Preface

This report is written for the Master of Science Project, CT 4061, of the faculty of Civil Engineering of 'Delft University of Technology' (TU Delft), the Netherlands. It is a multidisciplinary project, executed by four students, of which Rik Beekx is specialized in Hydraulic Engineering, Ingrid Jensen is specialized in Water Resources Management and Yvonne Mikkers and Jojanneke Dirksen are specialized in Sanitary Engineering. The project is hosted by 'Escuela Superior Politécnica del Litoral' (Espol), in Gauyaquil, Ecuador.

This document is the final report of the project. All information, analysis, conclusions and recommendations are gathered in this report. It has been written with the consultation of the supervisors in Delft and in Guayaquil.

We would like to take the opportunity to thank our supervisors,

Prof. van der Graaf, ir. Veldkamp, ir. H.J. Verhagen, ing. J.Chang, ing. Jara, ing. Torres, ing. Sanchez, ing. Vargas , Teresa Vera and all of the staff of Espol and the other persons who made our stay in Ecuador pleasant and our project a success.

We also thank our sponsors,

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Abstract

This report is about several issues concerning the oil refinery of Petroecuador in La Libertad, Ecuador.

The first issue involves the accessibility of the pier of the refinery where coastal tankers moor to load or unload their cargo. The current depth at the pier is too small for large coastal tankers to fully load when mooring at the pier. In order to increase their cargo capacity, the depth in the mooring area should be increased. To expand the present pier into deeper water would be too expensive. Another option is dredging. The stability of the pier, dolphins and gantry might be endangered by dredging activities, but since only 2 to 3 meters of soil need to be dredged, and because the gantry has a reach of 10 m., it is possible to design the dredging area far enough from the pier to not have an effect on the pier- and gantry stability. The dolphins are closer to the dredging area and need to be replaced. The new dolphin design is adjusted to the dredging depth. The new mooring area is only accessible for the largest mooring coastal tankers when fully loaded during several hours a day, because of the tides. Therefore they might have to wait in the mooring area for several hours if the pier is occupied. Smaller or unloaded tankers can reach the mooring area at any time.

The second issue is about oil leakage from the refinery into the surrounding soil. On the area of the refinery of La Libertad petrochemical substances have been leaking into the subsoil. This has lead to the contamination of the soil around the refinery of La Libertad. Most of the pollution is found on the area of the refinery itself. However, on the ocean side of the refinery the contamination reaches out of the refinery's territory. A gasoline smell can be detected and petrochemical substances have been found in the subsoil of the beach. In this area there is a primary school, a military campus and a little village called La Carioca, that all suffer from this contamination. The source of the contamination of the beach area was a leaking pipe, upstream of the beach. This pipe has been traced and has been replaced by a new pipe. The contamination of the soil in this area causes risks for the human health and for the environment. To reduce the impact of the pollution in the contaminated area on the ocean side of the refinery of La Libertad different alternatives have been generated. Only methods have been used that involve groundwater or groundwater flow. The best way to reduce the impact of the contamination is to construct a slurry wall between the ocean and the contaminated plume. On the downstream side of the contaminated groundwater plume three extraction wells are located. The plume is entirely captured in the combined capture zone of these wells so the contamination can not spread any further. On the upstream side of the contaminated groundwater plume an injection well is located. This injection well makes sure that the plume can be removed more quickly. Additives can be put in the water that is injected through the injection well. This will cause more components of the oil mixture to dissolve in the water so more of the pollution can be removed from the soil.

The third issue discussed in this report is on the API separator of the refinery. The city of La Libertad has plans to enhance the ocean site and the boulevards in order to make the city more appealing for tourists. The refinery situated near the beach at La Libertad discharges its process water partially untreated onto the beach. The discharged water is clearly polluted, oil stains are present and a gasoline smell can be detected. Therefore the situation at the refinery concerning the wastewater treatment is investigated.

These investigations have shown that there are three main water flows at the refinery. These are: the process water, treated in API-Separator 1, drainage water of tanks where refined oil is stored, treated in API-Separator 2 and cooling water. The cooling water is not treated at all. Since all three flows are combined in one pipe before entering the beach, they all could be a source of pollution. Water samples of the water flows have been taken and indicated that all three flows were highly polluted and do not meet the standards set in Ecuadorian environmental laws. Inspections of the existing API separators revealed that these are not being operated wisely, furthermore their designs are not conform regulations of the American Petroleum Institute (API). Water samples taken of the in- and effluent of both API-Separators demonstrated that the removal efficiency of the separators is 95%. Analysis of the water quality of the cooling water made it clear that this water flow is the main source of pollution. Unfortunately it was not possible to analyse this wastewater further to determine the exact composition of the water. Therefore it was impossible to investigate the possible treatment methods for this water.

Because years of experiences have indicated that treatment of wastewater contaminated with hydrocarbons is done most efficiently with an API-Separator this report only focuses on this treatment method.

The functioning of API-Separators has been examined thoroughly, resulting in two alternatives to achieve a better effluent quality. The first alternative is about the possibilities to improve the existing separators. Although the suggested alterations would probably result in a better treatment this alternative is rejected because it is not possible to meet the design criteria. The second alternative involves a complete new API-Separator which replaces the existing two. The design was made according the API standards resulting in the most efficient treatment of wastewater. Therefore this alternative is considered the most desirable.

Even though implementing a new treatment facility is the best alternative, it is not very likely that will be accomplished in the near future. At the refinery the environmental problems are numerous. The refinery does not regard it of primary concern to diminish these sources of pollution. Therefore it is recommended to at least adjust the existing API separators in order to enhance the effluent quality.

1. Introduction

The Santa Elena peninsula is situated at the west coast of Ecuador, see the circle in figure 1.1. This region used to be an agricultural area, but due to a lack of water and deforestation it has become a semi-desert. Many inhabitants searched their fortune elsewhere in the country. In the seventies the government started petrochemical activities in the region (state owned company Petroecuador, of which Petroindustrial runs the refinery in La Libertad and Petrocommercial deals with the commercial issues). This resulted in an economical boost but it had a negative effect on the environment.

Figure 1.1: Map of Ecuador

In the beginning of 1999 Espol's faculty of maritime engineering amongst others, started a project in order to redevelop the peninsula. Part of this project is a design for the coastal protection of and the improvement of the tourist industry by regenerating the beaches of La Libertad.

La Libertad is primarily a port, with as its major activities the petrochemical industry, commerce and fishery. It is also an interesting place to develop for tourism. A plan view of the coastal zone of La Libertad is shown in figure 1.2.

Figure 1.2: Plan view of the coastal zone of La Libertad

The beach of La Libertad itself could be used for tourist purposes if the beach would be enlarged. At this moment five T-groynes are being constructed there to protect the coast of La Libertad against severe wave attack. This also leads to sand accretion for the beach. North of La Libertad is the beach 'Playa Cautivo', that also can be interesting for tourism. In order to be attractive for tourists however, it should not be contaminated and it might be enlarged.

At the northeast side of the beach of La Libertad an oil refinery can be found. Crude oil is transported through pipes from oil fields on the peninsula to the refinery. At the ocean side of the refinery a pier has been made in order to pump the oil in and out of oil tankers. Also some buoys in front of the coast are being used for the same purpose. The current depth at the pier is too small for large coastal tankers to fully load when mooring at the pier.

North of the pier a pipe with process cooling water (contaminated with oil) discharges into the ocean. Inhabitants of that area have been complaining about the unhealthy petrol smell around the refinery. Especially on the ocean side, the smell is very bad. To investigate the complaints, holes have been dug. They clearly show that oil is leaking out of the plant.

This report is about several issues concerning the oil refinery of Petroecuador in La Libertad. In chapter 2 the accessibility of the pier of Petroindustrial in La Libertad is discussed. In chapter 3 a study is made of the oil leakage from the refinery into the soil. In chapter 4 the wastewater treatment at the refinery is investigated.

2. Accessibility of the pier of Petroindustrial in La Libertad

The accessibility of the pier at the oil refinery in La Libertad is limited because of the limited depth around the pier head where the tankers moor. This chapter deals with this topic and proposes some possible solutions, taking into account the loads on the pier, the resistance of the pier, the possibility to use dredged soil for beach nourishment and the possibility that this soil is polluted.

2.1 Introduction

The petroleum industry in La Libertad uses several ways to transport oil products to and from the refinery. There are several terrestrial pipelines going inland. Also there are some buoys positioned about 4.5 km off the coast to load and unload large tankers (> 40,000 tons), that can't approach the shore because of there draught. There is also a pier where tankers with less draught can moor. This pier is equipped with pipelines that go directly into the refinery. Figure 2.1 shows the pier.

Figure 2.1: Pier with pipelines leading into the refinery (September 2005)

The reason why only small tankers can moor at the pier is that the depth around the pier head is only about 4.5 meters in respect to mean low tide (MLWS). This can be seen in annex 2.1. Figure 2.2 gives an impression of depths in the coastal area relative to MLWS.

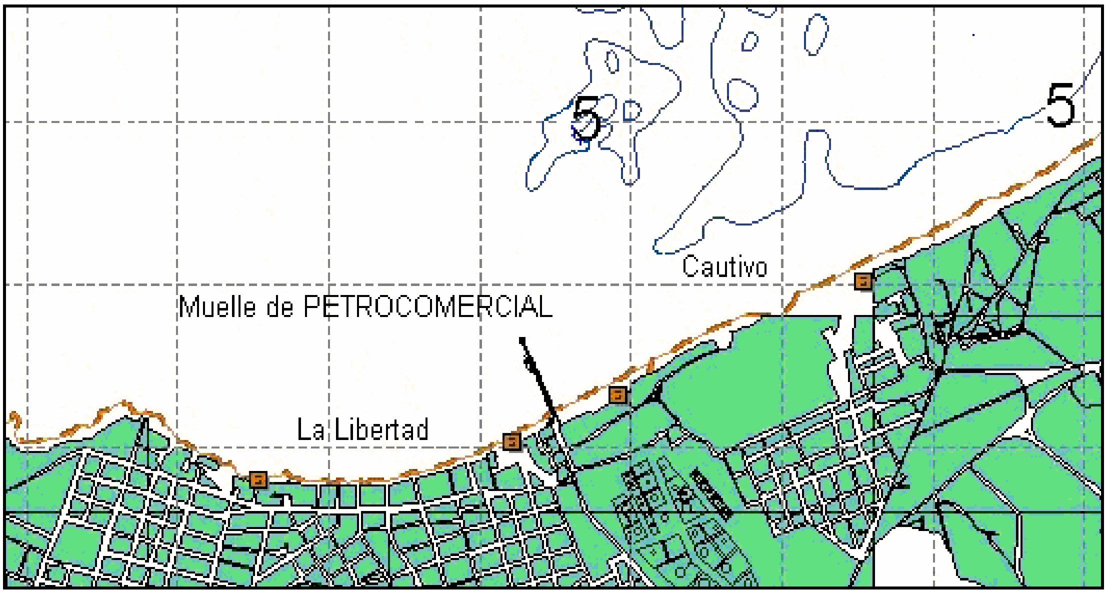


Figure 2.2: Depth contours of –5 m. MLWS. Source: [2.1]. (1 square ~ 1500 m x 1500 m)

The limited depth near the pier can have several consequences for the ships that moor at the pier:

- Only ships with a small draught can moor.
- Large ships can only moor when they are not completely filled (see figure 2.3).
- Tides limit the loading/unloading time for mooring ships.

Figure 2.3: ship mooring at the pier, not utilizing maximum draught (September 9th 2005)

The depth around the pier head has not changed much since the pier was designed (-4.3 m. MLWS, 1968), but the amounts and sizes of ships that have to load or unload at the refinery have increased since than. The coastal tankers that moor at the pier are mostly not fully loaded, because their draught would be too large to reach the pier. On average 25 coastal tankers, that require a minimum depth between 5.0 m. and 7.0 m. when fully loaded, moor at the pier. The filling of a coastal tanker takes about 24 hours, and the capacity of the pier is too small. Therefore the tankers have to wait (often for several days) before they can moor (figure 2.4).

The off shore buoy that large tankers use is expensive (4.5 km of pipeline) and the capacity of the present buoy is insufficient. A new buoy is going to be placed even further off shore to increase the mooring capacity for large tankers. Because of the lack of mooring capacity it is not an option for the coastal tankers to moor at the buoys as well.

