4}\pi d^2$

Shell area resisting the force at the ends = $\pi d \times t$

Longitudinal stress $\sigma_L = \frac{p \times \frac{1}{4}\pi d^2}{\pi d \times t} = \frac{pd}{4t}$

For the case of a storage tank, the longitudinal stress will be the hydrostatic pressure acting on the bottom plate of the structure.

VII B External loads

Storage tanks are generally classified as short tanks, with a with a height-to-diameter (h/d) ratio of lower than 0.5. Because tanks are extremely thin-walled structures, wind is usually a primary design constraint.

Wind

External pressure due to wind can be extremely damaging to a tank, because generally the surface area is large and develops large forces. A wind speed of 190 km/h corresponds to a wind load of 0.86kN/m^2 on the vertical projected area of the tank shell and 1.44kN/m^2 for the uplift pressure on the roof. For other wind speeds, the pressure can be multiplied by a factor $\left(\frac{V}{190}\right)^2$. Reference is made to [6]& [23].
The horizontal wind pressure on the shell is calculated with:

$$q_{w,h} = 0.86 \times \left(\frac{V}{190}\right)^2$$

where:

V	= the design wind speed	[km/h]

The vertical uplift pressure on the roof is calculated with:

$$q_{w,v} = 1.44 \times \left(\frac{V}{190}\right)^2$$

The wind load on the tank surface is calculated with:

$$F_{wind} = q_{w,v} \times A = q_{w,v} \times d \times H$$

where:

q_w	= wind load	[kN/m^2]
d	= diameter	[m]
H	= height	[m]

The API Standard 650 states that for the design velocity of wind, a basic wind speed should be used which depends on the location of the project area. For Stolthaven, which is located in Louisiana, this is ≈ 259 km/h. (see Figure 3-5 in chapter 3.2.2).
The corresponding horizontal wind pressure is:

$$q_{w,h} = 0.86 \times \left(\frac{259}{190}\right)^2 = 1.6 \, {}^{kN}\!/_{m^2}$$

The corresponding vertical uplift pressure is:

$$q_{w,v} = 1.44 \times \left(\frac{V}{190}\right)^2 = 2.7 \, {}^{kN}\!/_{m^2}$$

For the design of storage tanks with external pressure (vacuum) as normal operating condition, the corresponding wind pressure is higher.
The horizontal wind pressure on the shell is then calculated with:

$$q_{w,h} = 1.48 \times \left(\frac{V}{190}\right)^2$$

where:

V	= the design wind speed	[km/h]

The wind pressure is uniform over the theoretical buckling mode of the tank shell.

Flood-induced loads

Approximately 30% of Louisiana's land mass lies in the Special Flood Hazard Areas (SFHAs). The SFHA is the land area covered by the floodwaters of the base flood on National Flood Insurance Program (NFIP) maps. In this area the NFIP's floodplain management regulations must be enforced and the mandatory purchase of flood insurance applies. This is land which is subject to a 1 percent or greater chance of flooding in any given year (base flood). These areas are delineated on a community's FIRM (Flood Insurance Rate Maps) as A-zones or V-zones.

Coastal V-zone: extends from the offshore to the inland and is subject to high-velocity wave action from storms or tsunamis.

Coastal A-zone: lies landward of a V-zone or landward of an open coast without mapped V-zones. The main source of flooding is coastal storms, with a potential base flood wave height between 0.46m and 0.91m (1.5-3.0 feet).

A-zone: in these areas the potential source of flooding is runoff from rainfall, snowmelt or coastal storms where the potential flood wave height is between 0 and 0.91m (0-0.3 feet).

X-zone: the flood hazard is less severe here than in the SFHA.

The Stolt-Nielsen terminal is located near Braithwaite, in Plaquemines Parish, Louisiana. On the Effective Flood Insurance Rate Map, Braithwaite is categorized as an A-zone [25]. Also the RoyalHaskoningDHV study on waves shows a 1/100y wave height of 0.34m, which indicates that the area most likely lies in a A-zone.

During extreme events, such as Hurricane Katrina and Isaac, structures laying in floodplains within high flood-induced hazard areas can be subject to a series of loads including hydrostatic, hydrodynamic loads and wave loads. To be able to calculate the flood loads, first the design flood depth and the design flood velocity is needed. For the determination of the flood loads, water levels and wave heights with a return period of 100-years will be used. This is required by the ASCE (American Society of Civil Engineers). The 1/100y design water level E_{sw} is NAVD88+5.6m, and the 1/100y design wave height is 0.34m.

Design flood depth d_s

The local design stillwater flood depth is first needed to calculate the loads.

$d_s = E_{sw} - GS$

where:

d_s	= design flood (stillwater) depth	[m]
E_{sw}	= design stillwater flood elevation above datum	[m+NAVD88]
GS	= lowest eroded ground elevation adjacent to a structure	[m+NAVD88]

The 1/100 years design water level E_{sw} is NAVD88+5.6m. The average ground elevation GS is approximately NAVD88+1m. This gives a local design stillwater depth d_s of approximately 4.6m.

[25] http://maps.riskmap6.com/LA/Plaquemines/

<u>Design flood velocity *V*</u>
Lower bound:
$$V_L = \frac{d_s}{t}$$
Upper bound:
$$V_U = (gd_s)^{0.5}$$
where:

$V_L; V_U$ = design flood velocity [m/s]
d_s = design stillwater depth [m]
t = 1 sec
g = gravitational acceleration [9.81m/s^2]

For a flood site which is distant from the flood source (such as Zone A) the lower bound velocity is used.

$$V_L = 4.6\, {}^{m}/_{s}$$

The flood loads can be distinguished in:

a. Hydrostatic loads
b. Hydrodynamic loads
c. Wave loads; breaking wave load
d. Debris impact load

A more detailed description of the loads and how these can be calculated is given below.

Hydrostatic loads
The hydrostatic load is caused by the water depth. It includes:

- the pressure at any depth due to the hydrostatic pressure on the vertical element of the structure
- the buoyancy pressure on the horizontal element of the structure

The hydrostatic pressure against the vertical wall can be calculated with:
$$F_{h,static} = \frac{1}{2}\rho g h^2$$
Where:

ρ = density of the fluid [kg/m^3]
g = acceleration due to gravity [m/s^2]
h = water level upstream and downstream of the wall [m]

For the determination of the hydrostatic pressure on the tank, the tank will be considered as empty (h_{ds} = 0), although this will not be the case in practice, where a minimum liquid height is required. This situation presents the ultimate loading state.
$\rightarrow$ For $d_s = 4.6m$: $F_{h,static} \approx 46\, {}^{kN}/_{m^2}$

For the assessment of the vulnerability of horizontal structural elements or the overall stability of a building, buoyancy should also be taken into account as it applies a potentially unbalanced uplift force and affects the resistance of gravity-based structures against sliding and overturning. The following can be estimated per unit length:
The buoyancy force ($F_{v,static}$) acting on the structure is calculated with:

$F_{v,static} = V_{dis} \times \rho_w \times g$

where:

V_{dis} = The displaced volume of the tank = $1/4 \times \pi \times D^2 \times d_s$ [m^3]

D = Diameter of the tank [m]

d_s = design stillwater depth [m]

ρ_w = Water density [kg/m^3]

g = Gravitational acceleration [m/s^2]

Hydrodynamic loads

The hydrodynamic loads are the result of the flowing water against and around the structural element. They are most of the time lateral loads caused by the impact of the moving mass of water and the drag forces as the water flows around the structure. These hydrodynamic loads also include the effects of broken and non-breaking waves striking the structure. ASCE 7-10 states that for the determination of the hydrodynamic loads "a detailed analysis utilizing basic concepts of fluid mechanics" is to be used. ASCE 7-10 does not provide any equations for this. According to the Coastal Construction Manual the hydrodynamic load can be calculated with:

Drag force:

$$F_{dyn} = \frac{1}{2} C_d \rho V^2 A$$

where:

F_{dyn} = horizontal drag force acting at the stillwater mid-depth (half way between the stillwater elevation and the eroded ground surface) [kN]

C_d = drag coefficient: recommended values are 1.2 for round piles

V = velocity of water [m/s]

A = surface area of obstruction (tank) normal to the direction of flow A = width * min [d_s, height of tank] [m^2]

With the use of the Reynolds number (Re), the drag coefficient C_d can be read from Figure 11-19. The Reynolds number can be calculated with the following equation:

$$R_e = \frac{DV}{\upsilon}$$

Where:

D = the diameter of the cylindrical tank [m]

V = velocity of water [m/s]

υ = kinematic viscosity = 10^{-6} [m^2/s]

The existing storage tanks have large diameters between 15 and 35m, that is why large Re-numbers are expected in the order of 10^7 and 10^8.

Because the estimated velocity can vary between V_L and V_U, the Re-number for both velocities should be determined.

In the figure below, the C_D coefficient related to these high Re-numbers is around 1.

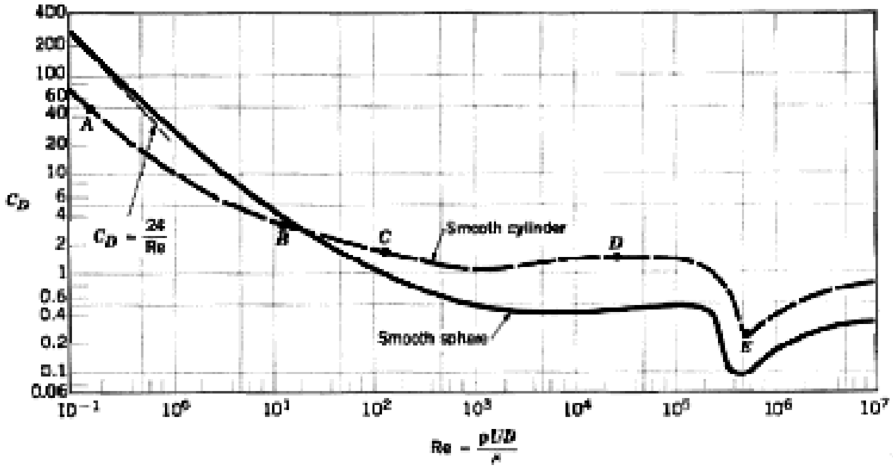


Figure 11-19 Relation between Re and CD [ref.: https://www.princeton.edu/~asmits/Bicycle_web/blunt.html]

Breaking wave loads

Since the storage tanks are large structures, they can be related to vertical walls instead of cylindrical piles or columns. Cylindrical piles or columns are slender structures where the diameter is very small related to its height. The storage tank on the other hand has a diameter to height ratio in the order of 0,5. Therefore the wave pressures will be determined on the basis of formulas for wave loading on vertical walls.

According to linear wave theory for non-breaking waves against a vertical wall, the wave height H in front of the wall is double the incoming wave height H_i in the case of total reflection. This causes a temporary water level rise. If this is considered a stationary load, the following rule of thumb can be applied to calculate the maximum wave pressure against the wall:

$$P_{max} = \frac{1}{2}\rho g H_i^2 + d\rho g H_i$$

Where:

P_{max}	= maximum wave pressure	[N/m]
ρ	= specific weight of water	[kg/m^3]
g	= gravitational acceleration	[m/s^2]
H_i	= the wave height of an incoming wave	[m]
d	= depth of the breakwater floating tank: d=draught of the tank	[m]

Impact load

The impact loads are loads resulting from any object transported by floodwater striking against buildings and other structures. These loads are determined using the rational approach as concentrated loads acting horizontally.

$$F_i = WVC_D C_B C_{Str}$$

where:

F_i	= impact force acting at the still water elevation [kN]
W	= weight of the object [kN] For W a weight of 450 kg (1000 pounds) is recommended in areas where the nature of potential debris in unknown. Objects with this weight could include parts of damaged buildings, poles, but also empty storage tanks.

V = velocity of water [m/s]
C_D = depth coefficient, for reduced debris velocity as water depth decreases; see Figure 11-20
C_B = blockage coefficient, for the reduction in debris velocity due to screening by trees, other structures etc.; see Figure 11-21
C_{Str} = Building structure coefficient
= 0.2 for timber pile and masonry column supported structures 3 stories or less in height above grade
= 0.4 for concrete pile or concrete or steel moment resisting frames 3 stories or less in height above grade
= 0.8 for reinforced concrete foundation walls (including insulation forms)

Flood Hazard Zone and Water Depth	C_D
Floodway[a] or Zone V	1.0
Zone A, stillwater flood depth $\geq$ 5 ft	1.0
Zone A, stillwater flood depth = 4 ft	0.75
Zone A, stillwater flood depth = 2.5 ft	0.375
Zone A, stillwater flood depth $\leq$ 1 ft	0.00

Figure 11-20 Depth coefficient C_D by Flood Hazard Zone and water depth

Degree of Screening or Sheltering within 100 Ft Upstream	C_B
No upstream screening, flow path wider than 30 ft	1.0
Limited upstream screening, flow path 20-ft wide	0.6
Moderate upstream screening, flow path 10-ft wide	0.2
Dense upstream screening, flow path less than 5-ft wide	0.0

Figure 11-21 Values of blockage coefficient C_B

VII C Load combinations

According to ASCE 7-10 (American Society of Civil Engineers) there are two methods for combining loads:

- Combining factored loads using strength design
  The load combinations and load factors given here shall be used only in those cases in which they are specifically authorized by the applicable material design standard
- Combining nominal loads using allowable stress design
  Loads listed herein shall be considered to act in the following combinations; whichever produces the most favorable effect in the building, foundation or structural member being considered

The load combinations given are in accordance with the allowable stress design (ASD method) (The coastal construction manual uses these combinations). These combinations are considered to act in the following combinations for buildings in Zone V and Coastal A Zone (Section 2.4.1 of ASCE 7-10), whichever produces the most unfavorable effect on the building or building element:

Combination No. 1: D
Combination No. 2: $D + L$
Combination No. 3: $D + (Lr \text{ or } S \text{ or } R)$

Combination No. 4: $D + 0.75L + 0.75(Lr \text{ or } S \text{ or } R)$
Combination No. 5: $D + (0.6W \text{ or } 0.7E)$
Combination No. 6a: $D + 0.75L + 0.75(0.6W) + 0.75(Lr \text{ or } S \text{ or } R)$
Combination No. 6b: $D + 0.75L + 0.75(0.7E) + 0.75S$
Combination No. 7: $0.6D + 0.6W$
Combination No. 8: $0.6D + 0.7E$

The following symbols are used in the definitions of the load combinations:

D = dead load
L = live load
E = earthquake load
F = load due to fluids with well-defined pressures and maximum heights (e.g., fluid load in tank)
F_a = flood load (hydrostatic loads and all components of the flood loads)
H = loads due to weight and lateral pressures of soil and water in soil
L_r = roof live load
S = snow load
R = rain load
W = wind load

A structure which is located in the flood zone, V-Zones or Coastal A-Zones, 1.5 F_a shall be added to the combinations 5, 6 and 7 and E shall be set to zero in 5 and 6.

In the calculations, the load combination which has the most unfavorable effect will be considered and will be elaborated.

Appendix VIII Stress distribution in a cross-section of a common composite slab

General info

A composite slab consists of a profiled thin steel plate which constructively co-operates with the concrete that is casted on top of it. [35] The slab consists of:

- a concrete slab
- a steel profile
- connectors (mainly dowels)

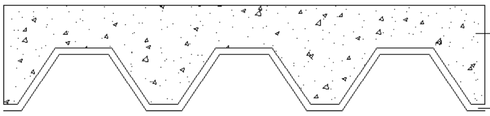
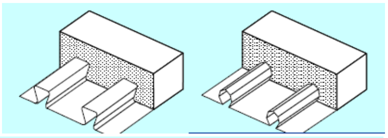


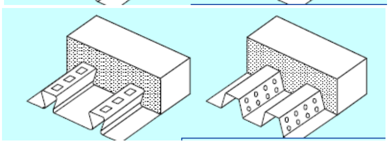
Figure 11-22 **Example composite slab**

The steel-concrete connection is realized by one or a combination of the following options:

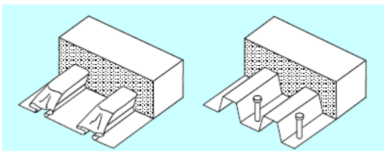
- frictional interlock



- mechanical interlock



- end anchorage



Dowels, also called shear studs, are the mainly used connectors for the composite structure.

Stress-distribution in a cross-section

In the case of positive bending and where the neutral axis is positioned above the steel profile, the moment capacity of a cross-section of a composite slab is given below.

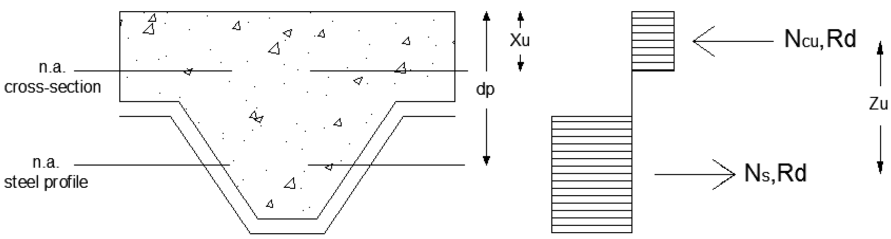


Figure 11-23 **Stress distribution in a cross section**

The nominal compressive force in concrete $N_{cu,Rd}$ is given by the formula below, where x_u is the depth of the concrete compressive zone:

$$N_{cu,Rd} = \frac{0.85 \cdot f_c}{\gamma_c} \cdot b \cdot x_u$$

where:

f_c	= cylinder strength of concrete	[N/mm^2]
γ_c	= partial material safety factor for concrete	[-]
b	= width of the section (per meter width)	[mm]
x_u	= depth to the concrete compressive zone	[mm]

The allowable tensile force in the steel plate is given by:

$$N_{S,Rd} = A_p \cdot \frac{f_y}{\gamma_a}$$

where:

A_p	= effective cross section of the steel plate	[mm]
f_y	= yield strength of steel	[N/mm^2]
γ_a	= partial material safety factor for steel plates	[-]

The depth of the compressive zone can be obtained by considering the equilibrium of forces:

$$N_{S,Rd} = N_{cu,Rd}$$

$$\rightarrow \quad x_u = \frac{A_p \cdot f_y \cdot \gamma_c}{0.85 \cdot b \cdot f_c \cdot \gamma_a}$$

The arm of internal leverage is determined with:

$$\rightarrow \; z_u = d_p - \frac{1}{2} x_u$$

where:

d_p	= distance between the top of the concrete and the neutral axis of the steel profile	[mm]
z_u	= arm of internal leverage	[mm]

Taking the moment about the center of the depth of the compressive zone gives the plastic moment resistance of the section:

$$M_{pl,Rd} = A_p \cdot \frac{f_y}{\gamma_a} \cdot z_u$$

Shear stud connectors

The strength of shear studs is equal to the lowest value of:

1. Shear of the studs:

$$P_{Rd} = \frac{0.8 \cdot f_u \cdot \frac{1}{4}\pi d^2}{\gamma_v}$$

2. Crushing of the concrete before the stud:

$$P_{Rd} = \frac{0.29 \cdot \propto \cdot\, d^2 \sqrt{f_{ck} \cdot E_{cm}}}{\gamma_v}$$

where:

f_u	= tensile strength of the dowel material. Most commonly used: = 450 N/mm^2	
d	= diameter of the dowel shaft	[mm]
$\propto$	= factor related to the influence of the length of the dowel	[-]

$\propto = 1$ for $h_{sc} \geq 4d$

$\propto = 0.2\left(\frac{h_{sc}}{d} + 1\right)$ for $3d \leq h_{sc} < 4d$

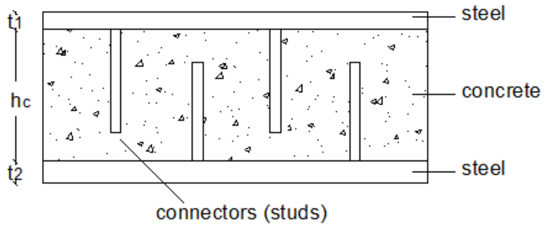
h_{sc} = length of the dowel [mm]

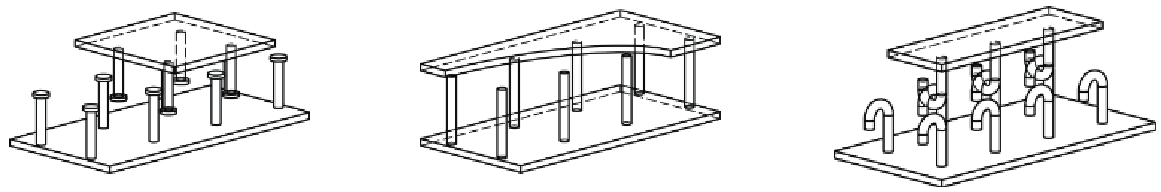
Number of studs can be determined with the acting shear force and the strength of the shear studs:

$$n = \frac{N_{S,Rd}}{P_{Rd}}$$

Appendix IX SCS sandwich slab

General info
Sandwich structure comprises three major structural parts: face plates, sandwich core, and mechanisms to transfer shear between face plates and core.





(1) Double-skin sandwich construction (DSC)

(2) Bi-Steel sandwich construction (Bi-Steel)

(3) Alternative sandwich construction

Figure 11-24 **Construction layouts of the SCS sandwich structure**

If a structure is potentially subject to significant bending moment, cyclic loading and large impact loading arising from hazardous environment, the SCS sandwich system serves as an appealing alternative to existing stiffened steel plate structures. The advantages of SCS include, but not limited to, the following: 1) economical and optimized design to achieve highs stiffness and strength; 2) improved impact resistance, especially leakage control after punching failure of the steel plates; 3) compared with stiffened plate, the exposed steel surface area is less and hence the amount of the protection coating can be reduced; 4) require less stiffeners and therefore less welding which eventually leads to improved fatigue performance; 5) concrete core provides good acoustic and thermal insulation; and 6) prefabrication and modular construction reduce construction time.

Stress distribution in the steel-concrete-steel sandwich slab (SCS sandwich slab)
The stress distribution in a cross-section of the slab is comparable with an ordinary composite structure. The steel plates take up the tension stress, where the concrete core takes up the compressive stress.

The plastic moment resistance of a SCS sandwich section subjected to bending can be determined by assuming a fully plastic rectangular stress block in the concrete. The concrete below the neutral axis is assumed to be cracked and does not contribute to the strength of the section.

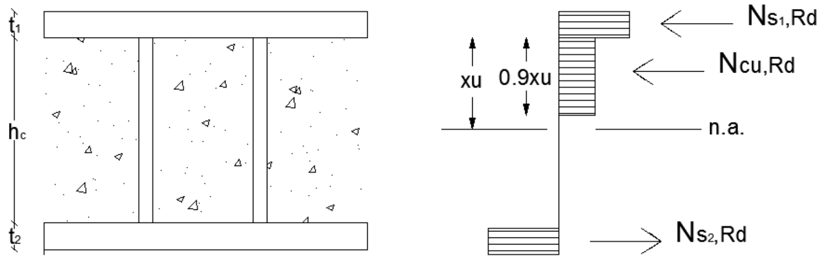


Figure 11-25 **Force distribution in the section at fully plastic stage**

The concrete is assumed to be in compression, which results in a resisting force acting in the center of an equivalent rectangular stress block with a depth of 0.9 times the depth to the neutral axis. The nominal compressive force in concrete is given by:

$$N_{cu} = \frac{0.85 \cdot f_c}{\gamma_c} \cdot b \cdot 0.9x_u$$

The forces in the steel plates depend on the yield strength and shear strength of material used for the connectors in resisting interfacial shear stresses in between the steel plate and the concrete core. It is also assumed that sufficient shear connectors are provided to prevent local bucking of the steel plate in compression. (Ref: Analysis and design of steel-concrete composite sandwich systems subjected to extreme loads &)

$$N_{S1,Rd} = b \cdot t_1 \cdot \frac{f_y}{\gamma_a} \ ; \ N_{S2,Rd} = b \cdot t_2 \cdot \frac{f_y}{\gamma_a}$$

The depth of the neutral axis can be obtained by considering the equilibrium of forces:

$$N_{S1,Rd} + N_{cu,Rd} = N_{s2,Rd}$$

The neutral axis can therefore be determined with:

$$x_u = \frac{1.176 \cdot \gamma_c \cdot f_y \cdot (t_2 - t_1)}{0.9 \cdot \gamma_a \cdot f_c}$$

Taking the moment about the centre of the compression steel plate gives the plastic moment resistance of the sandwich section:

$$M_{pl,Rd} = b \cdot t_2 \cdot \frac{f_y}{\gamma_a} \cdot \left(h_c + \frac{t_1}{2} + \frac{t_2}{2}\right) - \frac{0.85 \cdot f_c \cdot b \cdot 0.9x_u}{\gamma_c} \cdot \left(0.5(0.9x_u) + \frac{t_1}{2}\right)$$

where:

f_c	= cylinder strength of concrete	[N/mm^2]
γ_c	= partial material safety factor for concrete	[-]
b	= width of the section (per meter width)	[mm]
x_u	= depth to the neutral axis	[mm]
$t_1; t_2$	= thickness of steel plate	[mm]
f_y	= yield strength of steel	[N/mm^2]
γ_a	= partial material safety factor for steel plates	[-]

If the steel plates have similar thickness and strength, the SCS sandwich beam can be considered as an under reinforced beam

The SCS sandwich beam will deflect extensively and develop extensive and wide cracks in the final loading [8,9]. After yielding of tension steel plate, the cracking of the concrete will continue to rise towards the compression steel plate. In this case, the strain at the bottom plate is very large compared to top steel plate

Shear stud connectors
For the strength of the shear connectors is referred to Appendix VIII.
This section provides some design criteria for the application of shear stud connectors to composite elements. [36] & [37]

Note: These criteria are developed from scale model tests of single line elements, which may not fully represent the behavior of a slab or wall construction.

These design criteria are applied of shear stud connectors:

- Buckling in the compression steel plate depends on the space between the shear connectors. For a plate fixed at both ends the following applies:

  $$\frac{s}{t} = \frac{\pi^2 \cdot E}{3(1-\upsilon^2) \cdot \sigma_{cr}}$$

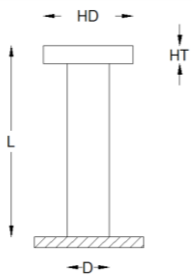
  where:

  s = the longitudinal spacing of shear connectors
  t = thickness of the steel plate
  E = Young's modulus of steel = 210000 N/mm^2
  σ_{cr} = critical buckling stress = σ_y (yield stress)
  υ = Poisson's ratio of steel = 0.3

  This leads to:

  $\frac{s}{t} \leq 52$ for S275 steel
  $\frac{s}{t} \leq 46$ for S355 steel

- Maximum connector spacing should satisfy the following:
  - s < 4L
  - s < 3*slab thickness
  - s < 0.6m



- Stud connectors subjected to pull-out forces should satisfy the following:
  - L ≤ 10*D
  - HD ≤ 1.5*D
  - HT ≤ 0.4*D
  - stud connectors welded on tension plates should stretch into the concrete compression zone or to the other steel plate
- If the steel plate is thin and the stud connector is large, it is possible that the stud will burn through the plate during the welding process. There is also the possibility that the plate is too flexible for the size of the stud. Therefore these criteria can be used
  - D ≤ 2.5*t (compression face)
  - D ≤ 2*t (tension face)

  This however also depends on the welding machine and the manufacturer's advice on minimum plate thickness for certain diameters.

Appendix X Stability of the structure

X A Vertical stability

For the vertical stability of the structure, the vertical effective soil pressure ($\sigma_{k,max}$) should not be larger than the maximum bearing capacity of the soil (p'_{max}) [38].