Figure 2.4: Coastal tankers waiting to be loaded/unloaded at the pier (September 9th 2005)

The beaches of La Libertad are suitable for tourism, and there are plans to develop the beach of La Libertad (south of the pier, figure 2.5) and Playa Cautivo (north of the pier, figure 2.5). 5 T-groynes are being built at the beach of La Libertad, to protect the shore and to accrete sand for the beach. Beach nourishment using the sand from dredging around the pier is an option. The quality of the sand is doubtful, because of some pollution sources near the pier (figure 2.5).

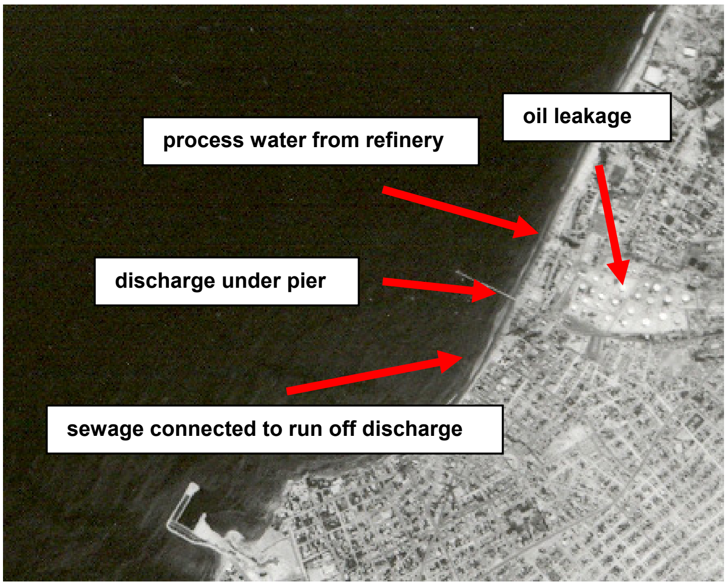


Figure 2.5: Sources of pollution near the pier

The leakage of oil from the refinery is analyzed in chapter 3. The discharge of the process water from the refinery is dealt with in chapter 4. The effects of these different sources of pollution have to be taken into account when analyzing the possibility to nourish the beach with sand from this area. The problem of beach erosion, and the availability of sand near the pier are illustrated by figure 2.6.

Figure 2.6: Aerial view of eroded beaches and sediment around the pier

2.2 Analysis

When dealing with the accessibility of the pier in La Libertad, several issues have to be taken into account. The background information has already been given in the previous paragraph.

- The largest coastal tankers that moor at the pier need a depth of – 7.0 m MLWS, if they are fully loaded and during mean low tide. Yet the depth at the pier head is only – 4.5 m MLWS. The limited depth is the main reason for reviewing the pier design.

- The beach of La Libertad and Playa Cautivo would benefit from nourishment. Tourism in the area of La Libertad would increase if the beaches were to be enlarged.

- The sand around the pier might be polluted. In that case the sand cannot be used for beach nourishment or it should be cleaned first, which brings along extra costs.

The relation between these seemingly different issues is considerable. If dredging around the pier is considered an option, the sand could be used for beach nourishment. If polluted however, the cleaning of this soil might become expensive and other options could be considered. The sources of the pollution should be analyzed as well. The pollution sources are the subjects of chapter 3 and 4 of this report.

2.3 Objective of this chapter

The objective of this chapter is to analyze whether it is feasible to either extend the pier of Petroindustrial or to increase the depth in the area around the pier in order that ships with a larger draught or more cargo can moor at the pier or can be mooring for a longer period.

2.4 Requirements

In this paragraph the boundary conditions, the constraints and the assumptions will be stated. These result in the program of demands.

2.4.1 Boundary Conditions

In this subparagraph the conditions resulting from law, nature or other elements outside the designer himself are stated.

- It is desirable that coastal tankers with the size stated in table 2.1 are able to moor at any time, while fully loaded.

Table 2.1: Data of the largest ship to moor at the pier (source: [2.3])

Data of Design Ship	
Type	Coastal tanker
Length	99.10 m.
Width	13.20 m.
Maximum Draught	6.20 m.
Dead Weight	3,436 tons
Cargo Capacity	3,300 tons
Minimum depth needed with maximum cargo	7.0 m.

- On average 25 coastal tankers per month moor at the pier.
- The different mooring tankers need a depth of 5.0 m to 7.0 m when fully loaded.
- The depth of the mooring area is optimal with a minimum of 7.0 m.
- The route for the ships to and from the mooring place must have at least the same depth as the mooring depth.
- The pier is in a poor state and needs maintenance (source: [2.4]).
- Tidal data [2.1] tabla 6 and annex 2.2: the difference between mean high tide and mean low tide is 2.5 meters.

MHWS = + 2.50 m MLWS

- Alongshore current velocities,

From [2.1] some data are derived on the alongshore-current velocities at La Libertad (figure 2.7).

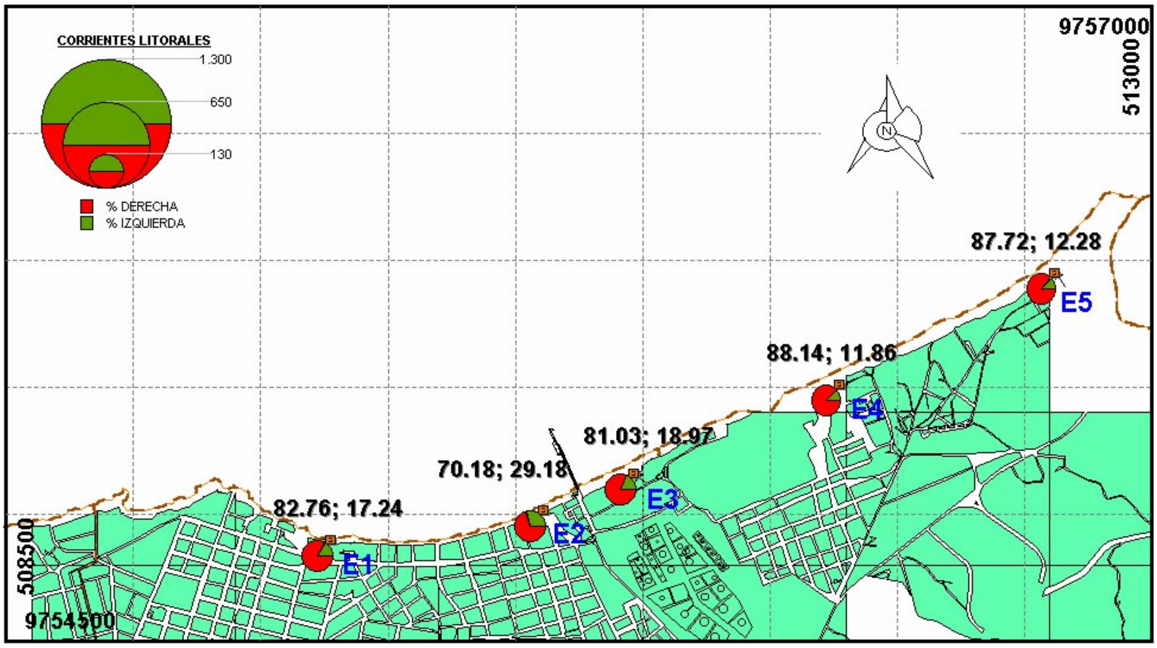


Figure 2.7: Data of measuring stations at the coast of La Libertad. Red: eastward current, green westward current. (1 square ~ 1500 m x 1500 m)

For the design current near the pier especially station E2 and E3 are interesting. The largest velocity of the current measured there was in station E3, with a magnitude of 0.29 m/s eastward. Applying a safety factor of 1.5 (ultimate limit state, dynamic load, [2.5] table 25.1), the design **current velocity in ultimate limit state** is:

u = 0.435 m/s.

- Wave height in ultimate limit state

Because the expected lifetime of the pier is already passed (see subparagraph 2.4.2), a wave height with an occurrence probability of once in a hundred years is considered acceptable for the new design. From [2.1], figura 22 and tabla 14, it can be read that the significant wave height in that case is 2.30 m. (according to the method *'Mayencon'*). This is not the maximum wave height, but because of the large safety period of 100 years, and the shallow water at the coast that causes the waves to break, this is considered an acceptable value. A safety factor will not be applied to this value, since it is a probabilistic value. Hence the design value of the **wave height for the ultimate limit state** is:

H = 2.30 m

2.4.2 Constraints

In this subparagraph the restrictions that are imposed by the designer are stated.

- The pier was designed for a lifetime of about 30 to 35 years, and was built in 1969. This means that it has passed its designed lifetime. Therefore a design storm with an occurrence probability of **once in a hundred years** is considered acceptable.
- If the soil is polluted it cannot be used for beach nourishment, or it has to be cleaned first.
- Sedimentation and erosion processes must be taken into account.
- Proposed alternatives may not have a negative effect on the local oil industry.
- The costs of the project may not exceed its benefits.

2.4.3 Assumptions

In this subparagraph the assumptions that are made in order to come to the conclusions of this chapter are stated. Major assumptions should be checked afterwards in order to validate the conclusions. If it is necessary to check the assumption afterwards, it is also stated in the recommendations.

- The T-groynes that are being built (September 2005) at the coast of La Libertad have no significant influence on the alongshore current or the occurring waves near the pier (measurements date from 1994 to 2000) because they are situated 300 meters West from the pier and are only 30 meters into the ocean. Calculations on the influence of the breakwaters on the pier have been done during the design of the breakwaters [2.7]. The result of several model tests running the program GENESYS show that no significant effects on currents, waves, sedimentation and erosion occur near the pier as a result of the breakwaters.
- The sources of the pollution will be taken care of in order to keep the beach attractive for tourism. If this is not done, nourishment of the beaches is useless.
- The financial benefits should be larger than the costs of the project. This should be further analyzed. This is an economical analysis and not suitable for this technical report.
- The soil that is dredged can be used to nourish the beach of La Libertad, or playa Cautivo.
- The reparations mentioned in [2.8] are carried out correctly and completely and have the designed effect on the load distribution of the piles.

2.4.4 Program of demands

The requirements that have to be met are stated in a program of demands.

- The design depth in the mooring area is – 7.0 m. MLWS.
- The pier must be able to carry its own load and the loads induced by tides, waves and currents.
- The dolphin must be able to carry its own load and loads induced by tides, waves, currents and accidental contact with a mooring coastal tanker.
- The gantry carrying structure must be able to carry it's own load and the loads induced by tides, waves and currents.
- The coastal tankers need to be able to reach, to moor at and to leave the pier.

2.5 Alternatives

In this paragraph several alternatives will be analyzed. A qualitative comparison will be made between each alternative and this will also be compared with the present situation. Finally, using a multi criteria analysis, a preferred alternative will be chosen of which a design with more computations will be made in the next paragraph.

2.5.1 Maintaining the present situation

The buoys that are used for large tankers (> 40,000 tons) already have a lack of capacity, so it is not an option for the coastal tankers to use these. Tankers have to wait several days before they can moor at the pier. On average 25 coastal tankers moor at the pier each month. If the coastal tankers keep using the pier as they do now, mostly not completely filled, this is an inefficient manner of transporting their cargo. The lifetime of the pier is already passed. An economic analysis should be made of whether it is feasible to maintain the present situation, especially while the pier is in poor shape and needs maintenance. It could be that the best solution would be to completely replace the pier. Since direct costs however play a major role, this is not considered a feasible possibility.

2.5.2 Pier adaptation

An option to enlarge the depth around the pier is to expand the pier further into the ocean. From the depth report in annex 2.1 (the extended version in Autocad is used to determine the distance) it can be seen that the coast is very shallow in this part. This means that the pier would have to be expanded over more than 1,700 m. to reach the optimum depth of – 7.0 m. MLWS. (Compare this to the length of the present pier, which is 330 m. To reach a depth of – 5.0 m. MLWS it would be necessary to expand the pier over 170 m.) This large distance would make the expansion of the pier far to expensive, so just expanding the pier is not considered a good option. However if by dredging the stability of the pier would be undermined, partly expanding the pier in combination with dredging might be an option. The new part of the pier can be made more stable so dredging can be done without problems. Also other adaptations to the pier can be considered.