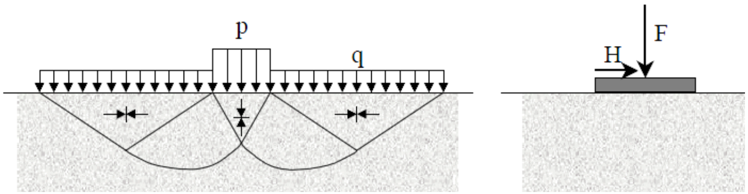


Figure 11-26 Sliding surfaces by Prandtl and Brinch Hansen

$\sigma_{k,max} < \frac{p'_{max}}{\gamma_m}$ where: γ_m = material factor

The maximum soil bearing pressure is determined with the Brinch Hansen method for shallow foundations. This is based on the sliding circles of the method of Prandtl. It is characterized by the soil conditions, between drained and undrained situations.
In this case, undrained soil will be analyzed. Here the cohesion c' is replaced by the undrained shear strength f_{und}. The internal friction is also set to $\emptyset'$= 0.

$$p'_{max} = c' \cdot N_c \cdot s_c \cdot i_c + q' \cdot N_q \cdot s_q \cdot i_q + 0.5 \cdot \gamma' \cdot B \cdot N_\gamma \cdot s_\gamma \cdot i_\gamma$$

With the bearing capacity coefficients, shape factors and inclination factors (for undrained soil):

$$N_c = (N_q - 1)\, cot\, \emptyset' \qquad s_c = 1 + 0.2\frac{B}{L} \qquad i_c = 0.5\left(1 + \sqrt{1 - \frac{H}{A \cdot f_{und}}}\right)$$

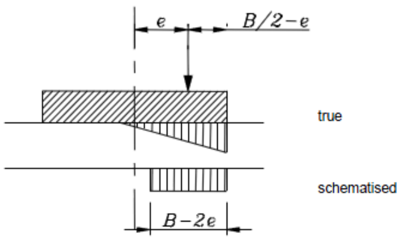
$$N_q = \frac{1 + \sin \emptyset'}{1 - \sin \emptyset'} \cdot e^{\pi \tan \emptyset'} \qquad s_q = 1 + \frac{B}{L}\sin \emptyset' \qquad i_q = \left(1 - \frac{0.7 \cdot H}{F + A \cdot f_{und} \cdot \cot \emptyset'}\right)^3$$

$$N_\gamma = 2(N_q - 1)\, tan\, \emptyset' \qquad s_\gamma = 1 - 0.3\frac{B}{L} \qquad i_\gamma = \left(1 - \frac{H}{F + A \cdot f_{und} \cdot \cot \emptyset'}\right)^3$$

where:

p'_{max}	= maximum soil bearing pressure	[kN/m^2]
c'	= cohesion	[-]
q'	= effective stress at the depth of but next to foundation surface	[kN/m^2]
γ'	= effective volumetric weight	[kN/m^3]
$\emptyset'$	= effective angle of friction	[°]
A	= the effective foundation area	[m^2]
f_{und}	= the design value of the undrained shear strength	[kPa]
N_c	= bearing capacity coefficient from cohesion	[-]
N_q	= bearing capacity coefficient from surcharge including soil coverage	[-]
N_γ	= bearing capacity coefficient of soil below the foundation	[-]
s_c	= shape factor from cohesion	[-]
s_q	= shape factor from surcharge including soil coverage	[-]
s_γ	= shape factor of soil below the foundation	[-]

i_c	= inclination factor from cohesion	[-]
i_q	= inclination factor surcharge including soil coverage	[-]
i_γ	= shape factor of soil below the foundation	[-]
H	= the shear force, i.e.: component of the force in the plane of the foundation surface	[kN]
F	= component of the exerted force perpendicular to the foundation surface	[kN]
L	= the length of the effective foundation area, for circular slabs: *L* = *B*	[m]
B	= the width of the effective foundation area, for circular slabs: *L* = *B*	[m]
	=width - e = width - $\frac{\sum M}{\sum V}$	



$\phi'_{e;d}$	N_c	N_q	N_γ
0°	5	1	0
5°	6.5	1.5	0
10°	8.5	2.5	1
15°	11	4	2
20°	15	6.5	4
22.5°	17.5	8	6
25°	20.5	10.5	9
27.5°	25	14	14
30°	30	18	20
32.5°	37	25	30
35°	46	33	46
37.5°	58	46	68
40°	75	64	106
42.5°	99	92	166

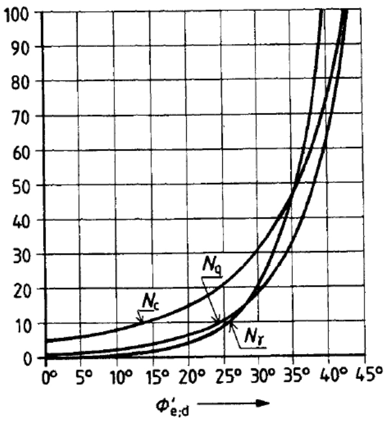
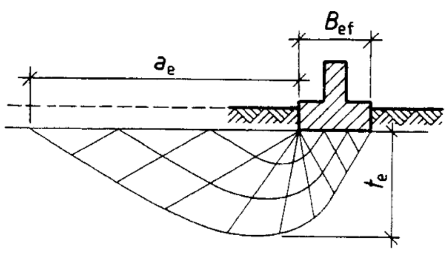


Figure 11-27 Bearing force factors as functions of the angle of internal friction



The depth and width of the sliding surface should be determined for an indication on to which soil layers the sliding circle reaches.

The vertical effective soil stress can be calculated with:

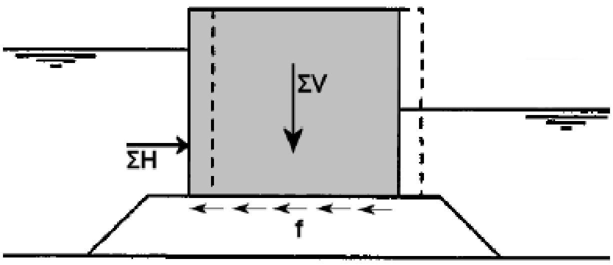
$$\sigma_{k,max} = \frac{F}{A} + \frac{M}{W} = \frac{\sum V}{{}^{1}/_{4} \cdot \pi \cdot d^2} + \frac{\sum M}{{}^{1}/_{32} \cdot \pi \cdot d^3}$$

where:

$\sum V$	= total of the acting vertical forces	[kN]
$\sum M$	= total of the acting moments, around point K, halfway the width	[kNm]
A	= area of the foundation	[m^2]
W	= section modulus of the contact area of the foundation	[m^3]
d	= diameter of the foundation	[m]

X B Horizontal stability

The horizontal forces acting on a structure are transferred to the base of the structure. The structure will slide aside if the friction force cannot withstand the total horizontal force (ΣH). This friction force is established by the multiplication of the total vertical loads acting on the structure (ΣH) with a dimensionless friction coefficient *f*. To prevent sliding of the structure, this friction force should be larger than the total horizontal loads acting on the structure.



The formula is:

$\Sigma H < f \cdot \Sigma V$

Some friction coefficients are given:

- ribbed bed = 0.6
- steel-concrete = 0.4

X C Rotational stability (overturning)

According to API Standard 650 the safety against overturning with regard to the wind load is checked on the basis of two criteria. Both criteria involve the working moments of the active loads on the structure.

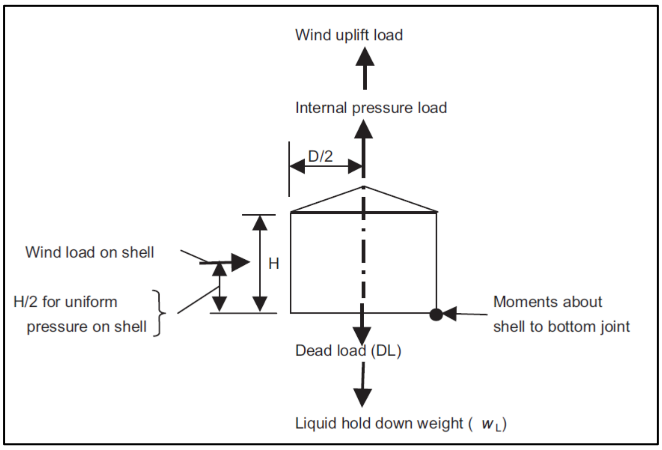


Figure 11-28 Loads working on tanks due to wind, internal pressure, dead load and liquid weight

Based on the figure above the following input is needed for the calculation of the stability against overturning:

1. Wind load
2. Weight of the tank and liquid weight
3. Load due to the internal pressure in the tank

For unanchored tanks the following 2 uplift criteria have to be satisfied:

1. $0.6M_w + M_{pi} < \frac{M_{DL}}{1.5}$

2. $M_w + 0.4M_{pi} < \frac{(M_{DL}+M_F)}{2}$

where:

M_{pi} = moment about the shell-to-bottom joint from design internal pressure (P),
M_w = overturning moment about the shell-to-bottom joint from horizontal (V_h) plus vertical wind pressure (V_v),
M_{DL} = moment about the shell-to-bottom joint from the weight of the shell and roof supported by the shell (D_L),
M_F = moment about the shell-to-bottom joint from the weight of the liquid (W_L)

Appendix XI The maximum bearing capacity of a laterally loaded pile

The maximum bearable horizontal load on a pile is determined according to the theory of Blum. With this theory the maximal absorbable load and related deformations of a pile can be calculated, assuming the following:

- A limit state in which the soil pressure is considered entirely passive
- The ground is homogenous, so it can be schematized as one layer
- The pile is assumed to be fixed against deflections at the theoretical penetration depth t_0
- The moment at t_0 is considered to be zero
- A lateral force at t_0 is allowed if the real length of the pile is taken to be 1.2 t_0

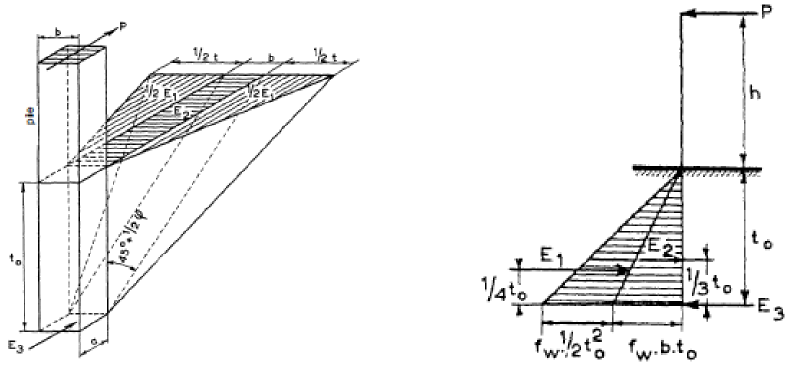


Figure 11-29 Schematization according to Blum, where the soil wedge causes a passive resistance [Left] and the ideal loading situation [right]

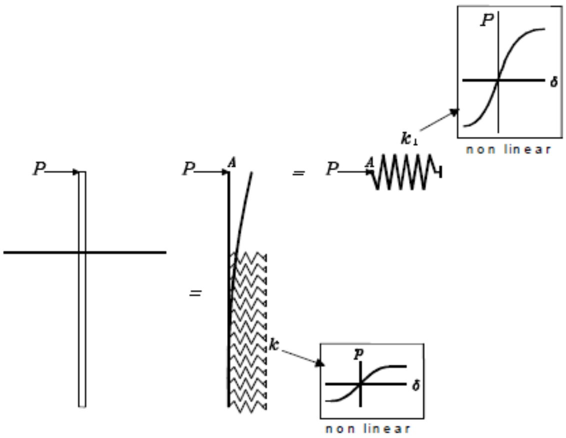


Figure 11-30 Schematization of a horizontally loaded pile

Strength

The maximum absorbable force P can be calculated from the balance of moments at depth t_0. The following equation is used:

$$P = \gamma' \cdot K_p \cdot \frac{t_0^3}{24} \cdot \frac{t_0 + 4d}{t_0 + h}$$

Where:

γ'	= effective volumetric weight of the soil (weight under water)	[kN/m3]
K_p	= passive soil pressure coefficient The passive soil pressure coefficient K_p can be calculated with the internal friction of the soil.	[-]

$$K_p = \frac{1 + \sin\phi'}{1 - \sin\phi'}$$

where: ϕ' = angle internal friction
For ϕ'=17.5°, K_p = 1.86

d	= diameter of the pile	[m]
t_0	= depth where the moment of the ideal load is zero = t/1.2	[m]
t	= practical embedded depth	[m]
h	= length of the unsupported part	[m]

Stiffness

A pile which is loaded perpendicularly to its axis will bend. The embedded part of the pile will have a displacement which results in a passive resisting force. The pile can be modelled as a cantilever beam with a horizontal concentrated load.

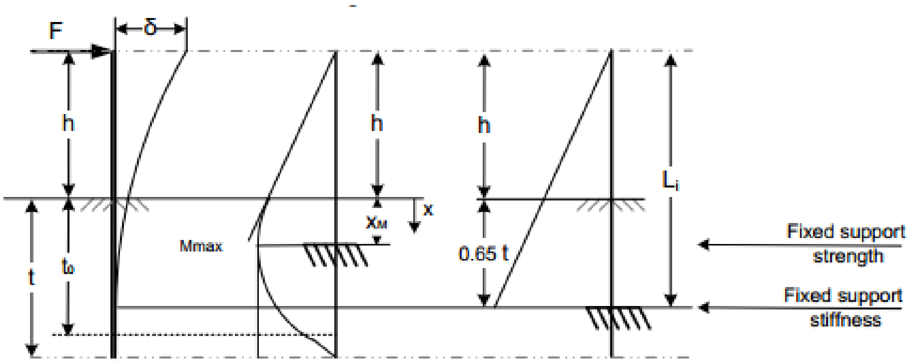


Figure 11-31 **Schematization of the elastic curve, Blum's bending moment diagram (strength) and moment diagram for a cantilever beam with load**

The maximum displacement can be written as:

$$\delta = \frac{F \cdot {L_i}^3}{3EI} \rightarrow \delta = \frac{F \cdot (h + 0.65 \cdot t)^3}{3EI}$$

where:

δ	= Displacement of the pile head	[m]
F	= Load	[N]
L_i	= *h + 0.65t*	[m]
E	= Young's modulus of the pile material	[N/m^2]
I	= Moment of inertia of the pile	[m^4]
h	= length of the unsupported part	[m]
t	= practical embedded depth	[m]

Strength: momentcapacity

The maximum working moment on the pile M_{Ed}should be smaller or equal to the maximum allowable moment $M_{pl,Rd}$:

$M_{Ed} \leq M_{pl,Rd}$

$$M_{Ed} = F \cdot (h + t_0)$$

where:

F	= The horizontal force working on the pile	[kN]
t_0	= depth where the moment of the ideal load is zero = t/1.2	[m]

h = length of the unsupported part [m]

$$M_{pl,Rd} = \sigma_y \cdot W$$

where:

σ_y = yield stress $[N/mm^2]$
W = Section modulus of the pile $[mm^3]$

Appendix XII Input data for calculations

Input data for example calculations in Appendix XIII

Stored liquid	
specific weight (γ_l) (the largest value is used in calculations)	0.85-1.6 g/ml
Soil (soft gray clay) (undrained)	
effective angle of friction ($\emptyset'$) volumetric weight (γ) effective volumetric weight (γ') undrained shear strength (f_{und}) ground elevation (average)	0° 17 kN/m^3 7 kN/m^3 50 kPa 1m
Water (flood)	
specific weight (γ_w) 1/100y flood height flood height on terminal (h_w) 1/100y wave height (H_i)	10 kN/m^3 5.6m 4.6 m 0.34m
Wind	
velocity (V)	259 km/hr
Concrete	
specific weight (ρ_c) concrete class cylinder strength of concrete (f_c) material factor (γ_c) E_{cm}	25 kN/m^3 C25/30 25 N/mm^2 1.5 31000 N/mm^2
Steel	
specific weight (ρ_s) steel class representative yield strength material factor (γ_a)	78.5 kN/m^3 S355 355 N/mm^2 1.1

Table 11-1 General input data

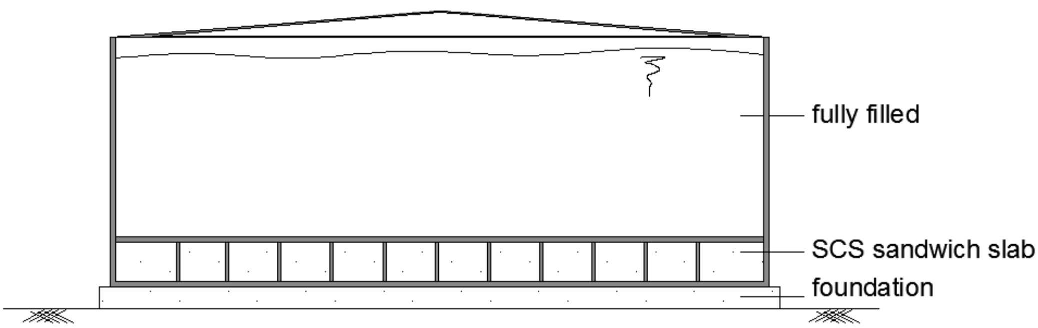
Appendix XIII Calculations for Tank A

This appendix presents stability calculations for Tank A.