2.5.3 Dredging

Since the present pier is in poor shape, dredging near the pier involves some risk. The stability of the pier after dredging has to be analyzed, since the dredging will certainly affect the strength and stability of the piles that are founded in the soil that will be partly dredged. It is expected that in order to reach the optimal depth of – 7.0 m. MLWS some adjustments need to be made on the pier design, especially on the dolphins and the gantry carrying structure. The effects of scour, erosion and sedimentation have to be taken into account in the design, so neither the depth nor the stability of the pier will be endangered. If dredging is done, the dredged soil can be used for beach nourishment at La Libertad. The soil should then not be polluted or it should be cleaned.

2.5.4 Multi criteria analysis

The previously mentioned alternatives will now be compared with each other using a multi criteria analysis. The criteria that are mentioned in the previous paragraph are used. The result can be seen in table 2.2. (+ means that it has a positive effect, – means that has a negative effect and 0 means that it has no significant effect). The table is explained further in this subparagraph.

Table 2.2: Multi Criteria Analysis

Alternative criterion	Present situation	Pier expansion	Dredging	Adapt design + Dredging
1. Costs	–	– – – –	– –	– – –
2. Economy - refinery	–	+ +	+ +	+ +
3. Economy - tourism	–	0	+ +	+
4. Durability	– –	0	– –	+
5. Functionality	– –	0	+	+ +
6. Feasibility	0	–	0	+
7. Environment	–	0	+	+
8. Scenery	0	–	0	0
9. Risk	–	– –	–	0
Result	**– 9**	**– 6**	**+ 1**	**+ 5**

Present situation:

1. The costs are negative, because the pier needs maintenance.
2. The coastal tankers cannot be completely filled; this is bad for the economy of the refinery.
3. The beach stays polluted and small; this is bad for the tourist economy.
4. The pier is in poor condition.
5. The pier does not function properly (too little depth).
6. Feasibility is not an issue in the present situation since it has already been built.
7. The polluted soil and beaches are negative for the environment.
8. A pier is already there so the scenery remains unchanged.
9. The risk of failure of the pier is present because of the poor condition of the pier [2.4].

Pier expansion:

1. Because of the large distance over which the pier would have to be expanded, it is a very expensive alternative.
2. The coastal tankers will be able to carry their maximum load, which is beneficial to the refinery economy.
3. It will not have a negative effect on the present tourist activity in the area.
4. The durability of the present part of the pier would be increased by maintenance work, but because of the size of the new pier part, the durability remains questionable.
5. The gain in functionality for larger loads to be loaded or unloaded is undone by the necessary maintenance for such a large pier.
6. The large scale of the project makes the feasibility questionable.
7. There might be changes in the environment due to the effects of the pier, but they are not necessarily negative.
8. A large pier like this would be negative for the scenery, although it might be argued that it is a tourist attraction.
9. A large pier stretching into the ocean brings along large risks.

Dredging

1. Dredging brings along costs.
2. The coastal tankers will be able to carry maximum load, which is beneficial to the refinery economy.
3. The dredged soil can be used (if it is clean enough) to nourish the beach of La Libertad.
4. The durability is influenced in a negative manner because the present pier is already in a poor state and might be damaged by the dredging.
5. Coastal tankers with a larger draught may be able to moor; however this depends on the possible dredge depth.
6. The feasibility depends on the stability of the pier.
7. If the dredged soil is polluted it will be either cleaned or removed, so this is good for the environment.
8. Dredging has no influence on the scenery.
9. Dredging might endanger the stability of the pier.

Combination of an adapted design and dredging

1. Dredging and adapting the design bring along costs.
2. The coastal tankers will be able to carry their maximum load, which is beneficial to the refinery economy.
3. The dredged soil can be used (if it is clean enough) to nourish the beach of La Libertad.
4. If this alternative is carried out correctly the durability should not be an issue (the present part of the pier should however be repaired).
5. Coastal tankers that require a depth of – 7.0 m. MLWS will be able to moor fully loaded, even during mean low tide, so the functionality is fine.
6. This alternative is considered feasible.
7. If the dredged soil is polluted it will be either cleaned or removed. This is good for the environment.
8. This alternative has no significant effect on the scenery.
9. The dredging brings along a risk for the stability of the pier, but with an adapted design for the pier, the dolphins and/or the gantry carrying structure, the risks are controllable.

Conclusion on the Multi Criteria Analysis

A general idea has been created of the advantages and disadvantages of some different alternatives. The alternative of ***an adapted design combined with dredging*** is clearly the preferred design (table 2.2). The advantage of this alternative is that the adaptation of the pier, dolphins and/or gantry carrying structure will allow the dredging to reach the desired depth, so the tankers will be able to carry their full cargo capacity while mooring at the pier. Which adaptations will be necessary will be analyzed further on in this chapter.

2.6 Design calculations

The coastal tankers moor near the pier head using mooring buoys and their anchors. They are attached to the pipelines of the refinery using a gantry (figure 2.8). To protect the pier, several dolphins are placed. The tankers are not supposed to come in contact with the pier or the dolphins.

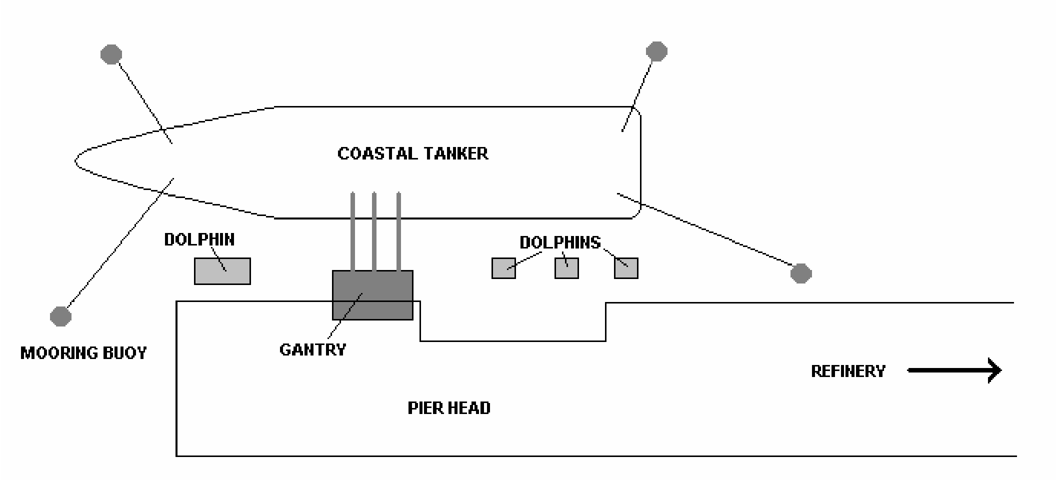


Figure 2.8: Schematization of a coastal tanker mooring at the pier (plan view)

To calculate the stability of the pier with the optimal dredging depth, the following key elements will be analyzed;

- The vertical loads of the pier and the piles on the subsoil.
- The loads on the pier resulting from currents and waves in ultimate limit state.
- The loads on the dolphins resulting from design ship impact, combined with wave and current loads.
- The characteristics of the soil in which the piles of the pier and the dolphins are founded, in order to determine the soil resistance.
- The resistance of the pier as a whole.
- The resistance of the dolphins.
- The design of the gantry.
- Scour, erosion and sedimentation effects.

It should be noted that the conditions in the project area are rather ideal for a pier and the mooring process. Mooring operations at the pier are rarely delayed because of high waves (waves over 2.00 m. rarely occur). This is mainly caused by the shallow coastal zone with small depths, which causes waves to break before they can reach large heights.

2.6.1 Vertical loads on the pier

If a ship (with the design values for the measures) moors at the head of the pier, it lays besides pile group 28 to 35 (annex 2.2), and further into the ocean. These pile groups have all about the same vertical load, depth and design [2.8]. To make a general calculation of the possible dredge depth, the heaviest load of pile groups 28 to 35, with a safety factor applied, in combination with the smallest carrying capacity will be taken as general design values for the pier. In table 2.3 all vertical design loads on the subsoil are calculated. Figure 2.9 shows the structural design of the pier.

Figure 2.9: Piles, beams and crossbeams with pipes

The design of the dolphins and the gantry are not suited for any change in depth [2.9], because the dolphins are partly founded on the top soil layers and the gantry piles only reach a depth of – 6.13 m. MLWS. So if dredging near the dolphin or gantry is applied, the structures of the gantry and the dolphins have to be changed. Figure 2.10 shows a dolphin, the gantry and the structure that is carrying the gantry.

Figure 2.10: Dolphin, gantry and gantry carrying structure

Table 2.3: Vertical loads of the pier (data from [2.8] and [2.9])

Piles					
Length of a pile	L	=	13	m	
Diameter of a pile	D	=	0.45	m	
Material of a pile					reinforced concrete
Mass of a pile	M	=	4745	kg	
Number of piles supporting crossbeam	N	=	4		
Crossbeams					
Height of a beam	H	=	0.75	m	The beam is not prismatic
Width of a beam	W	=	0.45	m	
Length of a beam	L	=	7.6	m	
Material of a beam					reinforced concrete
Number of beams per pile group	N	=	1		
Mass of a beam	M	=	19700	kg	
T-beams					
Height of a beam	H	=	0.73	m	The beams are not prismatic
Width of a beam	W	=	0.6	m	
Length of a beam	L	=	9.6	m	
Material of a beam					reinforced concrete
Number of beams left side (# 35)	N	=	8		
Number of beams right side (# 33)	N	=	6		
Total mass of a T-beams per pile group	M	=	33650	kg	
Deck					
Height of the deck	H	=	0.1	m	
Width of the deck	W	=	4	m	
Material of the deck					concrete
Mass of deck per pile group	M	=	9600	kg	
Extra					
Building ('caseta')			8650	kg	
Part of weight from building carried by # 34			0.25		
Total weight of pipes for water and oil			6189	kg	
Asphalt layer			10000	kg	
Variable load			-	-	negligible
Total mass per pile			**26692**	**kg**	
Safety factor			**1.2**		
Design load per pile			**32031**	**kg**	
Equivalent mass per pile			**23529**	**kg**	1/3(pile mass) + other vertical loads

2.6.2 Loads on the pier resulting from the alongshore current

The coastal currents around the piles of the pier induce drag and lift forces. These forces are calculated in this subparagraph. The following calculations are done according to the method mentioned in [2.5], chapter 6. The values used can be found in § 2.4 of this report.

The piles of the pier are cylindrical. The diameter of the piles is 0.45 m. [2.8]. Hence:

Re = $^{ud}/_{v}$ = 0.435 x 0.45 / 10^{-6} = **195,750** with:

Re	= the Reynolds number	
u	= design velocity of the alongshore current	= 0.435 m/s
v	= kinematical viscosity	= 10^{-6} m^2/s
D	= diameter of a pile	= 0.45 m

With this value from [2.5] figure 6.1 the value of the static drag coefficient (Cd) can be read.

Cd = 1.2

The distance between the piles is 0.55 m [2.8] Hence:

L/d = 0.55 m / 0.45 m = 1.22

Since the piles are cylindrical and the alongshore current is generally perpendicular to the strongest direction of the pile rows, the influence of obliquely incident currents will be ignored (also the maximum current velocity is always alongshore). Hence the static lift coefficient (Cl) is:

Cl = 0

From [2.5] figure 6.4, the minimum value for the Strouhal number (S) is read:

S = 0.18

Hence the frequency of vortex shedding (fs) becomes:

fs = uS/D = (0.435 x 0.18) / 0.45 = **0.174 Hz**

In order to calculate the own frequency of the pier, it is first schematized to a mass-spring system as shown in figure 2.11. Damping from the water has been ignored in this schematization.