Tank A	
diameter (d_t)	30 m
height (h_t)	14.08 m
area (A_t)	707 m^2
shell thickness (average) (t_s)	8 mm
bottom plate thickness (t_b)	10 mm
roof thickness (t_r)	5 mm
roof height (h_r)	0.9 m
Foundation	
diameter (d_f)	30+1=31m
area (A_f)	755 m^2
thickness (t_f)	0.3 m
section modulus (W)	2925 mm^3

Table 11-2 Input data of Tank A

XIII A Vertical stability Tank A



Method (see appendix X A):

- Soil bearing capacity (p'_{max}) with the Brinch Hansen formula
- Vertical effective soil stress ($\sigma_{k,max}$) with the generated stresses of the acting forces at the bottom of the foundation

The most unfavorable load combination is the case where the storage tank is fully filled, wind is acting on the structure and the terminal is not flooded. In this case the vertical pressure on the ground is the largest (buoyancy load, present if the terminal is flooded, is an upward load which will lower the vertical pressure on the ground).

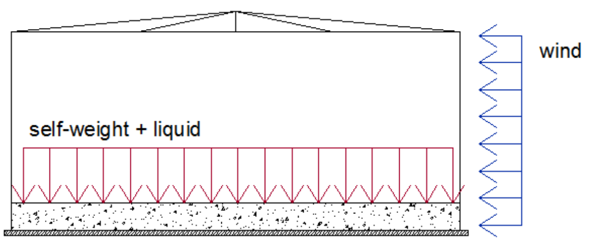


Figure 11-32 Load combination for the bearing capacity of soil

Input data for the vertical stability calculation is given in Table 11-1 and Table 11-2.

For the foundation a 1m larger diameter is chosen.
Boring stats from the current Stolt-Nielson location shows an overall of soft gray clay in the first soil layer in the area (see Appendix II). Off course a soil investigation should be done for the land of the new terminal. The foundation is a slab-on-grade, which means that there is no surcharge present and assuming undrained soil characteristics, the second and third part of the Brinch Hansen equation can be neglected.

The equation can be written as:
$p'_{max} = f_{und} \cdot N_c \cdot s_c \cdot i_c$, where: $c'$is replaced with f_{und}

Determination of the vertical effective soil stress $\sigma_{k,max}$	
No tank uplift: Buoyancy load = weight of structure + ballast weight	
Buoyancy load = $A_t \cdot h_w \cdot \gamma_w$	32517 kN
Concrete height needed ($h_{ballast}$) = $\frac{Buoyancy\ load}{A_t \cdot \gamma_c}$	2.76m
Total vertical load (∑V)	
Shell weight = $\pi d \cdot h_t \cdot t_s \cdot \gamma_s$	833 kN
Bottom plate weight = $A_t \cdot t_b \cdot \gamma_s$	555 kN
Roof weight = $\pi \cdot \frac{1}{2} d \cdot \sqrt{h^2 + r^2} \cdot t_r \cdot \gamma_s$	278 kN
weight of the structure:	1666 kN
Foundation weight = $A_f \cdot t_f \cdot \gamma_c$	5663 kN
Liquid weight (fully filled) = $A_t \cdot (h_t - h_{ballast}) \cdot \gamma_l$	127998 kN
Total vertical load (∑*V*) = **1.2*(weight of structure + ballast + liquid + foundation)** (1.2 = safety factor)	220255 kN
Total horizontal load (∑H)	
Wind load = 0.86 $kPa \cdot \left(\frac{V}{190}\right)^2 = 1.6 \frac{kN}{m^2}$	
Wind force = 0.6 · wind load · $d_t \cdot h_t$ (0.6 = safety factor)	405 kN
Total horizontal load (∑H)	405 kN
Total moment (∑M)	
Due to wind: wind foce · 0.5 · h_t	2854 kNm
Vertical effective soil stress $\sigma_{k,max}$	300 kN/m^2
Determination of the max. bearing capacity p'_{max}	
Bearing capacity factor	
N_c (Figure 11-27)	5
Shape factor	
$S_c = 1 + 0.2 \cdot 1$	1.2
Inclination factor	
$I_c = 0.5\left(1 + \sqrt{1 - \frac{405}{754.8 \cdot 50}}\right)$	1
Max. bearing capacity p'_{max} :	299 kN/m^2
Unity check for vertical stability	
Vertical stability: $\sigma_{k,max} < p'_{max}$	**Not OK**
The subsoil does not provide sufficient stability and preventive measures should be taken (see 8.1.1)	

Table 11-3 **Calculation of the vertical stability for Tank A**

XIII B Horizontal stability Tank A

The calculations are done with the use of Appendix X B. And the loads are determined with the equations given in Appendix VII B.

Wind load = $0.86\ kPa \cdot \left(\frac{V}{190}\right)^2 = 1.6\frac{kN}{m^2}$
Wind force (F_{wind}) = wind load $\cdot\, d_t \cdot h = 0.6\ \cdot 1.6 \cdot 30 \cdot (14.08 - 4.6) = 273kN$
[0.6 = safety factor]

Wave load: $P_{max} = \frac{1}{2}\rho g H_i^2 + h_d \rho g H_i$
$\rightarrow\ P_{max} = 1.5 \cdot \left(\frac{1}{2} \cdot 1000 \cdot 9.81 \cdot 0.34^2 + 4.6 \cdot 1000 \cdot 9.81 \cdot 0.34\right) = 23984\, {}^{N}\!/_{m} = 23.98\, {}^{kN}\!/_{m}$
[1.5 = safety factor]
Wave force (F_{wave}) = $P_{max} \cdot d_t = 23.98 \cdot 30 = 720\text{kN}$

Hydrodynamic load: $F_{dyn} = \frac{1}{2} C_d \rho V^2 A$

$$\rightarrow\ F_{dyn} = 1.5 \cdot \left(\frac{1}{2} \cdot 1 \cdot 1000 \cdot 4.6^2 \cdot 30 \cdot 4.6 \Big/ 1000\right) = 2190kN$$

Total horizontal loads on the tank = $F_{wind} +\ F_{wave} +\ F_{dyn} = 3179kN$

The weight of the tank is low compared to the ballast weight of the concrete, that is why only this is taken in the vertical load.
The weight of the concrete is equal to the buoyancy load (Table 11-3) = 32517 kN
The friction coefficient between concrete and steel is 0.4

Unity Check:
$\Sigma H < f \cdot \Sigma V\ \ \rightarrow\ \ 3179\ < 0.4 \cdot 32517\ \ \rightarrow\ \ 3179\ <\ 13007\ \ \rightarrow \boldsymbol{OK}$

XIII C Rotational stability (overturning)

The calculations are done with the use of Appendix X C. And the loads are determined with the equations given in Appendix VII B.

For unanchored tanks the following 2 uplift criteria have to be satisfied:

1. $0.6M_w + M_{pi} < \frac{M_{DL}}{1.5}$
2. $M_w + 0.4M_{pi} < \frac{(M_{DL}+M_F)}{2}$

wind		Moment [kNm]
vertical	1.6 kN/m^2	4757.91
uplift	2.7 kN/m^2	28627.76
	M_w	33385.68
	M_{DL}	487755.00
	M_F	0
internal pressure 25psf = 1.2 kN/m2		
	M_{pi}	12723.45

0,6M_w+M_{pi}	**M_{DL}/1,5**	**Ratio**	**safety against overturning**
kNm	kNm		If Ratio>1 ; not safe
32755	325170	0.1	safe

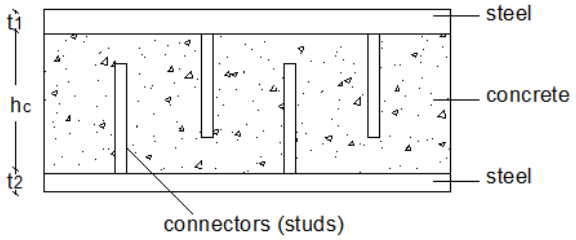
M_w+0,4M_{pi}	**M_{DL}+M_F/2**	**Ratio**	**safety against overturning**
kNm	kNm		If Ratio>1 ; not safe
38475	243878	0.2	safe

Table 11-4 Results for the rotational stability

XIII D Determination of the slab thickness for Tank C according to the design value of the maximum occurring bending moment

These calculations are done with the formulas given in Appendix IX.

For the floating tank, a design calculation should be done for the bottom plate of the structure. The SCS slab is also used here.
Most unfavorable load combination: Dead weight + buoyancy load, flood is present.



Assumptions:

- The use of the SCS sandwich slab for the transfer of the bending moment in the bottom plate.
- A steel plate thickness of 8mm (t_1) and 6mm (t_2) are chosen. The slab tends to have tension on top because of the buoyancy force, so the steel plate on top has a thicker value.
- Concrete height (h_c) of 60mm.

Method:

- Calculation of the design value of the maximum occurring bending moment
- Calculation of the allowable moment with the plastic moment capacity of the SCS sandwich slab
- Unity check

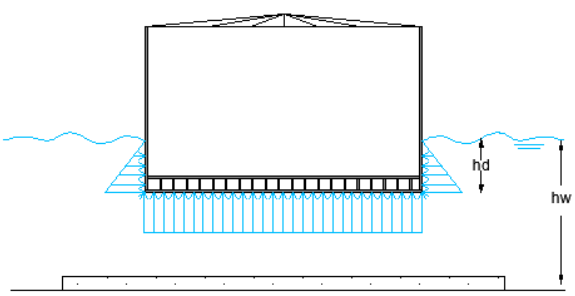


Figure 11-33 Floating tank under hydrostatic pressure and buoyancy load

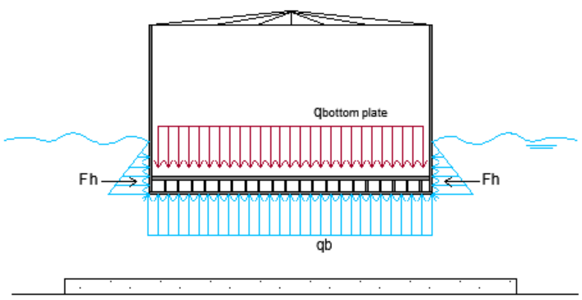
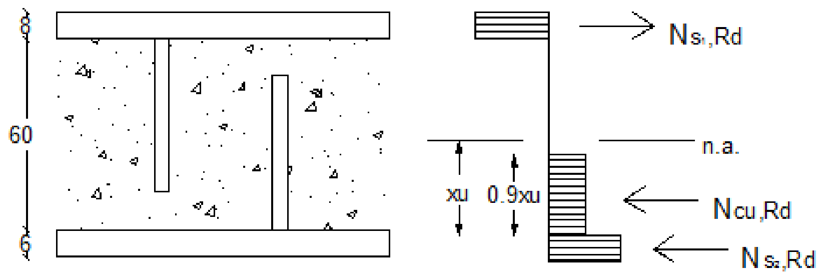


Figure 11-34 Working forces on the bottom plate of the tank

<u>Calculation of the maximum allowable moment:</u>



The tensile force in the steel plates:

$$N_{S1,Rd} = 1000 \cdot 8 \cdot \frac{355}{1.1} = 2582 \cdot 10^3 N = 2582 kN$$

$$N_{S12,Rd} = 1000 \cdot 6 \cdot \frac{355}{1.1} = 1936 \cdot 10^3 N = 1936 kN$$

The depth of the neutral axis can be obtained by considering the equilibrium of forces:

$$N_{S1,Rd} + N_{cu,Rd} = N_{s2,Rd}$$

$$\rightarrow\ x_u = \frac{1.176 \cdot 1.5 \cdot 355 \cdot (8-6)}{0.9 \cdot 1.1 \cdot 25} = 50.6 mm$$

Taking the moment about the center of the compression steel plate gives the plastic moment resistance of the sandwich section:

$$M_{pl,Rd} = 1000 \cdot 8 \cdot \frac{355}{1.1} \cdot \left(60 + \frac{6}{2} + \frac{8}{2}\right) - \frac{0.85 \cdot 25 \cdot 1000 \cdot 0.9 \cdot 50.6}{1.5} \cdot \left(0.45 \cdot 50.6 + \frac{6}{2}\right)$$
$$= 156.4 \cdot 10^6 Nmm = 156.4 kNm$$

Calculation of the design value of the maximum occurring moment:

Load of the SCS sandwich bottom plate:

$$q_{bottom\ plate} = 0.06 \cdot 25 + (0.006 + 0.008) \cdot 78.5 = 2.6\ {}^{kN}/_{m^2}$$

Weight of the structure:

Shell = $\pi \cdot 30 \cdot 11.3 \cdot 0.008 \cdot 78.5$	=	669	kN	
Bottom plate (SCS) = $706.9 \cdot 2.6$	=	1838	kN	
Roof = $\pi \cdot \frac{1}{2} 30 \cdot \sqrt{0.9^2 + 15^2} \cdot 0.005 \cdot 78.5$	=	278	kN	+
Weight of the structure	=	2785	kN	

Draught of the floating tank:

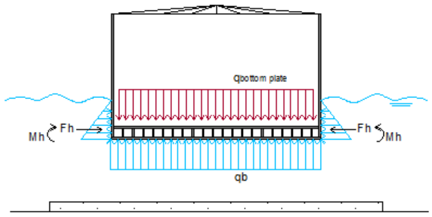
$$h_d = \frac{2785}{706.9 \cdot 10.25} = 0.38\ m$$

Buoyancy load:

$q_b = 1.5 \cdot 0.38 \cdot 10.25 = 5.85\ {}^{kN}/_{m^2}$ [1.5 = safety factor]

Hydrostatic load against the bottom plate causes a moment at the end of the plates, because the force is active at 2/3 of the draught:

$$F_h = \frac{1}{2} \cdot 0.38 \cdot 5.85 = 1.11 kN$$



$M_h = 1.11 \cdot \frac{0.38}{3} = 0.14kNm$

Design value of the total maximum occurring moment:

$$M_{Ed} = \frac{3 + 0.3}{16} \cdot (5.85 - 2.6) \cdot 15^2 = 137.1kNm$$

Design value of the maximum occurring moment at mid-span:

$M_{Ed,mid} = M_{Ed} - M_h = 137kNm$

Unity check

$M_{Ed,mid} \leq M_{pl,Rd} \rightarrow 137 < 156.4 \rightarrow OK$

A concrete core height of 60mm is sufficient for the moment distribution.

XIII E Maximum bearable horizontal load on a pile

These calculations are done with the formulas given in Appendix XI.

The floating tank is guided by a number of columns (circular hollow sections) around the tank. With this calculation the number of columns can be determined by calculating the maximum bearable load one column can absorb. The draught (h_d) of the tank is calculated on page 11-155 and is equal to 0.38m.

The piles are embedded in the subsoil and have to take on horizontal forces. The maximum bearable horizontal load on the pile is determined according to the theory of Blum.
The maximum absorbable force P can be calculated from the balance of moments at depth t_0:

$$P = \gamma' \cdot K_p \cdot \frac{t_0^3}{24} \cdot \frac{t_0 + 4d}{t_0 + h}$$

Assumptions/notes:

- The height *h* at which the tank is connected to the piles depends on the floating height of the tank (in reference with the ground level). The tank is assumed to be connected at 0.3m above the bottom of the tank. In this way the connection is close to the stiffened bottom plate. As can be seen in the previous section, the thickness of the bottom plate is 74mm (taking the steel plates of 6 and 8mm into account). So if the tank is still on the ground the height *h* is equal to 0.3 + 0.3 = 0.6m. This can be seen in Figure 11-35.
- Assuming 10 circular hollow sections with a diameter of 800mm.
- Thickness > 1/80 diameter = 10mm. A thickness of 20mm is chosen.
- t_o is the depth where the moment of the ideal load is zero and is at first assumed to be 5m.

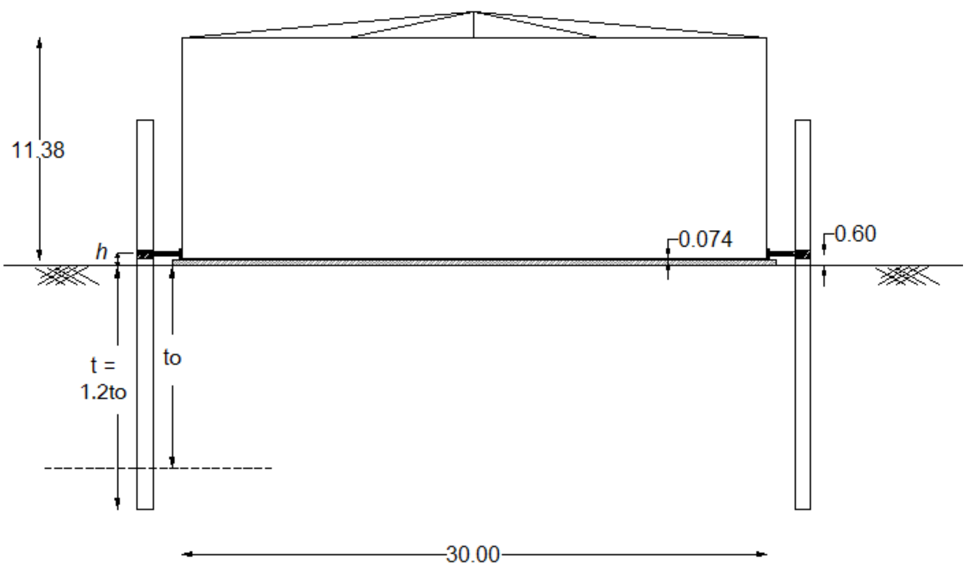


Figure 11-35 Floating tank with guiding columns

The piles are subjected to more than one load:

1. Loads on the tank, which are transferred to the column via the tank-column connection:
   - wind load
   - wave load
   - hydrodynamic load
2. Load on the pile: hydrodynamic load

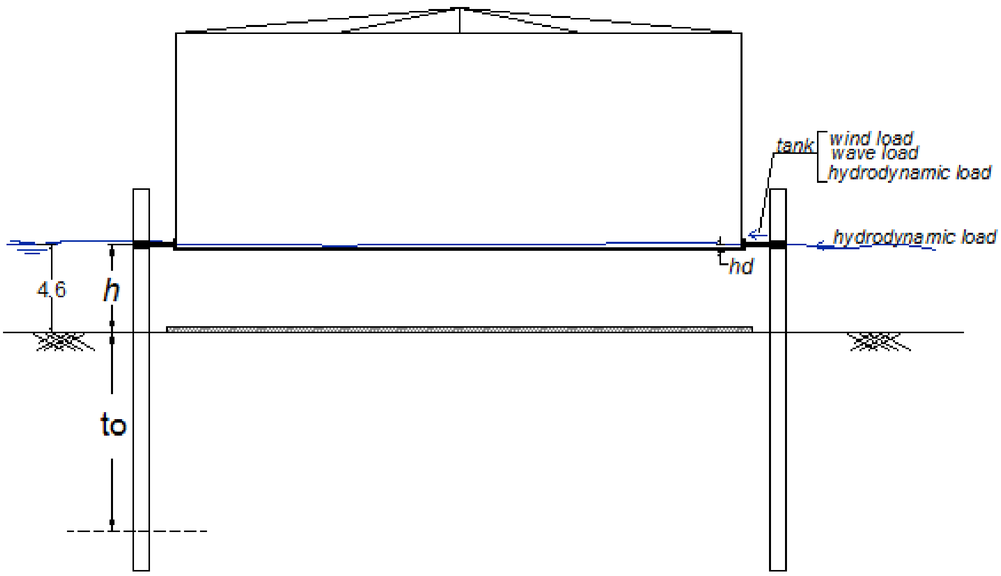


Figure 11-36 Loads working on the tank and the pile

The height of the connection *h* is now:
h = waterlevel - draught + connection height at the tank = 4.6 - 0.38 + 0.3 = 4.52m

Determination of the loads on the tank

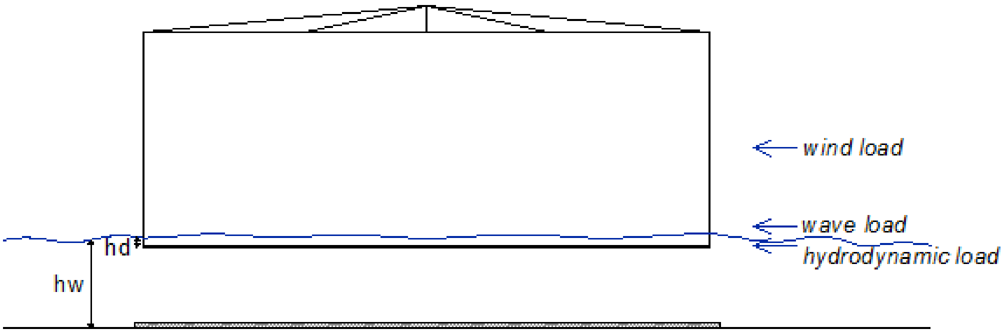


Figure 11-37 Loads working on the tank

Wind

Wind load = $0.86\ kPa \cdot \left(\frac{V}{190}\right)^2 = 1.6\frac{kN}{m^2}$
Wind force (F_{wind}) = $\text{wind load} \cdot d_t \cdot h = 1.6 \cdot 30 \cdot (11.3 - 0.38) = 524.2kN$

Waves

In this case the structure is not on the ground, but is floating. Therefore d_s will be substituted for *d* in the second part of the equation, because *d* causes the static load on the structure.

Wave load: $P_{max} = \frac{1}{2}\rho g H_i^2 + h_d \rho g H_i$
$\rightarrow\ \ P_{max} = \frac{1}{2} \cdot 10.25 \cdot 9.81 \cdot 0.34^2 + 0.38 \cdot 10.25 \cdot 9.81 \cdot 0.34 = 1880\,{}^{N}/_{m} = 1.9\,{}^{kN}/_{m}$
Wave force (F_{wave}) = $P_{max} \cdot d_t = 1.9 \cdot 30 = 57\text{kN}$

Hydrodynamic

Hydrodynamic load: $F_{dyn} = \frac{1}{2} C_d \rho V^2 A$

$$\rightarrow \quad F_{dyn} = \frac{1}{2} \cdot 1 \cdot 1025 \cdot 4.6^2 \cdot 30 \cdot 0.38 \Big/ 1000 = 123.6\ kN$$

Total loads on the tank = $F_{wind} + F_{wave} + F_{dyn} = 705kN$

10 piles have to transfer a load of 705kN to the ground.
Load on 1 pile: $\frac{705}{10} = 70.5kN$

Determination of the loads on the pile

Waves on the pile
The force (F_D) resulting from breaking wave acting on a rigid vertical pile or column is assumed to act at stillwater elevation and can be determined by[26]:

$$F_{d,pile} = 0.5 \gamma_w C_D D H_b^2$$

With:

C_D	= coefficient of drag for breaking waves, = 1.75 for round piles or columns and = 2.25 for square piles or columns	[-]
D	= pile or column diameter for circular sections, or for a square pile or column, 1.4 times the width of the pile or column	[m]
H_b	= Breaking wave height = 0.78d_s d_s = 4.6m (1/100y flood)	[m]

$$\rightarrow \quad F_{d,pile} = 0.5 \cdot 10.25 \cdot 1.75 \cdot 0.8 \cdot (0.78 \cdot 4.6)^2 = 92kN$$

Total load on 1 pile: $F_{tot,pile}$ = 1.5 * (70.5 + 92) = 244.5kN [1.5 = safety factor]

The maximum absorbable force P can be calculated from the balance of moments at depth t_0. The height h is now: $h = (h_w - h_d) + 0.3 = 4.6 - 0.38 + 0.3 = 4.52m$ (see Figure 11-36)

$$F_{tot,pile} \cdot (t_0 + h) = \gamma' \cdot K_p \cdot \frac{t_0^3}{24} \cdot (t_0 + 4b)$$

$$\rightarrow \quad F_{tot,pile} \cdot (5 + 4.52) = \gamma' \cdot K_p \cdot \frac{t_0^3}{24} \cdot (t_0 + 4b)$$

$$\rightarrow \quad 244.5 \cdot 9.52 = 7 \cdot 1.86 \cdot \frac{5^3}{24} \cdot (5 + 4 \cdot 0.8)$$

$$\rightarrow \quad 2324.6 \neq 556.1$$

$$\rightarrow \quad \text{Not OK}$$

For the assumed diameter and embedded length, the moments are NOT in equilibrium.

For Tank A: diameter = 30m and height = 11.38m, an iterative calculation is done for the calculation of the balance of moments.
The table below presents results for the depth t_o which is required for the moment equilibrium belonging to the diameters 0.8m and 1.0m and the selected number of piles n of 8 and 4. With these

[26] ASCE 7-10

4 combinations, the thickness of the pile is also checked. This is done by checking the momentcapacity of the pile. Here it is assumed that the piles have a steel grade of S355 (the representative yield strength = 355N/mm^2). A unity check is done with the formulas given in Appendix XI on page 11-148.
The result of the thickness required to withstand the working moment on the pile is also given in the table below.

	d [m]	n	to [m]	Total length t+h [m]	Total length of all piles [m]	Acting moment per pile Med [kNm]	Required thickness [mm]
1	0.8	8	8.3	14.48	115.84	3469.09	43
2	0.8	4	9.4	15.8	63.2	5604.19	72
3	1	8	8.4	14.6	116.8	3943.18	30
4	1	4	9.5	15.92	63.68	6130.95	48

Table 11-5 Results of t_o for 4 different diameter/number of piles combinations

The assumed thickness of 20mm is for non of the above combinations sufficient to withstand the working moment. The results presented in row 4 seem the most favorable when used in practice, because the total length of the piles in row 2 and 4 is less than the other 2 iterations and from these 2 options the required thickness in row 4 is far less than the one in row 2. For this reason the diameter/number of piles combination of row 4 is chosen.

The following pile dimensions are chosen:
- Diameter: d = 1m (1000mm)
- Theoretical embedded depth: t_o = 9.5m
- Number of piles: n = 4
- Thickness of the hollow section: d = 48mm

This results in:
- The embedded depth: t = 1.2* t_o = 11.4m
- Length of the pile (from the bottom of the pile to the connection point of the tank): t + h = 11.3 + (4.6-0.38+0.3) = 15.92m
- Total length of the pile (+ extra safety height of 0.5m): 15.92 + 0.5 = 16.42m

<u>Deflection</u>
The deflection for the piles with the above given dimensions can be determined.
The total load acting on the pile $F_{tot,pile}$ for the given dimensions and number of piles equals 402.6kN. With this force and the total length of a pile the deflection is calculated.
The deflection equals:

$$\delta = \frac{F \cdot (h + 0.65 \cdot t)^3}{3EI} = \frac{437.3 \cdot 10^3 \cdot (4.52 \cdot 10^3 + 0.65 \cdot 11.4 \cdot 10^3)^3}{3 \cdot 210000 \cdot \frac{1}{64} \cdot \pi \cdot (1000^4 - 952^4)} = 134.4mm = 0.13m$$

Appendix XIV External pressure

The effects of the external pressure are studied by determining the critical buckling pressure by 2 different methods.