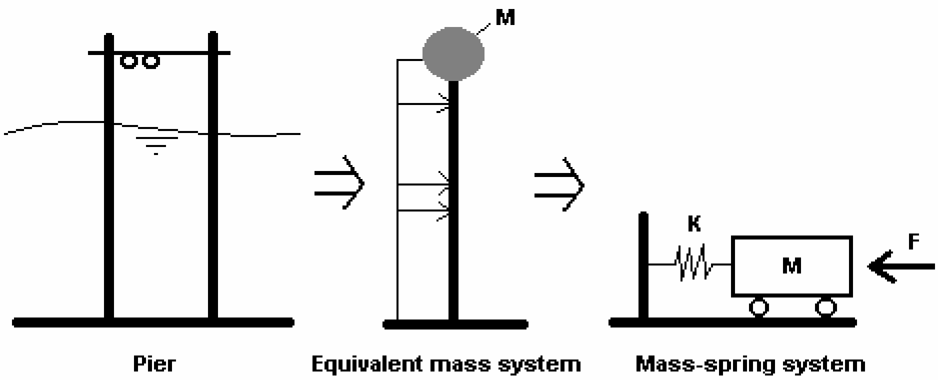


Figure 2.11: Schematization of the pier to a mass-spring system

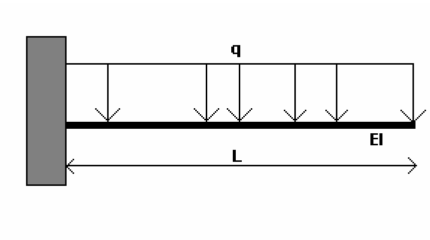
The connection of the piles with the pier deck is simple and will be seen as a hinge like connection. Therefore every pile can be analyzed by itself, for as far as dynamics are concerned. The mass of the pier and its equivalent mass are obtained from table 2.7.

M_{eq} = 1/3(pile mass) + other vertical loads = **23,529 kg**

The moment of inertia (I) of a pile can be calculated as follows:

I = ¼ pi (½ D)^4 = **2 x 10^9 mm^4**

The spring constant (k) can be calculated using the following relations:



$$w = \frac{ql^4}{8EI}$$

Figure 2.12: Relation between w, q, L and EI

E = Elasticity modulus of concrete **= 30,000 N/mm^2**

k = F / w = (8EIqL / (qL^4) = 8EI/L^3 **= 280 N/mm**

With these data the natural frequency can be calculated:

$$\omega_n = \sqrt{\frac{k}{m}} = 0.11 \text{ rad/s} \qquad f_n = \frac{\omega_n}{2\pi} = 0.017 \text{ Hz}$$

Hence **fs > fn**. This means that there is no immediate danger of resonance occurring.

Also the dynamic drag coefficient (C'd) can be calculated, using the maximum value for C'd as:

C'd = 0.25 Cd = **0.3**

Since fs/fn = 10 (not near 1), an acceptable design value for the dynamic lift coefficient (C'l) is:

C'l = C'd = **0.3**

With these data the drag force (Fd) and lift force (Fl, perpendicular to u) can be calculated using the equations from [2.5] chapter 6.

F_d = (½ ρ u^2) (Cd + C'd) A = **786 N** per pile

F_l = (½ ρ u^2) (Cl + C'l) A = **157 N** per pile

With;	ρ	= density of the seawater	= 1025 kg/m^3
	u	= current velocity	= 0.435 m/s
	Cd	= static drag coefficient	= 1.2
	C'd	= dynamic drag coefficient	= 0.3
	Cl	= static lift coefficient	= 0
	C'l	= dynamic lift coefficient	= 0.3
	d	= diameter of the pile	= 0.45 m
	h	= height of the pile (from ground level to top)	= 12 m
	A	= surface projected in current direction = d x h	= 5.4 m^2

2.6.3 Loads on the pier resulting from waves

The wave activity in the coastal zone induces drag and lift forces on the pier. These forces are calculated in this subparagraph. The following calculations are done according to the method mentioned in [2.5], chapter 14. The values used can be found in § 2.4 of this report.

From the depth chart (annex 2.1) it can be seen that the coastal zone is very shallow. High waves will have started braking long before they reach the pier, so the method for braking waves from [2.5] § 15 will be used.

The maximum force (Fmax) and moment (Mmax) resulting from waves on a pile can be calculated with the formulas:

$$\mathbf{F_{max}} = C^*_D K_D H^2 \tfrac{1}{2} \rho g D = \mathbf{20{,}923\ N}$$

$$\mathbf{M_{max}} = F_{max}\, d S_d = \mathbf{162{,}573\ Nm}$$

C_D	= drag coefficient	= 0.7	C^*_D	= 2.5 Cd	= 1.75
K_D	= drag correction	= 1.0	S_D	= resultant drag correction	= 1.11
H	= wave height	= 2.3 m	D	= pile diameter	= 0.45 m
d	= depth	= 12 m	g	= gravity coefficient	= 9.8 m/s^2
Ts	= wave period	= 19 s			

2.6.4 Loads resulting from accidental contact with the dolphins

Though hydraulic structures are normally not designed for accidental contact with ships, these dolphins are only meant for this exact purpose. Therefore they will be analyzed for an accidental ship impact at maximum mooring velocity. Mooring loads are according to [2.5] § 23. Figure 2.13 shows the fenders that are used to protect the dolphins.

Figure 2.13: Fenders used to protect the dolphins

Fender stiffness = **k ≈ 100 kN/m (rubber fender)**

Mooring velocity = $\mathbf{v_m}$ **= 0.30 m/s** (maximum for a ship < 10,000 ton)

Water displacement = $m_w = \rho L \frac{1}{4} \pi D^2$ = 3,066,696 kg = **3,067 tons**

Hydrodynamic coefficient = $C_h = \dfrac{m_s + m_w}{m_s}$ = **1.46**

Volumetric water displacement of the ship (when fully loaded, using Archimedes' law);

I = (3436 + 3300) tons x 1000 kg/ton x 1 l/kg x 0,001 m^3/l = **6,736 m^3**

Block coefficient

$C_b = \dfrac{I}{BLD}$ = 6.736/(13,20 x 99,10 x 6,20) = **0.83**

Inertia radius of the design ship;

k = (0,19 Cb + 0,11) L = (0,19 x 0,83 + 0,11) 99,10 = **26.5 m**

Radius of the contact point to the mass center point of the ship is assumed ¼ of the ships length;

r = 25 m

The maximum angle between the aforementioned radius and the velocity of the ship is assumed;

γ = 10°

This gives for the eccentricity coefficient:

$C_E = \dfrac{k^2 + r^2 \cos^2(\gamma)}{k^2 + r^2}$ **≈ 1**

Because of the use of fenders, the ships haul is considered relatively stiff:

Softness coefficient = **Cs = 1.0**

Because the pier construction is very open, no hydrodynamic damping of any significance will occur:

Configuration coefficient = **Cc = 1.0**

With these data the kinetic energy that has to be absorbed by a dolphin can be calculated:

$\mathbf{E_{kin}}$ = $\frac{1}{2} m_s v_s^2 C_H C_E C_S C_C$ = ½ x 6,736 x 0.30^2 x 1.46 x 1 x 1 x 1 = **435 kJ**

From this the maximum force induced on the dolphin by a ship can be calculated:

$F = \sqrt{2kE_{kin}}$ = **152 kN**

Because during the mooring the ship is attached only to buoys and its anchors (see figure 2.8), no mooring rope loads are induced on the dolphins or on the pier.

2.6.5 Vertical soil resistance, method Terzaghi

Because a Standard Penetration Test (SPT) was used to determine the soil characteristics, in [2.8] the method of 'Terzaghi' [2.12] is used to determine the vertical soil resistance, and hence the bearing capacity of a pile. This will be replicated here, with some minor changes because of the changed depth after dredging. The result is comparable with the result from [2.8], as it should be. The soil characteristics used in these calculations are based on sample P-C (taken between half and the end of the pier) from [2.8].

From pile group 22 onward to the Oceanside of the pier, the soil layers are very similar, comparing the thickness and consistency of the soil layers (see annex 2.3). Therefore it is considered acceptable to generalize the layers of the soil with the data of table 2.4.

In [2.8] the loading capacity of the piles is calculated using the method 'Terzaghi'. A similar calculation for the maximum depth with the design loads is done in table 2.8. The capacity of a pile, without dredging, in pile group 34 is **58.77** tons [2.8]. For the design calculations, mostly the same values will be taken as the values stated in [2.8], since the only difference is the depth as a result of the dredging, and only the friction component of the top layers will therefore change. The bearing capacity of the pile tip remains unchanged (only for a small factor, because in [2.8] a pile diameter of 0.44 m is used, while in the design drawings [2.9] this is 0.45 m.)

Table 2.4: Soil characteristics & bearing capacity calculation

layer	***depth***	***soil type***	***N***	***a***	***p***	***L***	***Leq***	***c***	***Nc***	***Nq***	***Nγ***	***Df***	***γ***	***N0***	***De***	***γF***	***qac***	***qc***
unit	m MSLW			t/m2	m	m	m	kg/cm2				m	t/m3				t/m2	t
1	-4.6 - -5.4	loose soil																
2	-5.4 - -7.2	SM - loose	9.5	1.2	1.41	0.2	0.2											0.29
3	-7.2 - -14	CH - hard	25.63	6.75	1.41	4	3.4											32.4
4	-11.2	pile point	20.67		1.41			1.25	5.7	1	0	3.08	2	1	1.76	3.08	74.33	11.8
Total																		**44.6**

The formula used to calculate the bearing capacity from friction is:

qc	**= a x p x L**	
a	= adhesion between piles and soil	= 1.2 t/m^2 (from – 5.4 m to – 7.2 m. MLWS)
		= 6.75 t/m^2 (from – 7.2 m to - 14 m MLWS)
p	= pile perimeter = πD	= 1.41 m
Leq	= effective length of the pile	= 0.85 x length in soil

The formula used to calculate the pile tip bearing capacity is:

$qac = c \times Nc + \gamma \times Df \times Nq + \frac{1}{2} \times B\gamma \times N\gamma$

c = material cohesion below the pile tip	= 1.25 kg/cm^2 (-11.2 MLWS)
γ = Natural density of the soil	= 2 t/m^3
Df = depth	= 3.08 m (lowest value 'cuadro 5' [2.8])
Nc = correction factor	= 5.7
Nq = correction factor	= 1
Nγ = correction factor	= 0

The correction factors are based on an angle of internal friction of 0° (the lowest value is used since the exact value is not known.)

The representative soil characteristics, used for these calculations, are shown in figure 2.14 (also see annex 2.3), and in table 2.4. In this table also the bearing capacity is calculated.

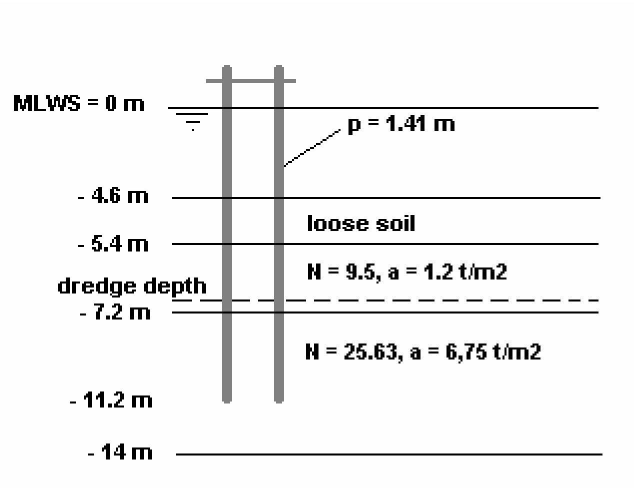


Figure 2.14: Soil characteristics

With a dredge depth of – 7 m MLWS, using the method Terzaghi, the bearing capacity per pile is:

44.6 tons.

2.6.6 Horizontal soil resistance, method Blum

To analyze the horizontal soil resistance, the method 'Blum' as described in [2.5] § 39 is used. Blum uses the following formula to calculate the maximum horizontal bearing capacity at the top of a pile:

$P = \gamma' K_P \frac{t_0^{\ 3}}{24} \cdot \frac{t_0 + 4b}{t_0 + h}$ **= 13,073N** per pile

with:

$t_0 = \frac{t}{1,2}$ = 3.5 m

t = depth of the pile in the ground = 4.2 m

φ' = angle of internal friction of the soil ([2.5] table 26-3, clay) = 22.5 º

$K_P = \frac{1 + \sin(\varphi')}{1 - \sin(\varphi')}$ (Passive ground pressure coefficient, according to Rankine, [2.5] § 31.4)

Kp = 2.24

γ' = effective soil density (under water) = 9,555 N/m^3

$x_M^{\ 2}(x_M + 3b) = \frac{t_0^{\ 3}}{4} \cdot \frac{t_0 + 4b}{t_0 + h}$

x_M = depth of maximum moment = 1.20 m

b = width of the pile (diameter) = 0.45 m

h = height of the pile above the soil = 12 m

$\delta = \frac{P(h + 0,65t)^3}{3EI}$ = horizontal deformation = 231 mm

Mmax = maximum moment in pile = 207 kNm

In fact this formula is for a force at the top of the pile (like loads from mooring ships), and the case of wave and current loads has more the character of a distributed load along the pile, which causes the resultant force to act lower on the pile, which is positive for the bearing capacity. However since the moment caused by a distributed load is smaller than the calculated force P, this value will be taken as the design force for the horizontal bearing capacity. An extra safety factor will not be applied in this particular case.