XIV A Design criteria in API Standard 650

API Standard 650 gives minimum requirements for tanks which need to operate with external pressure [23]. It is intended for tanks subject to uniform external pressure, which does not exceed 6.9kN/m^2.
Due to the varying thicknesses the shell height is here converted into a transformed shell height H_{TS}.
The transformed shell has a height equal to H_{TS} and a uniform thickness equal to the topmost shell thickness (1st course).
The design external pressure for an unstiffened shell is calculated with:

$$P_s \text{ or } P_e \leq \frac{E}{15203\psi\left(\frac{H_{TS}}{D}\right)\left(\frac{D}{t_{smin}}\right)^{2.5}}$$

with: $H_{TS} = h_1\left(\frac{t_{s1}}{t_{s1}}\right)^{2.5} + h_2\left(\frac{t_{s1}}{t_{s2}}\right)^{2.5} + .. h_n\left(\frac{t_{s1}}{t_{sn}}\right)^{2.5}$

where:

t_{smin}	= the nominal thickness of the thinnest shell course	[mm]
H_{TS}	= the Transformed height of tank shell	[m]
$h_1, h_{2...}h_n$	= height of shell courses 1, 2, ...n, where the subscript numbering is from top to bottom	[m]
$t_{s1}, t_{s2...}t_{sn}$	= the nominal thickness of cylindrical shell course 1, 2...n,	[mm]
P_s	= the total design external pressure for design of shell	[kN/m^2]
	= the greater of 1. The specified external pressure, P_e, excluding wind or 2. W+0.4P_e	
W	= the maximum wind pressure	
	= 1.48 (V/190)^2	[kN/m^2]
V	= the specified design wind velocity	[km/h]
P_e	= the specified external pressure	[kN/m^2]
D	= the nominal tank diameter	[m]
E	= the modulus of elasticity	[N/mm^2]
ψ	= the stability factor	
	= 1 for W +P_e ($P_e \leq$ 0.25kN/m^2)	
	= [P_e+0.7]/0.95 for W+P_e (0.25kN/m$^2 \leq P_e \leq$ 0.7 kN/m^2)	
	= [P_e/0.48] for W +P_e	
	= 3 for specified external pressure only	[-]

The wind pressure here is uniform over the theoretical buckling height of the tank shell.

For a specific design external pressure, the required thinnest shell course can be calculated:

$$t_{smin} \geq \frac{47.07(\psi H_{TS} P_s)^{0.4} D^{0.6}}{(E)^{0.4}}$$

To increase the resistance to buckling under external pressure loading, the tank shell can be strengthened with circumferential stiffeners.
The equation for t_{smin} can be rearranged to calculate the maximum spacing of intermediate stiffeners, also called “safe height”:

$$H_{Safe} = \frac{(t_{smin})^{2.5}(E)}{15203(D)^{1.5}(P_s)\psi}$$

The number of required intermediate stiffeners N_s is based on H_{Safe}. No intermediate stiffeners are required if the value for N_s is zero or negative.

$$N_s + 1 = \frac{H_{TS}}{H_{Safe}}$$

The maximum stiffener spacing L_x for each shell thickness shall be:

$$L_x = H_{Safe}\left[\frac{t_{sx}}{t_{smin}}\right]^{2.5}$$

where:

t_{sx} = the thickness of the shell in question [mm]

With the information given on page 8-77 and Figure 8-3, t_{smin} is calcuated for Tank A to see if the wall thicknesses are sufficient for the external wind pressure of 2.8kN/m^2.

Tank A		
Courses	t_i [mm]	h_n [mm]
1	6,5	3300
2	6,5	3300
3	6,7	3300
4	12	3300

D	30m
E	210.10^3 N/mm^2

Wind

If no flood is present, the stability factor $\psi = 1$.

H_{ts} [m]	ψ	Ps [kN/m^2]	t_{smin} [mm]	H_{safe} [m]	Nr. of stiffeners
10.1	1	2.8	10.2	3.2	2

Hydrostatic pressure

XIV B The weighted smeared method

This section gives a short description of the method. A full description of the method is given in the paper in 0 [28].

The weighted smeared method is a new method for the determination of the critical buckling resistance of cylindrical shells with varying thickness subjected to a uniform external pressure (Figure 8-10).

Here, the different thicknesses are converted to an equivalent thickness. By “smearing out” the different thicknesses, the equivalent thickness t_{eq} is related to the buckling mode by a certain weight according to its effect on the buckling resistance.

The equivalent thickness is determined with:

$$t_{eq}{}^3 = \left(\frac{1}{l}\right)\sum_{i=1}^{n}[t_i{}^3(H_1 - H_{i-1})],\ \ i = 1, 2, \dots, n$$

In which

$$H_i = \left(h_i - \frac{l}{2\pi}\sin\frac{2\pi h_i}{l}\right)$$

where:

l	= potential buckle height	[mm]
h_i	= the distance from the top to the base of a course	[mm]
t_i	= course thickness	[mm]

If the thickness changes *n*-times, the number of buckle heights will be *n+1*.

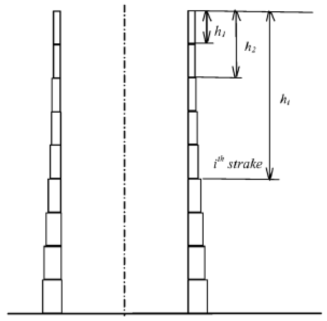


Figure 11-38 Definition of h_i, the distance from the top to the base of a course

The weighted smeared buckling pressure follows from the circumferential buckling pressure, which is based on the classical linear Donnell shell buckling theory. The weighted smeared buckling pressure is calculated with:

$$p_{cr,s} = 0.92E\left(\frac{r}{l}\right)\left(\frac{t_{eq}}{r}\right)^{2.5}$$

where:

E	= Young's Modulus of steel	[N/mm^2]
r	= radius of the tank	[mm]
l	= potential buckle height	[mm]
t_{eq}	= the equivalent thickness	[mm]

Note:
The article refers to EN 1993-1-6 (Eurocode 3 Part 1-6), where it states that the non-uniform wind pressure on the tank can be substituted by an equivalent uniform wind pressure [29]. *The equivalent uniform pressure q_{eq} is equal to the maximum wind pressure $q_{w,max}$ multiplied with a factor k_w (0.65 ≤ k_w ≤ 1). For this calculation it is assumed that $q_{eq} = q_{w,max}$.*

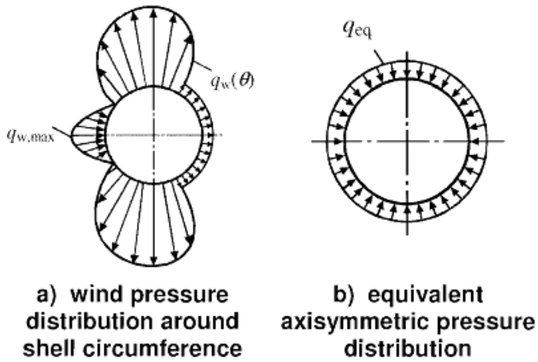


Figure 11-39 Transformation of typical wind external pressure load distribution **[29]**

<u>*Wind*</u>

With the information given on page 8-77 and Figure 8-3, a calculation check is done for Tank A to see if the wall thicknesses are sufficient for the external wind pressure.

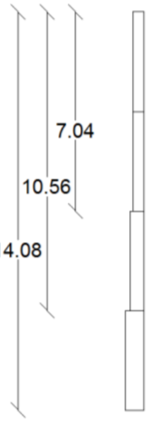
Tank A		
Courses	t_i [mm]	h_i [mm]
1	6,5	3520
2	6,5	7040
3	7	10560
4	12	14080

Table 11-6 **Thickness & height from the top to the base of each course**

Results

Each change of plate thickness
leads to a possible buckle height (in m.).
This is presented in the figure on the left.

The table below gives the results for the critical buckling pressures for each buckle height.



	t_i [mm]	**h_i [mm]**	**l [mm]**	**Contributing h_i [mm]**	**t_{eq}**	**$p_{cr,s}$ [kN/m^2]**
1	6.5	7040	7040	7040	6.5	1.61
2	6.5	7040	10560	7040	6.6	1.12
	7	10560	10560	10560		
3	6.5	7040	14080	7040	7.6	1.18
	7	10560	14080	10560		
	12	14080	14080	14080		

Table 11-7 **Results of the calculation of the critical buckling pressure $p_{cr,s}$**

<u>Description</u>
Table 11-7 shows the 3 possible buckle heights (l): 7040, 10560 and 14080mm. These are numbered from 1 to 3 in the table. Because the first 2 courses have the same thickness, they contribute as 1, which is the 1st buckle length of 7040mm. Each buckle height has its own contributing courses, for example: At number 3, where the buckle height is 14080mm, the contributing heights are the heights of all courses (h_1-h_4): 7040mm (the first 2 courses), 10560mm and 14080mm.
Each buckle length has their own contributing courses with their different heights. These heights are then transformed into an equivalent height for each buckle length. The weighted smeared critical pressure is then determined for each buckle height.

<u>Evaluation</u>
The critical buckling pressure for this tank is the lowest value of the 3 results given in Table 11-7, which is 1.12kN/m^2.

	$P_{cr,s}$ [kN/m²]	
	r = 15000mm	*r* = 7500mm
1	1.61	4.55
2	1.12	3.16
3	1.18	3.34

Table 11-8 Critical buckling pressures for r = 15000mm and r = 7500mm

Hydrostatic pressure

	t_i [mm]	h_i [mm]	l [mm]	Contributing h_i [mm]	t_{eq}	$p_{cr,s}$ [kN/m²]
3	12	3520	3520	3520	12	14.9
4	12	3520	7040	3520	13.1	9.2
	14	7040	7040	7040		

Table 11-9 Results of the calculation of the critical buckling pressure $p_{cr,s}$

The description of Table 11-9 is related to the description of Table 11-7,

Appendix XV **Paper:**

Practical calculations for uniform external pressure buckling in cylindrical shells with stepped walls

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Practical calculations for uniform external pressure buckling in cylindrical shells with stepped walls

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ABSTRACT

Metal cylindrical storage structures of significant size, such as silos and vertical-axis tanks, are almost always constructed from many short cylindrical shells of different thickness as the stress resultants on the wall progressively increase towards the base. The resulting increases in thickness are always made in step changes using metal sheets of uniform thickness because of the availability of such source materials. The result is a shell with a stepped wall with multiple discrete steps in thickness. Such shells are very susceptible to buckling under external pressure when empty or partially filled, but the buckling mode may involve only part of the shell height due to the changes in shell thickness. These changes must therefore be accounted for within the design process. A new method of determining the critical buckling resistance of such shells was recently developed, and although it has been shown to be valid, the methodology for its application in practical design has not been set out or shown. This paper therefore briefly describes the new method and demonstrates the manner in which it can be used to produce rapid, safe assessments of cylindrical shells with a wide range of patterns of wall thickness changes. The results are then suitable for direct introduction into such documents as the European standard on metal shells [1] and the ECCS Recommendations [2].

1. Introduction

Practical metal silos and tanks are almost always constructed from many short cylindrical shells of different thickness because the stress resultants on the wall progressively increase towards the base. In silos, these thickness changes are considerable because they relate to the progressively cumulative effect of wall friction between the solid and the containing shell, whilst in fluid filled tanks the variation relates only to the linear increase in internal pressure. In both cases, practical construction considerations mean that the increases in thickness are always made in step changes using metal sheets of uniform thickness because of the availability of such source materials. The result is a cylindrical shell with a stepped wall that has multiple discrete steps in thickness. Such shells are very susceptible to buckling under external pressure when empty or partially filled, but the buckling mode may involve only part of the shell height due to the changes in shell thickness. These changes must therefore be accounted for within the design process. Fig. 1 shows a typical tank structure taken from the European Recommendations on Shell Buckling [2].

A recently developed new method to evaluate the buckling strength of a cylinder with stepped walls has been described and verified elsewhere [3], but the application of the method itself and its results have not been set out.

In the European standard [1] and ECCS Recommendations [2], stepped wall cylinders under circumferential compression are transformed, first into a three-stage cylinder and thence into an equivalent uniform thickness cylinder based on previous research [4–6]. This two-stage process leads to a relatively complicated calculation that requires a chart that may be tricky to use, and where the mechanics of the buckling mode is somewhat hidden. The process is not very straightforward for application in either spreadsheets or simple computer programs that are intended to aid the practical design of silos and tanks.

This paper outlines the new "weighted smeared wall method", which has been verified [3] as an accurate and direct method of dealing with stepped-wall cylinders with any thickness variation. The method is based on the concept proposed by Trahair et al. [7] for corrugated stiffened walled cylinders under external pressure. In [3], buckling predictions were made for a wide range of geometries using the new direct design calculation method. These were compared with both accurate predictions from finite element calculations and the current Eurocode [1] rules. These extensive comparisons showed that the weighted smeared wall

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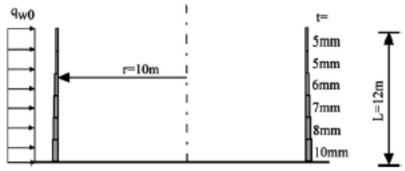


Fig. 1. Typical example of tank structure (taken from ECCS EDR5, 2008).

method is a very effective, accurate and informative method of capturing the buckling behaviour of stepped wall cylinders for a wide range of shells. However, the method of application of this method has not yet been set out, and its adoption into standards requires a clear demonstration of its practical use. The purpose of this paper is to set out the method of application of the new method and to illustrate its results.

2. Behaviour of cylinders of constant thickness under uniform external pressure

The uniform external pressure at which a cylindrical shell buckles is very sensitive to the geometry of the cylinder. The buckling strength drops rapidly as the buckle becomes longer, so longer cylinders have a lower critical buckling pressure. The buckling strength is also affected by the thickness of the cylinder and thinner cylinders have lower critical buckling pressures.

The theoretical linear bifurcation buckling pressure of a constant thickness cylinder under uniform external pressure can be obtained [7–10] by minimising

$$p_{cr} = \frac{Et^3}{12(1-\nu^2)r^3}\left(\frac{\pi r}{\ell}\right)^2\left(\alpha+\frac{1}{\alpha}\right)^2 + \frac{Et}{r}\left(\frac{\ell}{\pi r}\right)^2\frac{\alpha^4}{(\alpha+(1/\alpha))^2} \quad (1)$$

where $\alpha = \pi r/m\ell$, r is the radius of the shell, t is the thickness of the shell wall, E is the elastic modulus, ν is Poisson's ratio, m is the number of complete buckles around the circumference and ℓ is the half wave-height of the buckle, which is equal to the full height of a cylinder of uniform thickness. The buckling mode is assumed to be in the form of a half sine wave vertically, corresponding to the assumption that both ends of the buckle are pinned and free to displace axially during buckling.

Almost all silos and tanks fall into the category of short or medium-length cylinders. For external pressure buckling, a medium-length cylinder is defined in EN 1993-1-6 [8] as in the geometric range

$$20 < \frac{\omega}{C_\theta} < 1.63\frac{r}{t} \quad (2)$$

where ω is the dimensionless length parameter $\omega = \ell/\sqrt{rt}$ and C_θ is a buckling pressure factor to account for different boundary conditions. Short cylinders are all those whose length lies below the lower limit of the medium-length range. Whilst the complete cylindrical shell of a practical silo or tank is normally in the medium range, it may well become short where the buckling mode is confined to the upper courses.

For medium-length cylinders, minimising Eq. (1) with respect to m leads to the critical circumferential buckling wave number as [10]

$$m_{cr}^2 = \pi\left(\frac{r}{\ell}\right)\sqrt{\frac{r}{t}}\sqrt[4]{36(1-\nu^2)} \approx 7.5\left(\frac{r}{\ell}\right)\sqrt{\frac{r}{t}} \quad (3)$$

Table 1
External pressure factors for medium-length cylinders C_θ (EN 1993-1-6, 2007).

Case	Base boundary	Top boundary	External pressure factor C_θ
1	axially restrained	axially restrained	1.5
2	axially restrained	axially free	1.25
3	axially free	axially free	1.0
4	axially restrained	completely free	0.6

and the circumferential buckling pressure is given by the classical Eqs. (8)–(11)

$$p_{cr,D} = 0.92\left(\frac{E}{\omega}\right)\left(\frac{t}{r}\right)^2 \quad (4)$$

based on the classical linear Donnell shell buckling theory [12].

The above relationships are based on classical simply-supported boundary conditions S3 [11] at both ends of the buckle. Fortunately, different practical boundary conditions may be accounted for by modifying the external pressure buckling by the simple factors C_θ for medium-length cylinders [1]. These factors are shown in Table 1. The simply-supported reference condition is Case 3: the other practically important condition is Case 2, where the base boundary is restrained by a foundation or by lower parts of the cylinder when the buckle forms in only part of the shell. Thus, for practical structures, either the pinned end Case 2 or an enhancement of the buckling strength by exactly 25% may be expected.

3. Stepwise variable wall thickness cylinders buckling under uniform external pressure

Where a cylinder under uniform external pressure has a variable thickness that increases in a stepwise manner progressively from the top downwards, the top (thinnest) course is always involved in the buckling mode. The buckling strength may reduce as lower thicker courses are also involved because the increased buckle length can reduce its strength, but the contrasting effect is also present that the increased thickness of the lower courses may increase the buckling pressure. A simultaneous change of length and change of thickness produces two opposing trends, so it is not simple to determine how high the weakest buckle will be when the wall thickness varies in a stepwise manner. Any change in the wall thickness distribution can change the critical buckling mode and the corresponding buckling pressure.

These ideas were confirmed by the experimental study of Fakhim et al. [13] on buckling of stepwise variable wall thickness cylinders under hydrostatic external pressure. Their shorter shells buckled over the whole height, but in the longer shells, the buckling modes were confined to the top parts.

A fortunate aspect of the buckling mode is that the base of a buckle will always be either at or close to a change of plate thickness because the buckling pressure falls as the length increases. This is well illustrated in Fig. 2, taken from [10], which shows that the buckle which forms is essentially independent of the shell thicknesses in the region below the critical zone. The same buckle forms at the same external pressure whether the wall is slightly longer than the critical buckle, or very much longer, provided that the increases in thickness below that point are sufficient to counteract the reducing buckling pressure associated with the greater height of potential buckles.

One feature of the buckle forms shown in Fig. 2 is that a buckle whose lower boundary is at a change of plate thickness does not

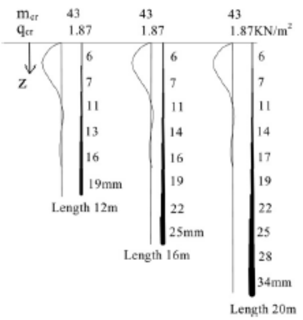


Fig. 2. Buckling modes in a 40 m diameter stepped wall tank (after Greiner, 2004 [10]).

respond to this condition as if it were pinned. Nor does the change correspond to a truly fixed boundary. Instead, the flexural stiffness of the course below the base of the buckle provides both a flexural and a translational elastic restraint to the buckle boundary, causing the complete buckle shape to penetrate into the lower course. This makes the buckle slightly longer than that corresponding to the point of plate thickness change (causing a reduction in buckling resistance), but it also includes flexural restraint from the lower course (causing an increase in buckling resistance). These two effects approximately cancel each other, so that a design model that assumes that the buckle terminates at the change of plate thickness is relatively accurate.

A further comment should also be made concerning the axial restraint at the base of this buckle. As noted above, the classical buckling pressure (Eqs. (1) and (4)) relates to pinned boundary conditions with freedom to displace axially during buckling. Where the buckle terminates within the shell wall at a change of plate thickness, the buckling strength is generally enhanced by an increase attributable to the elastic axial restraint from the lower courses. The axially free lower boundary (Case 3) is thus only strictly applicable to a buckle extending over the full height in an unanchored tank. Silos are always effectively anchored at the base.

4. The weighted smeared wall method for stepped walls

A corrugated stiffened shell wall may have both circumferentially and axially varying stiffnesses, and for these structures Trahair et al. [7] adopted a buckling mode with sinusoidal deformation in both directions. Within a potential buckle, they evaluated the corresponding "effective thickness" for bending and stretching separately in each direction. The equivalent thickness is related to the buckling mode by approximately "smearing out" the thickness steps, with the equivalent thickness weighted according to its effect on the buckling resistance according to the deformation mode. Thus the weighted smeared thickness is found by considering the contribution that each part of the wall makes to the strain energy of the buckling mode, and equating it to the energy of the destabilising pressures [9]. The process is equivalent to a single term Galerkin or a first Rayleigh–Ritz estimate. Following this equivalent thickness assessment, each potential buckling mode can be assessed using its corresponding effective thickness to find the lowest strength which gives the critical buckling mode. The weighted smeared wall method is based on this concept. Strictly speaking, the weighted thickness for stretching and that for bending should be different, but this leads to more complicated calculation procedures. However, it was found that the stretching strain energy for the buckling event is much smaller than the bending strain energy for the buckling event, which conforms to the physical characteristic for shell buckling, so the method presented here was trialled using both, then only stretching, and finally only bending in the evaluation. The final results using only the bending equivalent thickness produced a conservative and rather accurate assessment which was also the simplest, so it was adopted throughout as a single value.

As the thickness of the cylinder is constant around the circumference, the equivalent uniform thickness t_{eq} is found as a weighted average value of the thickness of each course

$$t_{eq}^3=\left(\frac{2}{\ell}\right)\int_0^{\ell} t_z^3\sin^2\left(\frac{\pi z}{\ell}\right)dz \tag{5}$$

Since the buckle extends over n courses of constant thickness, Eq. (5) reduces to the sum of the contributions from each constant thickness course and can be better expressed as

$$t_{eq}^3=\left(\frac{1}{\ell}\right)\sum_{i=1}^{n}[t_i^3(H_i-H_{i-1})], \qquad i=1,2,\cdots n \tag{6}$$

in which

$$H_i=\left(h_i-\frac{\ell}{2\pi}\sin\frac{2\pi h_i}{\ell}\right) \tag{7}$$

and h_i is the distance from the top of the cylinder to the bottom of the ith course (Fig. 3; for $i=1$, $h_0=0$ and $H_0=0$). Here, ℓ is the total height of the potential buckle being assessed, terminating at the base of one course.

As noted above, the chief difficulty in determining the buckling strength of a stepped wall cylinder is to find the critical buckling mode, which has the lowest buckling pressure. The following describes the required procedure. This is best set up in a spreadsheet, since all potential buckle heights also correspond to the heights of the individual courses.

The different heights of possible buckling modes are all examined in parallel. The half wavelength ℓ can extend over just the top thinnest courses only, or a longer part of the wall ending at a change of thickness, or over the whole height of the cylinder. Using Eqs. (6), (3) and (4), the equivalent thickness t_{eq}, the critical circumferential mode m_{cr} and the minimum buckling pressure p_{cr} for each assumed buckle height can be calculated, producing a different buckling pressure for each potential buckle height. The

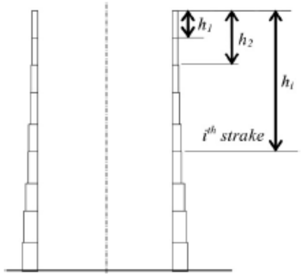


Fig. 3. The definition of h_i, the distance from the top to the base of a course.

buckle height with the lowest buckling pressure is then the final critical buckling mode for this set of wall thicknesses.

5. Verification of the weighted smeared wall method against finite element calculations

The verification of the weighted smeared wall method was first established [3] by using a defined set of wall thicknesses given in Annex A of the API standard 650 [14]. This wall thickness distribution naturally has a critical buckling height, but the verification was required to cover a much larger range of buckling modes with different heights. To test the method thoroughly, many different wall thickness distributions were used leading to many different buckling modes. Six different starting patterns of wall thickness were used. From each, a set of designs with different wall thickness distributions was produced by varying the step change between one course and its immediate neighbour. By scaling all the changes by the same amount, a new wall design is achieved on each occasion. A scaling that leads to no change from one course to the next produces a uniform wall, for which the buckle extends over the full height. A high scaling makes the second course much thicker than the first, which makes the buckle occur only in the top course. Intermediate scalings lead to buckles that occur in multiple course but do not extend over the full height. To retain generality, the same top course thickness t_1 was retained for all these wall profiles.

The scaling was undertaken as follows. The thickness of the thinnest courses t_{b1} was used as the reference thickness. The thickness of any other course was taken as the thickness of the thinnest course t_{b1} plus k times the difference between that course's reference value t_{bi} and the thinnest course t_{b1}. Thus $k=1$ corresponds identically to the reference design. A value of $k=0$ corresponds to a uniform wall and a large value for k gives a rapid increase in thicknesses, causing the buckles to be only in the thinnest course. The adjusted thickness of the ith course is obtained as

$$t_i=t_{b1}+k(t_{bi}-t_{b1}) \tag{8}$$

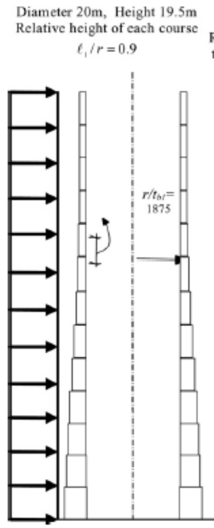
This device systematically transforms a reference design into a large range of alternative designs, all of which have quite different buckling modes. It is important to recognise that this process was used only to achieve a wide verification of the method, and is not a part of the method itself.

A reference design and the adjusted thicknesses of each course after two sample adjustments ($k=0.1$ and $k=4$) are shown in Fig. 4, where the thickness of each course has been made dimensionless relative to the top course as t_i/t_{b1}. Very different patterns of wall thickness distribution were achieved by adjusting the values of the factor k in Eq. (8) (Fig. 4).

A brief further verification of the process outlined in [3] is shown here to give a good understanding of the calculation process. The results of the thickness smearing process for several different cylindrical wall thickness distributions are shown in Fig. 5. Each line corresponds to a different cylinder design, and the points on the line indicate the equivalent thickness determined using Eqs. (6) and (7) for the chosen buckle height. These equivalent thicknesses t_{eq} are all shown relative to the thinnest thickness t_{b1} (=4 mm) because this generalises the image.

For each potential buckle height, the equivalent thickness t_{eq} and the buckle height may be substituted directly into the classical buckling pressure equation (Eq. (4)) to obtain the weighted smeared buckling pressure $p_{cr,S}$ as

$$p_{cr,S}=0.92E\left(\frac{\sqrt{rt_{eq}}}{\ell}\right)\left(\frac{t_{eq}}{r}\right)^2=0.92E\left(\frac{r}{\ell}\right)\left(\frac{t_{eq}}{r}\right)^{2.5} \tag{9}$$



Reference thickness t_{bi}	Reference design ($k=1$) t_{bi}/t_{b1}	Relative thickness ($k=0.1$) t_i/t_{b1}	Relative thickness ($k=4$) t_i/t_{b1}
4.0 mm	1	1	1
4.0 mm	1	1	1
4.0 mm	1	1	1
4.0 mm	1	1	1
4.5 mm	1.125	1.0125	1.5
5.0 mm	1.25	1.025	2
6.0 mm	1.5	1.05	3
8.0 mm	2.0	1.1	5
10.0 mm	2.5	1.15	7
12.0 mm	3.0	1.2	9
14.0 mm	3.5	1.25	11
16.0 mm	4.0	1.3	13
20.0 mm	5.0	1.4	17

Fig. 4. Different wall thickness distributions obtained by systematically varying a reference design.

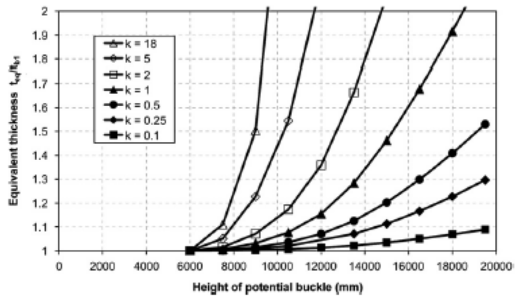


Fig. 5. Varying equivalent thickness evaluated for each potential buckle height in seven different wall thickness distributions.