2.6.7 Design combinations of loads and resistance

To determine the design loads on the pier, the worst-case scenario of the possible occurring loads is taken. In table 2.5 the result can be read. All values stated in table 2.5 have been calculated in the previous subparagraphs.

Drag forces by waves are only perpendicular to the coast, because of the refraction in the shallow coastal zone. Lift forces of waves are therefore only parallel to the shore. Drag forces from the alongshore current are only parallel to the shore and lift forces of the current therefore are only perpendicular to the shore. The design direction is parallel to the shore, because then the loads are distributed over only 2 pile rows instead of in the cross-shore direction, where the loads are distributed over 35 pile rows (so there is more spare capacity to absorb the energy of currents and waves).

Table 2.5: Design combinations of loads and resistance on the pier

	Max. value	Safety factor	Design value
Loads			
Vertical (N)	261585	1.2	313901
Horizontal (N)	786	1.5	1178
Resistance			
Vertical (N)	436638	1.2	363865
Horizontal (N)	13073	1.2	10894
Reliability	***Vertical***	***Horizontal***	
	1.16	**9.25**	

Table 2.5 shows that, taking into account some safety factors, the reliability (quotient of resistance and load) of the pier after dredging to a depth of – 7 m MLWS is 1.16 concerning vertical loads and resistance, and 9.88 concerning horizontal loads and resistance (a construction is considered safe if the reliability exceeds the value 1). This means that after dredging, only little vertical extra bearing capacity is available, but a lot of horizontal extra bearing capacity is available. Therefore, no adaptation of the pier will be necessary in order to allow the dredging activities. The dolphins and the gantry carrying structure need to be adapted in order to have them function properly after the dredging activities have been executed, unless the dredging area is situated far enough from these structures. In § 2.7 the designs of these structures are adapted to suit the dredging depth.

It should be noted that at this moment pile group 32 is settling for an unknown reason (it is thought that it might be a consequence of fishing boats accidentally hitting these piles, when coming to charge diesel. The pier is not designed for ship impact). Until it is known exactly what causes this settlement, no dredging should be done at all!

2.6.8 Scour, erosion and sedimentation

Because of the small difference between depth and draught, the chance exists that because of the propellers of large coastal tankers, the sandy soil under the tanker is stirred. Therefore bed protection might be necessary. Also natural coastal processes of sediment transport have to be taken into account. Calculations will be done according to the methods mentioned in [2.10], chapter 9.

Propeller load on the bed

d = 0.7 x 0.7 x 3.5 m = 1.7 m
Pmax = 10,000 kW
P = 5,000 kW (full power will not be used during mooring)
u0 = 1.15 $(P/\rho d^2)^{1/3}$ = 13.7 m/s
zb = 2 m
$u_b$max = 0.3 x u0 x d/zb = 3.5 m/s

It is expected that, in the mooring area, this load exceeds the wave and current loads (compare the maximum current velocity of 0.435 m/s), and therefore the load resulting from the propeller of the coastal tanker is taken as the design load, with a maximum velocity at the bed of:

$u_b$max = 3.5 m/s

Bed protection

For soil stability in the jet flow of a propeller, an Izbash type of equation is used to calculate the necessary stone diameter of the bed protection:

d_{n50} = 2,5 x $(u_b^2/2g\Delta)$ = **0.9 m**
(The slope correction factor is 1 for a horizontal bed protection.)

The scour depth without bed protection, according [2.10] equation (9.19), is:

$$h_s = x\sqrt{\frac{-\ln(\frac{u_c x}{5.6 u_0 d})}{15.7}} - z_b$$ = 2 m (maximum)

If bed protection would be applied, several layers of rock with d_{50} of 0.9 m would have to be installed, while the scour depth will reach only 2 m. Therefore it is advised not to apply bed protection, but to take the scour depth into account in the design. It is clear that because of this scour the stability of the pier is endangered if the dredging area is to be near the pier. Therefore in the design, the dredging area is placed further from the pier (max 10 m, this is the reach of the gantry).

Scour depth around piles

To calculate the scour depth around the piles, formula 4.7 given in [2.10] page 82 is used:

$$h_s/D = 2K_sK_\alpha K_u \tanh(h_0/D)$$

Since the scour depth increases with depth, the design depth after dredging during mean high tide is used, h_0 = 9.5 m.

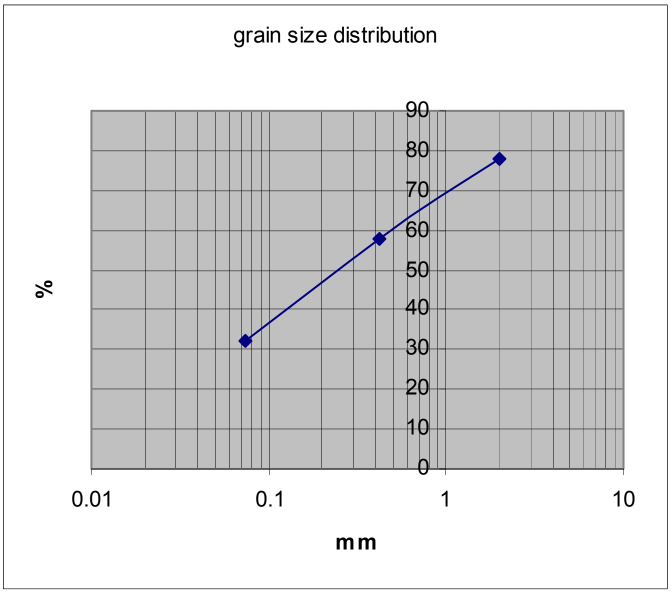
The piles are cylindrical, so K_s = 1 and K_α = 1.

Furthermore, u = 0.435 m/s and D = 0.45 m

For the grain diameter, the value of d50 will be used. Since there are only few data available, an estimate will be made using sample P-E depth 0.0 – 0.45 m. (measured from the soil surface) from [2.8], that was taken near the end of the pier (see annex 2.3) (This estimation will not be very accurate because of the lack of data). The analysis is shown in table 2.6 and figure 2.15.

Table 2.6: Grain size distribution

sample	depth		diameter		weight		d50	
P-E	0.00-0.45	m	2.000	mm	78	%	0.3	mm
			0.420	mm	58	%		
			0.074	mm	32	%		



Firgure 2.15: Graph of grain size distribution

This gives for the dimensionless diameter d* ([2.10] figure 3.2b, Shields-Van Rijn Graph):

$$d^* = 0.0003 \times (1.65 \times 9.8/(1.33 \times 10^{-6})^2)^{1/3} = 6.3$$

$$\psi = 0.045$$

kr is assumed to be 2 x d = 0.0006 m

The Chezy coefficient then becomes:

C	$= 18\log(12R/kr) = 18\log(12 \times 0.45/0.0006)$	$= 71\ \sqrt{m}/s$
u_c	$= \sqrt{(\Delta d\ \psi\ C^2)} = \sqrt{(1.65 \times 0.0003 \times 0.045 \times 71^2)}$	$= 0.34$ m/s (critical velocity)
u/u_c	$= 0.435/0.34$	$= 1.28$
Ku	$= (2 \times u/u_c – 1)$	$= 1.56$
h_s/D	$= 2 \times 1 \times 1 \times 1.56 \times \tanh(9.5/0.45)$	$= 3.12$

Hence the scour depth becomes:

$\mathbf{h_s} = 3.12 \times D = 3.12 \times 0.45\ m = \mathbf{1.4\ m}$

(This is a very high value considering the rule of thumb: hs = 2D = 2 x 0.45 m = 0.9 m. The difference is caused by a very small grain diameter.)

This depth has to be taken into account in the design of the dredging area. Applying a mild sloping bed profile from the pier to the mooring area will overcome the level difference created by the scour depth.

Erosion and sedimentation

Due to sedimentation and erosion processes, the bottom profile did not change very much in the past decades. The area that has to be dredged in order for the tankers to reach the pier can be dredged deeper than near the pier, because this will not compromise the stability of the pier. Therefore it is advised to dredge this area to a depth of – 9 m MLWS and to carry out annual inspections to monitor the depth in this area. If the depth decreases under – 8.7 m MLWS (=1.4 x draught), maintenance dredging should be carried out to maintain the passageway to the pier. It is also advised to mark the passageway with buoys to prevent ships from using shallower water.

If the costs of having a ship wait until high tide, to reach and leave the mooring area, are lower than dredging a canal, this might also be an option. Then extra waiting places can be made in the mooring area, so the loading and unloading of the tankers can continue, while other tankers wait in the mooring area until they can moor. Small tankers would neither have to use the canal, nor wait for high tide.

2.7 Design of adaptations

In the previous paragraphs several characteristic values have been calculated in order to make some adaptations in the present design of the pier to make it possible to increase the depth around the pier head using dredging. The general principle of the adapted design is shown in figure 2.16.

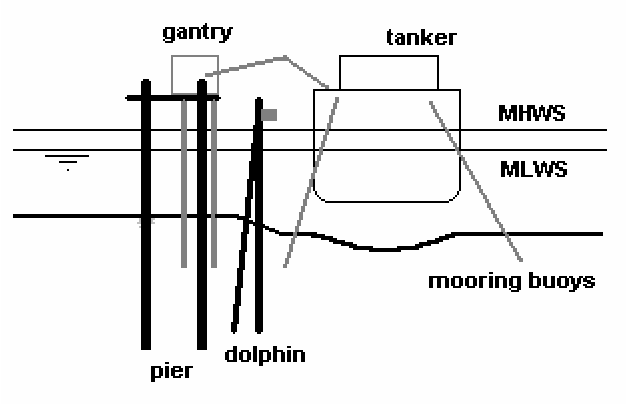


Figure 2.16: General principle of adapted design

To determine the dredge profile, not only the desired depth of –7 m MLWS has to be taken into account, but also the height of the bottom protection and the scour around the piles. Dredging a canal to the mooring area is optional, and a cost estimate of this will be made.

The Dolphins will have to be removed and rebuilt using an adapted design, in order to make dredging possible. The gantry and the pier itself are outside the influential area of the dredging, if the dredging is done far enough from the pier. The design of the dredging area will be made with the principle that the design ship must be able to moor at all times and load/unload using the gantry. The maximum reach of the gantry is 10 m.

In this paragraph the dredging area and depths, and the adapted design of the dolphins is presented. Technical drawings of the designs are included in annexes 2.3 and 2.4.

2.7.1 Dredging

To reach and leave the pier, two options can be considered. First, a two-way canal could be dredged in order that the largest coastal tankers that moor at the pier can reach and leave the pier at any time, high tide or low tide. This is however costly, so a second option is considered. Instead of dredging a canal, it can be accepted that large coastal tankers that are fully loaded can only reach the pier during a certain period around high tide. If the pier is occupied if they get there, they can wait in the mooring area until the pier is free. If they are unloaded, they can leave also at low tide. (For ships that come to load cargo, it is the other way around.)

To estimate the costs of dredging a canal, the following computations are done. Measures are determined using [2.11]. A standard cross-section of a canal is shown in figure 2.17. Using this figure, the dimensions of the canal that has to be dredged are determined.

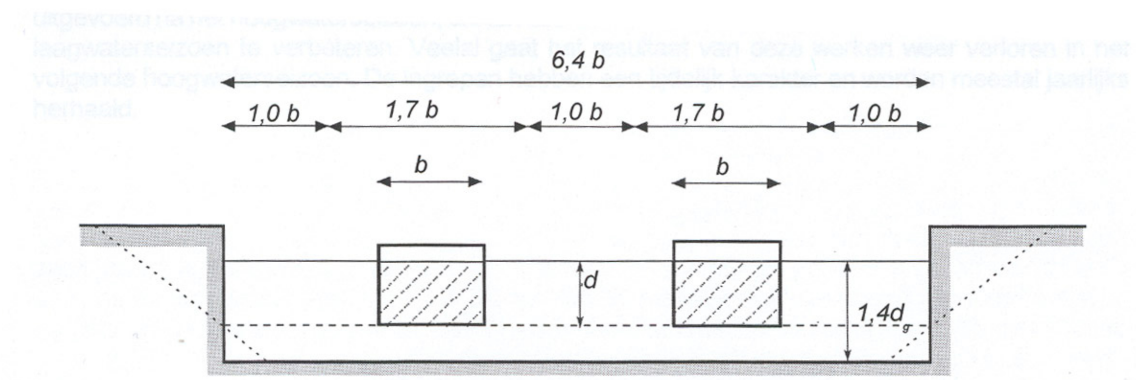


Figure 2.17: Standard cross-section of a canal [2.11]

b = width of the design ship = 13.20 m.

d = draught of the design ship = 6.20 m.

A mild slope using the natural angle of internal friction of the soil will be applied, but this will not be included in the design, because the bottom profile differs along the canal trajectory. It is expected that if a profile with a width of 6.4 x b and a depth of – 9 m. MLWS is dredged, natural processes will create an acceptable canal profile (the maximum increase in depth will be 4 meters, and 2.8 m of width on both sides at bed level is available for slopes anyway). The minimum depth for the canal is 1.4 x 6.2 = 8.7 m, but because of possible sedimentation processes, a **depth of - 9 m. MLWS** is applied, and annual inspections of the depth of the canal should be carried out. The **width of the canal** at the bed level will be 6.4 x b **= 84.5 m**. In this way a 2-way canal will be created in order for coastal tankers to reach the pier. (Dredging a one-way canal may also be considered, because a ship will only be fully loaded one-way, and empty the other way. But in case one ship comes to load, and the next to unload, it is more practical to have a two-way canal.) In order to connect the canal to water with a depth of – 7 MLWS, the canal has to be dredged over a distance of about 1.7 km to the north of the pier (see annex 2.1).

Costs of dredging a canal

The minimum costs of dredging a canal are estimated as follows:

Costs of mobilizing dredging equipment:

$ 100,000

(the equipment has to be mobilized for the dredging of the mooring area anyway, so this is not counted)

Costs of demobilizing dredging equipment:

$ 50,000

(the equipment has to be demobilized for the dredging of the mooring area anyway, so this is not counted)

The volume to be dredged for the canal:

1,700 m x 4 m (dredge depth – average depth) x 86 m (width, taking some caving in of the sides into account) = 584,800 m^3

Costs of dredging the canal ($3.00/ m^3):

584,800 x 3 = **$ 1,754,400**

Also the costs of annual inspections have to be taken into account. Since the canal is only convenient and not really necessary, it is not included in the design.

Mooring area

The mooring area will be designed in a way that the tankers can moor within the reach of the gantry, but the pier and gantry carrying structure stability is not endangered. Also enough room for coastal tankers to moor while waiting to moor at the pier is included. Mooring buoys should be installed in this area. Technical drawings of the design of the dredging area are included in annex 2.3.

2.7.2 Dolphin design

Before the dredging can start, the dolphins have to be removed, to prevent them from falling over during the dredging. After the dredging is done, completely new dolphins will be placed. These dolphins have to carry their own weight and resist the loads from accidental contact with mooring ships. The dolphins are not meant to be used for mooring, but only serve as fenders for the pier. Technical drawings of a new dolphin design are included in annex 2.4.

The predesign of the dolphin will be based on the existing dolphin design (see figure 2.10). The design depth near the dolphin will be taken – 7 m. MLWS. The depth of the piles is taken –14 m MLWS (this is the lowest point of the Standard Penetration Test). The pile profile is square, because then it is easier to construct the top part of the dolphin. A side of 0.50 m is taken for the pile profile. The whole structure will be made of reinforced concrete, and rubber fenders will be applied. Because of the limited space between the pier and the mooring coastal tankers (the reach of the gantry is only 10 m), the dolphin design has to be small. Also the piles cannot come to close to the existing structures, because it might damage the present piles when the new piles are constructed.

Loads

The influence of current loads will be ignored, because the general current is directed in the opposite direction of the ship impact load, and it is not significant in comparison to this load. Also wave forces are negligible. The schematization of the dolphin is shown in figure 2.18. The calculation for the vertical bearing capacity is based on the method 'Terzaghi', as in § 2.6.5. The calculations are conservative and not fit for a final design.

The vertical load per pile of the dolphin structure and the pile itself is:

2300 kg/m^3 x 9.8 N/kg x ((2 x 3.5 – 0.5) x 0.5 x 0.5 x 4 (diagonals) + 4.5 x 4.5 x 0.5 (top plate) + 17.5 x 0.5 x 0.5 x 4(piles)) + 4 x 0.5 x 0.2 (fenderbeams) / 4 (amount of piles) = 195 kN

The first vertical design load per pile is: Fv = 195 kN x 1.2 = **234 kN**

The second vertical design load per pile is: Fv = 195 kN x 0.9 = **175.5 kN**

The horizontal load is the maximum ship impact load, calculated in § 2.6.4:

The total horizontal design load is: Fh = 152 kN x 1.5 = **228 kN**

Resistance

To absorb the horizontal load, a combination of straight and obliquely placed piles could work, but since the reach of the gantry is only 10 m., and the piles of the dolphin may not come too close to the piles of the gantry or the pier, the lack of space causes a need for the piles to be placed vertically. This means that the large horizontal load may cause a vertical upward force on the front pile row of the dolphin. This can be adapted by increasing the weight of the dolphin structure. A general principle for the design with the pier and the loads is shown in figure 2.18.

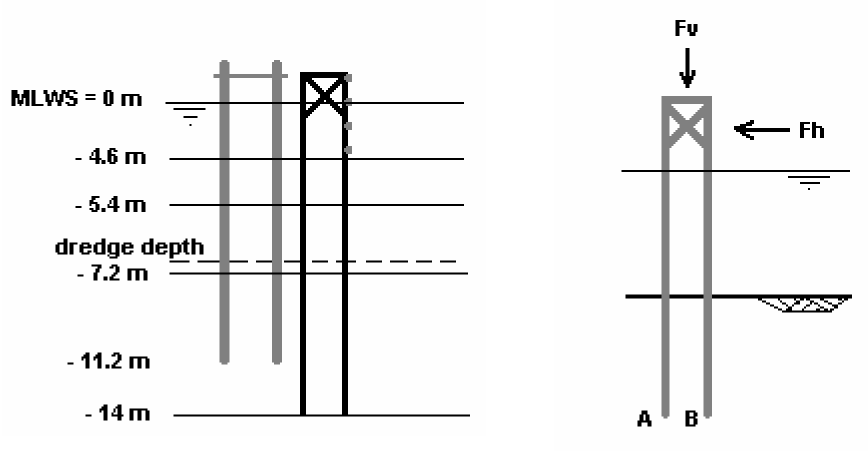


Figure 2.18: General principle of the dolphin structure

The load induced by the dolphins' own weight is schematized as one vertical force, with a resultant in the middle of the construction. The horizontal force is assumed to have a resultant force at the top construction (most unfavorable situation). To schematize the connection with the soil is very complex, a spring model could be applied and extensive computations could be done. However, for the predesign of the dolphin another approach will be used.

Vertical resistance

The bearing capacity of the soil is 74.33 t/m^2 and by friction 6.75 t/m^2. Hence the pile bearing capacity at the pile point is:

0.5 m x 0.5 m x 74.33 t/m^2 = 18.6 tons ~ 180 kN

and the pile bearing capacity from friction is:

0.5 m x 4 x (14 - 7.2) m x 6.75 t/m^2 = 91.8 tons ~ 900 kN

So the vertical bearing capacity per pile is:

180 kN + 900 kN = **1080 kN**

This is much bigger than the design load. So for the vertical load, only 1 pile would be sufficient. (this is not surprising since a pile only has to carry the dead weight of the dolphin).

Horizontal resistance

The horizontal bearing capacity is calculated with the method 'Blum', as in § 2.6.6. See table 2.7.

Table 2.7: Horizontal bearing capacity of a vertical pile

t	t0	φ'	Kp	γ'	b	h	xm	**Pmax**	**Mmax**	**δ**
6.8 m	5.67 m	22.5º	2.24	9,555 N/m3	0.5 m	10 m	2.39 m	**79.4 kN**	**1280 kNm**	**508 mm**

So at least 228 / 79.4 = 2.8 ~ 3 piles are needed to absorb the horizontal loads.

Moments

The danger exists that because of the large horizontal load, the front piles are pulled upwards. In complex models such calculations are done schematizing the soil as springs, but for this predesign, a simplification will be made. In this particular simplification, it is assumed that the soil will not take any moment, but only vertical and horizontal forces. In reality the soil will absorb some moment, which is favorable for the design.

The moment that is created by the horizontal and vertical loads at pile point level per pile (A and B in figure 2.18) is:

(17.5 m x 228 kN – 1.5 m x 702) / 4 = **734 kNm**

This can be neutralized with a couple of Av (upwards) = Bv (downwards) = 734 kNm / 3 m = 245 kN
Distract and add the most unfavorable vertical force per pile, 140.4 kN, and the vertical forces become:

Av = 245 kN – 175.5 kN = **69.5 kN** upwards
Bv = 245 kN + 234 kN = **479 kN** downwards

In reality these values will be lower, because some moment will be taken by the soil, which decreases the moment couple AB significantly.

The value Av should be smaller than the vertical bearing capacity per pile from friction, which is 900 kN (so this is correct). Bv should be smaller than the total vertical bearing capacity, which also is the case.

Pile stresses

Pmax = 79.4 kN, which creates a Mmax = 1280 kNm (table 2.7). It is assumed in this calculation that Fh (= 228 kN) is distributed over all 4 piles equally. (this assumption can be made because of the small size of the dolphin in respect to the ship size, and also because of the stiffness of the concrete structure). This gives a Fh per pile of 228 kN/4 = 57 kN. If assumed that the moment decreases linearly with the force (which in this case can be assumed because there is only one horizontal force and the distance to Mmax remains the same), then M = (57 kN / 79.4 kN) x 1280 kNm = 919 kNm. This maximum moment in the pile creates a stress of:

M / W = M / (1/6)bh^2 = 919 kNm / 0.02 m^3 = **44,112 kN/m^2** (= 44,1 Mpa) (either pressure or tension)

The maximum tension force is N / A = 109.6 kN / 0.25 m^2 = **438.4 kN/m^2**

The maximum pressure force is N / A = 390.4 kN / 0.25 m^2 = **1561.6 kN/m^2**

So the combined maximum stresses are:

Maximum pressure in a pile = 1561.6 kN/m^2 + 44,112 kN/m^2 = **45,674 kN/m^2**

The pressure can be taken if the concrete has a quality of **B 50** (~ 50,000 kN/m^2).

Maximum tension in a pile = 438.4 kN/m^2 + 44,112 kN/m^2 = 44,550 **kN/m^2**

Schematized as distributed triangularly, a load of 44,550 kN/m^2 x 0.5 m x 0.25 m x 0.5 = 2785 kN has to be taken by the reinforcement. If reinforcement with a strength of 470 N/mm^2 (~ 470.000 kN/m^2) is applied, 2,785,000 N / 470 N/mm^2 = 5,926 mm^2 of reinforcement steel has to be applied in half the cross section of a pile. This is about 9 bars with a diameter of 30 mm (Which is pretty much for this cross section, but possible).

Since the calculations were done very conservatively and with very unfavorable conditions and safety factors, it is expected that in a final design the moment will be less. This will lead to less reinforcement and a lower needed quality of concrete.

The maximum force that the dolphins can take is more than the maximum design load, but less than the maximum force the soil can take. This means the piles will collapse before the soil gives way. This is an advantage (if not a demand) for a dolphin structure. Especially in this case, where the soil stability is very important for the stability of the pier and the gantry.