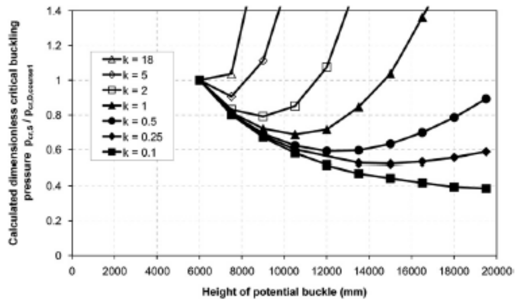


Fig. 6. Calculated weighted smeared buckling pressure for each potential buckle in several different wall thickness distributions.

where ℓ is the considered buckle height. This is valid provided the equivalent cylinder is of medium length, according to Eq. (2). But where $\ell < 20\sqrt{rt_{eq}}$, the equivalent cylinder is classed as short, an increase in the buckling pressure by the factor $C_{\theta s}$ occurs, and the increase in the weighted smeared buckling pressure $p_{cr,S}$ may be found either by the approximate formula given in Annex D of the European standard EN 1993-1-6 [1]

$$C_{\theta s} = 1 + \frac{3}{\omega_{eq}^{1.35}} \tag{10}$$

or more accurately by minimising Eq. (1) with respect to the circumferential buckling mode m. Naturally, it is the equivalent thickness that must be used in these evaluations. This process leads to a reference buckling pressure $p_{cr,S}$ for each potential buckle height.

The calculated pressures $p_{cr,S}$ are plotted in Fig. 6, again varying with the potential buckle height, and are shown relative to the classical pressure (Eq. (4)) $p_{cr,D,course1}$ for the uppermost uniform course alone. For a wall that is almost uniform in thickness ($k=0.1$) the buckling pressure drops progressively as longer and longer buckles are examined, finally resulting in the lowest pressure being associated with the full height of the cylinder (Fig. 4). By contrast, a wall that rapidly gets thicker ($k=18$) has its lowest buckling pressure for the shortest buckle (the uniform 6 mm section in Fig. 4). For intermediate cases, the buckling pressure drops as the buckle height becomes longer, but passes through a minimum before rising again, and it is this minimum that must be found for most practical wall thickness patterns.

The minimum of each curve in Fig. 6 gives not only the critical pressure, but indicates the physical height over which the buckle will form. These minima are next used to give an example verification of the accuracy of the process. The results of linear bifurcation elastic finite element predictions undertaken using ABAQUS as described in [3] are shown in Fig. 7. Two alternative base boundary conditions were used in these finite element calculations, S1 being axially restrained and S3 being axially free. For a very large range of wall thickness distributions, the predictions of the weighted smeared wall method clearly provide a close but safe estimate of the buckling strength over a wide range. It may also be noted that the finite element predictions for buckling modes that do not extend to the base are unaffected by the base boundary condition (S1 or S3). The results for low values of k, where the wall is rather uniform, are difficult to see in Fig. 7, so these are presented separately in Fig. 8.

When the wall is relatively close to uniform in thickness (Fig. 8), the buckles become very long, covering most of the wall, and the base boundary condition begins to affect the finite element predictions, with the axially unrestrained S3 boundary moving towards the C_θ factor of 1.0 (Table 1) whilst that for the axially restrained S1

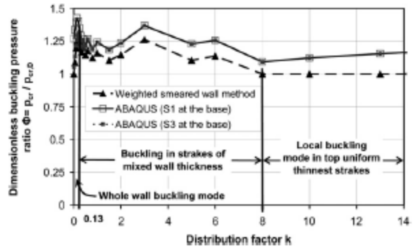


Fig. 7. Ratio of buckling pressure to the classical pressure for the top course as the wall thickness distribution is systematically varied.

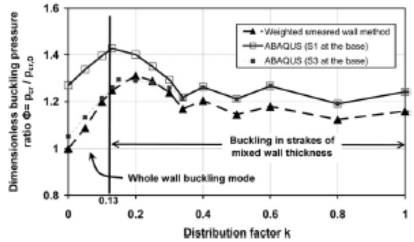


Fig. 8. Ratio of buckling pressure to the classical pressure for the top course as the wall thickness distribution is systematically varied.

boundary heads for 1.25. This separation begins whilst the critical buckle does not quite penetrate to the base, but the full $C_\theta = 1.0$ is not achieved until the wall is truly uniform. Again, the weighted smeared wall method provides a consistently conservative but quite close prediction of the linear bifurcation buckling pressure.

The above calculations indicate that both the critical buckling mode and the external pressure at buckling in a stepped wall cylinder are strongly influenced by the pattern of wall thickness distribution.

6. Adjustments to the buckling prediction of the weighted smeared wall method

From the above review and verification of this method, it is clear that the critical buckle can fall into one of three categories: buckling over the full height of the wall, buckling in only the thinnest part, or buckling involving mixed plate thicknesses but not the entire wall, as illustrated in Fig. 2. This classification easily follows the calculation described above, and the class can be used to give an empirical enhancement [3] of the weighted smeared wall prediction to bring it closer to the accurate finite element linear bifurcation result.

Where the critical buckle lies in the top thinnest part (thickness t_1), the buckling pressure is affected by the change of plate thickness that occurs in the first step (t_2/t_1). A conservative and close enhanced buckling prediction may then be obtained using the following adjustment

$$p_{cr} = [1.4 - 0.7(t_1/t_2)]p_{cr,S} \text{ with } (t_2/t_1 < 5) \tag{11}$$

Where the critical buckle covers the entire wall, the buckling strength is affected by the base boundary condition, as noted above. Tank walls that are not anchored have an S3 boundary condition which permits axial displacement during buckling, but silos and anchored tanks have an S1 boundary condition, with a consequently higher resistance to buckling under external pressure (Table 1). The buckling strength assessments for the unanchored tanks can be taken as $p_{cr} = p_{cr,S}$, which models the correct result to within about 5%. For silos and anchored tanks, the enhancement of the base anchorage depends on the ratio of the assessed equivalent thickness t_{eq} to the thickness of the bottom course t_b. An enhanced estimate can be conservatively and closely achieved using

$$p_{cr} = p_{cr,S}\left\{-1.08 + 4.4\left(\frac{t_{eq}}{t_b}\right) - 2.07\left(\frac{t_{eq}}{t_b}\right)^2\right\} \tag{12}$$

Where the critical buckling height lies between these two extreme cases, corresponding to buckling in courses of mixed thickness (Fig. 2), the base boundary condition only plays a small role for special cases where the buckle extends almost to the base, so it may be ignored. The two counteracting effects of the buckle extending into the thicker course and the elastic restraint from the thicker course effectively cancel, so that the simply supported buckling pressure is a good estimate of the final outcome, giving $p_{cr} = p_{cr,S}$.

7. An example calculation using the weighted smeared method

The design calculation is illustrated here using the tank example given in Section 11.5 of the ECCS Recommendations [2]. A tank of radius 15 m and a total height of 16 m has eight courses, each 2 m high, with plate thicknesses 6/ 6/ 8/ 11/ 12/ 14/ 16/ 19 mm. It provides a convenient illustration of the calculation process required in a spreadsheet to perform this assessment of the external buckling pressure.

There are 7 changes of plate thickness, leading to 8 possible buckle heights. The spreadsheet must be set up with the course heights and thicknesses t_i, from which the distance from the top to each change of plate thickness h_i, is determined. The calculation is then divided into a buckling prediction for each potential buckle height ℓ. For each buckle height, the weighted contribution of each course is found from Eq. (6) as

$$W_i = t_i^3\left\{(h_i - h_{i-1}) - \frac{\ell}{2\pi}\left(\sin\frac{2\pi h_i}{\ell} - \sin\frac{2\pi h_{i-1}}{\ell}\right)\right\} \tag{13}$$

with $h_0 = 0$ for the top course 1. The contributions of the courses that lie within the buckle are summed, then divided by the length, and the cube root taken to obtain the equivalent thickness t_{eq} (Eq. (6)). The relevant cylinder length classification is found from the test $\ell > 20\sqrt{rt_{eq}}$ for a medium-length cylinder, and where this test is met, the weighted smeared buckling pressure $p_{cr,S}$ is determined directly from Eq. (9). These calculations are shown in Table 2 and identify the critical buckle height in this example as 8 m, with buckling in courses of mixed thickness. No adjustment of the result (e.g. as per Eqs. (11) and (12)) is therefore necessary.

The critical pressure for this tank is given in [2] as 2.14 kPa, but the buckle height cannot be ascertained. Table 2 indicates a critical pressure of 1.99 kPa, which is 7% lower. However, a careful linear bifurcation analysis using ABAQUS yielded the critical pressure 1.996 kPa, so Table 2 is clearly the more accurate of the two estimates.

The above calculations are very easily set up in a small spreadsheet that can be repeatedly used with different course

Table 2
Calculations to determine the critical buckle height and buckling pressure.

Buckle height ℓ (mm)	Values of h_i (mm)	Weighted contributions W_i (all × 10^6)	Equivalent thickness t_{eq} (mm)	Equivalent length parameter ω_{eq}	Critical circumferential mode m_{cr}	Critical pressure $p_{cr,S}$ (kPa)
4000	4000		6.000	13.33	38	142
6000	4000	1.043	6.494	19.22	30	**1.988**
	6000	6.006				
8000	4000	0.864	7.597	23.70	25	2.09
	6000	1.676				
	8000	0.967				
10,000	4000	0.661	8.843	27.46	22	2.45
	6000	1.981				
	8000	3.431				
	10,000	0.840				
12,000	4000	0.5067	9.901	31.14	19	2.703
	6000	1.871				
	8000	4.863				
	10,000	3.456				
	12,000	9.495				

heights, different wall thicknesses and different diameters. It may also be noted that once the buckling strength begins to rise above a minimum, as happens here where the value 2.09 kPa is determined for the 8 m high buckle, there is no need to pursue the calculations further, as indicated by Fig. 6. The calculations for buckle heights of 10 m and 12 m are therefore shown in Table 2 for clarity and completeness only.

8. Conclusions

The buckling behaviour of cylinders with stepwise variable wall thickness under uniform external pressure was studied using the weighted smeared wall method [3]. Significant further demonstrations of the accuracy of the process over a wide range of geometries were given, and a demonstration of the design calculation process has been set out. In addition, the following conclusions may be drawn:

1. The weighted smeared wall method gives estimates of the critical buckling mode and the critical buckling pressure that are close to those of accurate linear bifurcation finite element predictions.
2. Three types of buckling mode have been identified: the whole wall buckling mode, local buckling in the top uniform thinnest courses only, and moderately large buckles in courses of mixed thickness. The buckle height corresponding to the lowest buckling pressure depends on the specific wall thickness distribution.
3. The design calculations presented here can easily be established in a small spreadsheet, which can be used repeatedly with different data to give accurate predictions. This calculation is considerably easier to perform than the method set out in the European standard [1] and the ECCS Recommendations [2].

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Appendix XVI **Results of the CBA**

			Scenario 1: Flood wall			Scenario 2: Tank A			Scenario 2: Tank B			Scenario 2: Tank C		
yr	tanks constructed	Total tanks	Costs [€]	Benefits [€]	Net [€]	Costs [€]	Benefits [€]	Net [€]	Costs [€]	Benefits [€]	Net [€]	Costs [€]	Benefits [€]	Net [€]
1	4	4	4.60E+07	4.80E+06	-4.12E+07	1.92E+07	4.80E+06	-1.44E+07	1.85E+07	4.80E+06	-1.37E+07	7.76E+06	4.80E+06	-2.96E+06
2	4	8	6.00E+06	9.60E+06	3.60E+06	1.92E+07	9.60E+06	-9.56E+06	1.85E+07	9.60E+06	-8.90E+06	7.76E+06	9.60E+06	1.84E+06
3	4	12	6.00E+06	1.44E+07	8.40E+06	1.92E+07	1.44E+07	-4.76E+06	1.85E+07	1.44E+07	-4.10E+06	7.76E+06	1.44E+07	6.64E+06
4	4	16	6.00E+06	1.92E+07	1.32E+07	1.92E+07	1.92E+07	4.08E+04	1.85E+07	1.92E+07	7.00E+05	7.76E+06	1.92E+07	1.14E+07
5	4	20	6.00E+06	2.40E+07	1.80E+07	1.92E+07	2.40E+07	4.84E+06	1.85E+07	2.40E+07	5.50E+06	7.76E+06	2.40E+07	1.62E+07
6		20		2.40E+07	2.40E+07		2.40E+07	2.40E+07		2.40E+07	2.40E+07		2.40E+07	2.40E+07
7		20		2.40E+07	2.40E+07		2.40E+07	2.40E+07		2.40E+07	2.40E+07		2.40E+07	2.40E+07
8		20		2.40E+07	2.40E+07		2.40E+07	2.40E+07		2.40E+07	2.40E+07		2.40E+07	2.40E+07
9		20		2.40E+07	2.40E+07		2.40E+07	2.40E+07		2.40E+07	2.40E+07		2.40E+07	2.40E+07
10		20		2.40E+07	2.40E+07		2.40E+07	2.40E+07		2.40E+07	2.40E+07		2.40E+07	2.40E+07
11		20		2.40E+07	2.40E+07		2.40E+07	2.40E+07		2.40E+07	2.40E+07		2.40E+07	2.40E+07
12		20		2.40E+07	2.40E+07		2.40E+07	2.40E+07		2.40E+07	2.40E+07		2.40E+07	2.40E+07
13		20		2.40E+07	2.40E+07		2.40E+07	2.40E+07		2.40E+07	2.40E+07		2.40E+07	2.40E+07
14		20		2.40E+07	2.40E+07		2.40E+07	2.40E+07		2.40E+07	2.40E+07		2.40E+07	2.40E+07
15		20		2.40E+07	2.40E+07		2.40E+07	2.40E+07		2.40E+07	2.40E+07		2.40E+07	2.40E+07
16		20		2.40E+07	2.40E+07		2.40E+07	2.40E+07		2.40E+07	2.40E+07		2.40E+07	2.40E+07
17		20		2.40E+07	2.40E+07		2.40E+07	2.40E+07		2.40E+07	2.40E+07		2.40E+07	2.40E+07
18		20		2.40E+07	2.40E+07		2.40E+07	2.40E+07		2.40E+07	2.40E+07		2.40E+07	2.40E+07
19		20		2.40E+07	2.40E+07		2.40E+07	2.40E+07		2.40E+07	2.40E+07		2.40E+07	2.40E+07
20		20		2.40E+07	2.40E+07		2.40E+07	2.40E+07		2.40E+07	2.40E+07		2.40E+07	2.40E+07
				NPV	€ 261,531,371		NPV	€ 239,422,768		NPV	€ 242,483,909		NPV	€ 292,378,841

Results of the CBA for scenario 1 and scenario 2 with all 3 tank variants.
Assumptions for the calculation are given in Table 9-5.