2.8 Conclusions

The current depth at the pier of Petroindustrial in La Libertad is too small for large coastal tankers to fully load when mooring at the pier. In order to increase their cargo capacity, the depth in the mooring area should be increased. To expand the present pier into deeper water would be too expensive. Another option is dredging. The stability of the pier, dolphins and gantry might be endangered by dredging activities, but since only 2 to 3 meters of soil need to be dredged, and because the gantry has a reach of 10 m. (ships moor using mooring buoys and do not come in contact with the pier or the dolphins), it is possible to design the dredging area far enough from the pier to not have an effect on the pier and gantry stability. The dolphins are closer to the dredging area and need to be replaced. The new dolphin design is adjusted to the dredging depth.

The new mooring area is only accessible for the largest mooring coastal tankers when fully loaded during several hours a day, because of the tides. Therefore they might have to wait in the mooring area for several hours (but less often and less time than under the present circumstances) if the pier is occupied. Smaller or unloaded tankers can reach the mooring area any time.

2.9 Recommendations

- An analysis of whether the costs of the project are lower than the benefits should be done.
- The quality of the soil should be analyzed to determine if the soil is usable for beach nourishment.
- The sources of pollution in the area of the refinery should be dealt with. (See chapter 3 and 4)
- Maintenance activities for the pier have to be carried out (see [2.4]).
- The reason why at pile group 32 the piles are settling has to be clear before dredging starts.
- A depth report of the mooring area should be made every 2 years in order to check the damage by the propellers of the ships and sedimentation and erosion effects.
- The dredged part of the mooring area should be clearly marked with buoys
- A Cone Penetration Test instead of a Standard Penetration Test could give some more information about the present problems of the pier.
- The pier cannot withstand a ship impact load. Therefore it should be properly protected by dolphins at all places where ships come near the pier, not only where the tankers moor.
- The design of the dolphin should be adapted to a final and structural design, before constructing it.
- Because large coastal tankers can only reach the pier in a certain period, a schedule has to be made in order to make the mooring process as efficient as possible, taking into account ship sizes, cargo and waiting space in the mooring area.
- The present mooring buoys have to be reinstalled after the dredging.
- Mooring buoys for waiting ships should be installed in the waiting area.
- The piles of the dolphins should preferably be placed with a vibration-free method to prevent damage to the present structures.

2.10 References

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3. Oil leakage on the beach of La Libertad

On the area of the refinery of La Libertad petrochemical substances have been leaking into the subsoil. This has lead to the contamination of the soil on and around the refinery of La Libertad. In this chapter a study will be made of the extend of the pollution on the beach of La Libertad and possible measures to reduce the impact of this contamination will be proposed.

3.1 Introduction

Inhabitants of La Libertad, living close by the refinery, have made some complaints about a gasoline smell. On the ocean side of the refinery, holes have been dug to investigate these complaints. The holes clearly show that petrochemical substances have been leaking into the soil (see figure 3.1).

Figure 3.1: The holes that show the contamination of the soil

To get an idea about the extend of the soil contamination, soil samples have to be taken on several locations on and around the refinery (see figure 3.2). To determine the locations where the samples will be taken, the area is studied. Points that look like they may be polluted are chosen as sampling locations. So there is no logical positioning of the samples.
This method has the advantage that fewer samples are taken than with a roster of sampling points. However it has a big disadvantage, since there are only points tested that almost certainly have a high pollution rate. It will also be hard to say anything about the general extend of the soil contamination.

Figure 3.2: Soil samples

There are a lot of different sources of contamination on the site of the refinery. For example, there are some basins that contain residue from the API-separators. But these basins do not have an impermeable layer, so the pollution can infiltrate in the soil. On other locations there are leaking pipes.

These different sources of contamination are not related with each other, and can be treated as separated problems.

Figure 3.3 shows the area of the refinery in La Libertad. The area inside the blue line is the refinery of La Libertad. The small dots in on the map are locations where soil samples have been taken.

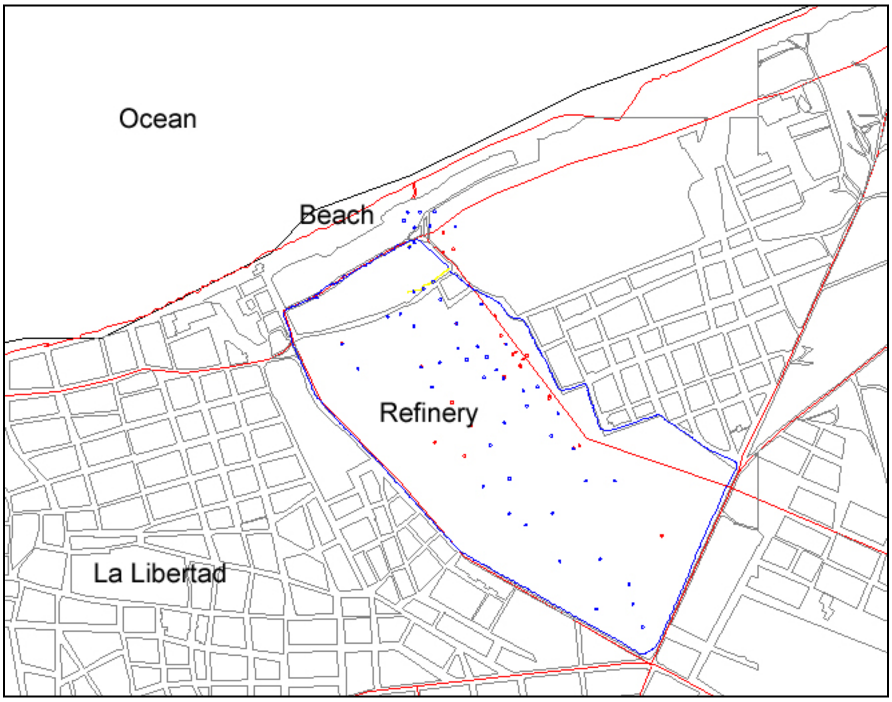


Figure 3.3: Location of the refinery of La Libertad

Most of the pollution is found on the area of the refinery itself. However, on the ocean side of the refinery the contamination reaches out of the refinery's territory and causes a treat to inhabitants of La Libertad. A gasoline smell can be detected and petrochemical substances have been found in the subsoil of the beach. In this area there is a primary school, a military campus and a little village called La Carioca, that all suffer from this contamination (see figure 3.4 and 3.5). This area will be the subject of this report.

Figure 3.4: The primary school and the village La Carioca, that suffer from the contamination

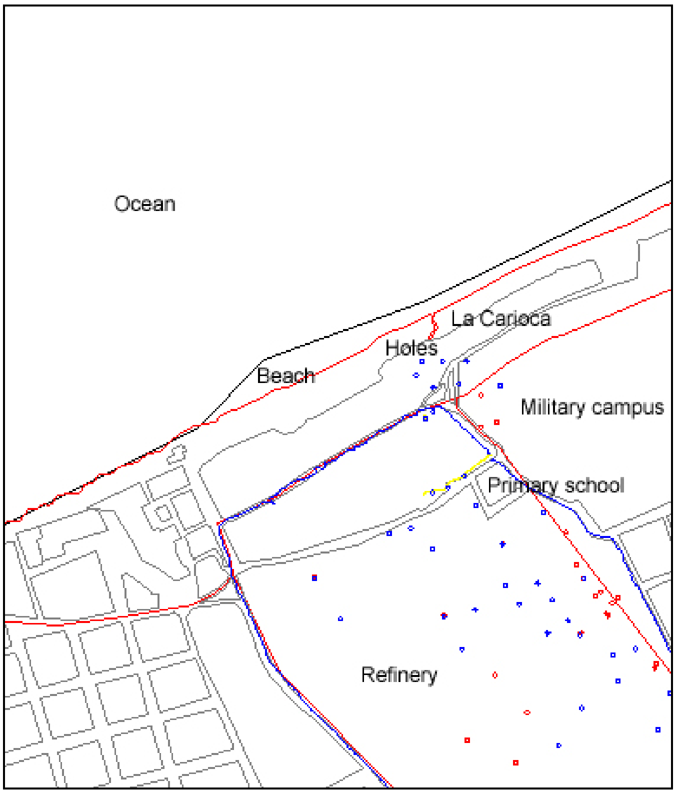


Figure 3.5: The ocean side of the refinery

The source of the contamination of the beach area was a leaking pipe, upstream of the beach. This pipe has been traced and has been replaced by a new pipe. This took away the origin of the contamination; otherwise it would not be feasible to do something about the contamination. The leakage of petrochemical substances caused the contamination of the soil, the groundwater and the ocean.

When oil leaks into the ground, it moves through the unsaturated zone under the influence of gravity. A part of the oil will be contained in this unsaturated zone. The amount of oil that is contained in the unsaturated zone depends on the oil retention capacity. If the amount of oil in the unsaturated zone is more then this oil retention capacity, oil will reach the groundwater level. Many components of an oil mixture are not highly soluble in water, so they will often be present as an immiscible (non-aqueous) phase. This separate liquid phase can serve as a long-term source of ground water contamination. Fluids that are not highly soluble in water are called non-aqueous phase liquids (NAPLs). There is a distinction between NAPLs that have a density less than water, such as gasoline or fuel, and ones that are more dense than water. The former are called light non-aqueous phase liquids (LNAPLs) and the latter dense non-aqueous phase liquids (DNAPLs).
When a LNAPL reaches the water level it will, at equilibrium, form a floating layer on the water (see figure 3.6). A DNAPL on the contrary, will sink through the water.

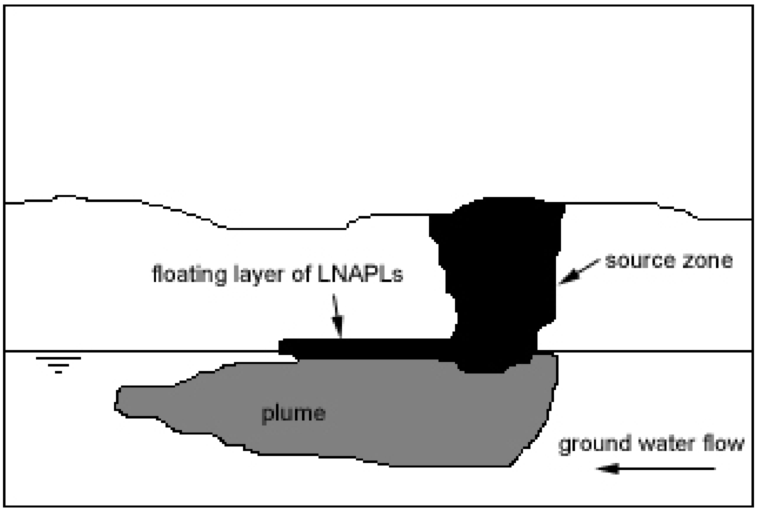


Figure 3.6: Model of LNAPL release

On molecular level, there is exchange of oil components between the different zones of the soil. Especially the better soluble oil components will dissolve in the groundwater, the more volatile oil components will spread with the air in the soil. From the groundwater and the air oil will adsorb to the solid phase of the soil. The amount of oil components dissolved in the water depends on the mass fraction of the component in the oil mixture and on the solvability in water of the component. This process is described by the law of Raoult, see [3.1]:

$$M \cdot S_w = C_w$$

with:

M	= mass fraction of the component in the oil mixture (-)
S_w	= solvability in water of the component in its pure form (μg/l)
C_w	= concentration of the component in water (μg/l)

This process is reversible. If the concentration in the groundwater decreases, the process reverses and a new equilibrium is formed.

When describing soil pollution, three zones are recognized: the source, the plume and the contaminated air in the soil.
The oil in the source zone causes the contamination of the groundwater and -air. The source zone is characterised by a large amount (kg) of oil in a small volume.
Some components of the oil mixture will dissolve in the groundwater or in infiltrating precipitation and will be spread by groundwater flow. This causes a contaminated groundwater plume. The contents and concentrations of the oil pollution in the plume changes due to dilution, retardation and breakdown. Dilution is the process of clean and contaminated groundwater mixing with each other.

This will reduce the concentration of the contamination. Retardation is the process where oil components will adsorb to organic components in the subsoil. This will change the contents and the concentration of the contaminants in the plume. Breakdown of oil components is a result of biological and chemical processes. The content and concentrations of the components will change due to these processes. The plume is characterized by a small amount of oil in a large volume. The plume can spread the contamination over a large area. This can result in a wide area that has an increased risk of health and environmental problems.
The contamination can also spread by evaporation, dissolving in the air in the ground. This evaporation happens mostly from the source, but also from the plume. The zone with contaminated air is characterized by a small amount of oil in a relatively large volume. Since this contaminated air can flow out of the soil, it can cause severe health risks over a large area.

In this case the subsoil is contaminated with petroleum products. Since petroleum products are light non-aqueous phase liquids (LNAPLs), we can assume that there are only light non-aqueous phase liquids (LNAPLs) to deal with in this situation.

3.2 Problem description

Several locations on and around the refinery of La Libertad have been contaminated by leakage of petrochemical products. This report will be focused on the contaminated area on the ocean side of the refinery. The contamination of the soil causes health problems and environmental problems. The most important humane risk is inhalation of air that has been in contact with oil, or oil components. The components of the oil mixture dissolved in the air in the subsoil can escape from this subsoil and are easily inhaled by people. Since there is a primary school, a military campus and the village La Carioca, this is a serious treat. The environmental problems consist of soil contamination, groundwater contamination and pollution of the ocean. The subsoil in the area around the leakages will be contaminated. Since this area is next to a beach, it is very well possible that the pollution will eventually reach the beach. This will be a problem for playing kids and all kind of animals. The contamination can also proceed to the ocean. This is a treat to the oceans ecosystem. The administration of La Libertad wants to make the beach area more appealing to tourists, so the pollution of this beach and the ocean is a serious problem. The last problem involves the groundwater. Due to the leakage of petrochemical products the groundwater is contaminated with oil. Because of groundwater flow the oil can be spread more widely. The Santa Elena Peninsula suffers from water shortage. The annual precipitation is only 300 mm. Water has to be brought to the peninsula from other regions for irrigation and drinking water supply. So every possible action should be taken to keep the groundwater clean.

Right now the administration of La Libertad tries to control the problem by pumping the oil out of the holes every ones in a while. This can take the smell away for a short period. Since the beach, the ocean and more importantly the groundwater can still get polluted, this is not really a sustainable solution for the problem.

3.3 Objective of this subproject

The objective of this subproject is to reduce the impact of the pollution with LNAPLs in the contaminated area on the ocean side of the refinery of La Libertad. The most important target will be to make sure that the risks for the human health and the environment are eliminated. Eliminating these risk will lead to a more attractive beach area. Another target will be to make sure that the pollution will not spread any further.

This report will be focused on reducing the impact of the LNAPLs pollution based on measures that involve groundwater and groundwater flow.

3.4 Requirements

In this paragraph the boundary conditions, the constraints, the assumptions and the program of demands are stated.

3.4.1 Boundary conditions

- The groundwater quality should meet Ecuadorian environmental laws, Leyes Ambientales: Libro VI anexo1 and RAOH (see [3.2] and [3.4]);
- The quality of the subsoil should meet Ecuadorian environmental laws, Leyes Ambientales: , Libro VI anexo 2 and RAOH (see [3.3] and [3.4]);
- Maximum amplitude of tide is 2.5 meter (see [3.5]);
- The average saturated aquifer thickness in the study area is 8 meter (see [3.6]);
- The locations and the results of the soil samples (see annex 3.2);

3.4.2 Constraints

- Only methods that involve groundwater or groundwater flow will be used;
- Any measure taken should not interfere with the refinery processes;
- The proposed measures should be implement able in Ecuador;

3.4.3 Assumptions

- The contamination only consists of light non-aqueous phase liquids (LNAPLs);
- Maximum pumping rate of a groundwater pump is about 2000 m^3/d;
- An injection well can be calculated as an negative extraction well;
- The contaminated ground water plume has the same dimensions as the contaminated area;
- There is enough groundwater available for extraction of the plume;
- The maximum drawdown is equal to the average aquifer thickness, so the maximum drawdown is 8 meter;
- The effective porosity is 25 %;
- The storativity is $4*10^{-5}$;
- The regional hydraulic gradient is 0.007;

3.4.4 Program of demands

Groundwater

According to [3.2] the maximum allowed concentration of Total Petroleum Hydrocarbons (TPH) in the groundwater is: 325 µg/l.

Subsoil

According to [3.4] the maximum allowed concentrations of Total Petroleum Hydrocarbons (TPH) in the subsoil are (see table 3.1):

Table 3.1: maximum allowed concentrations of TPH in the subsoil

agricultural function	industrial function	ecosystem
< 2500 mg/kg	< 4000 mg/kg	< 1000 mg/kg

3.5 Alternatives

3.5.1 General

To reduce the impact of the leakage of petroleum substances the source of the has to be located and taken away. The polluted area has to be remediated. The pollution can be treated in-situ, it can be removed with the soil, or it can be controlled and contained. Combinations of these methods are also possible.

<u>In-situ</u>

An important prerequisite for in-situ treatment is the removal of free floating LNAPLs. There are three principles for in-situ treatment of with oil contaminated soils:

- Evaporation
- Dissolving in water
- Disintegration and conversion

Evaporation is the mechanism that is used for vapour extraction. Fresh air is injected or flows into the subsoil. The volatile components of the oil mixture evaporate into the fresh air. The vapour-laden air is withdrawn under vacuum or using extraction wells. This technique is useful when the oil mixture contains a large amount of volatile components.

Dissolving components of the oil is the mechanism used for the interception system method, the pump and treat method and the soil flushing method. Groundwater, including dissolved components, is extracted from the ground and treated. An interception system uses natural groundwater flow to a trench to extract the water from the soil, the pump and treat method and the soil flushing method both use pumps to extract the water from the soil The groundwater level returns to its normal level, either by natural supplementation or by injection or infiltration of water. According to the law of Raoult, a part of the contaminants adsorbed onto the soil will dissolve in the water. This water is then extracted from the soil. This process repeats itself. In this way the soluble components of the oil mixture are removed from the ground. By the soil flushing method water containing additives is injected. The additives make sure that the oil, which is adsorbed to soil particles, will also dissolve in the water. The water is extracted with an extraction well and treated. By adding the additives more pollution can be removed from the ground.

By evaporation and dissolving components (a part of the) contamination is removed from the ground.

By disintegration and conversion the components are transformed into different, not harmful substances. The soil contaminants are degraded by biological breakdown and chemical oxidation. The success of this method depends strongly on the chemical characteristics of the oil mixture.

<u>Removal</u>

Another way to remove the pollution is to dispose the soil that is contaminated. The soil that has been dug away has to be disposed somewhere where there are facilities to make sure that the pollution will be contained. Another option is to treat the soil by for example soil washing or land farming. Soil washing uses water to dissolve the contaminants. It separates soil by particle size. Most contaminants bind to small particles. By separating the small particles from the rest of the soil, only a small amount of soil has to be disposed. Land farming uses microbiological processes and oxidation to degrade and transform the contaminants. The contaminated soil is mixed with soil bulking agents and nutrients, and is tilled into the ground. The contaminated soil is therefore mixed with uncontaminated soil.

<u>Control and containment</u>

Often it is not possible to effectively remove the pollution. Whether in-situ treatment is possible strongly depends on the chemical characteristics of the oil mixture. And it will often not be possible to just remove the contaminated soil due to existing functions on the surface.
In this case the containment should be controlled and contained.

3.5.2 Situation

The first action to be taken to remediate a polluted area is to stop the spill itself. The pollution of the beach area of La Libertad proves to be caused by a leaking pipe upstream of the study area. This pipe has been located and has been replaced by a new pipe. This has taken away the origin of the contamination. The next step is to remediate the contaminated area either by in-situ treatment, by removal or by control and containment as described above.
Since this subproject focuses on measurements involving groundwater and groundwater flow, only methods to remove the plume by hydraulic means and methods that control and contain the plume will be discussed.

First the extend of the contamination has to be determined. On and around the area of the refinery soil samples have been taken. Figure 3.7 shows the soil samples that have been taken in the area close to the beach.

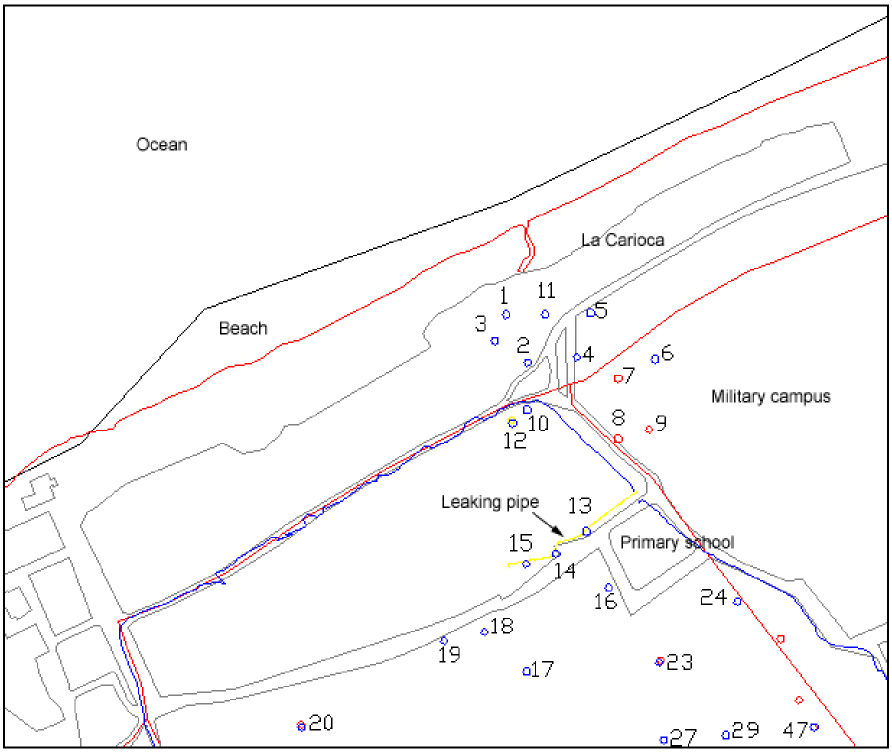


Figure 3.7: Locations of soil samples

The leaking pipe that caused the contamination of the beach area was between point 13 and 14. Figure 3.8 shows the location of the leakage.

Figure 3.8: Location of the leakage

The oil that leaked into the ground moved trough the unsaturated zone under the influence of gravity until it reached the groundwater level. The floating layer on top of the groundwater and the contaminated plume moved in the direction of the hydraulic gradient. The hydraulic gradient slopes down to the ocean, so the contamination caused by this leaking pipe will have spread in the direction of the ocean. For the investigation of the contamination of the beach area, only the soil samples 1 to

15 are relevant. The measured concentrations of total petroleum hydrocarbons (TPH) on these locations on different depths, measured from the surface, are given in table 3.2.

Table 3.2: Measured concentrations of TPH in the subsoil

location	depth (m)	TPH (mg/l)
P01	1,00	857,40
P01	2,00	10200,00
P01	2,65	42203,00
P02	1,90	279,64
P02	3,00	4212,80
P02	4,00	3758,80
P03	1,00	1082,60
P03	2,00	6721,00
P03	3,00	4611,00
P04	1,00	470,30
P04	2,00	13200,00
P04	3,00	140,33
P05	1,00	110,30
P05	2,00	5270,40
P05	2,60	748,40
P06	1,00	265,70
P06	1,40	101,20
P07	1,00	4557,00
P07	2,00	10961,00

location	depth (m)	TPH (mg/l)
P08	1,00	11354,00
P08	2,00	29762,00
P09	0,60	26827,00
P10	1,00	81,40
P10	2,00	24,20
P11	1,00	13028,00
P11	2,00	3218,90
P11	2,90	4191,00
P12	1,00	9892,00
P12	2,00	5647,50
P13	1,00	57,00
P13	2,00	18,30
P13	2,50	16,10
P14	1,00	134,90
P14	2,00	29,90
P15	1,00	142,90
P15	2,00	231,20

The area of the refinery has an industrial function, so the maximum allowed concentration of TPH in the subsoil is 4000 mg/kg. This applies for point 10, and 12 to 15. For the beach area the maximum allowed concentration of TPH in the subsoil for areas with an agricultural function will be used. So the so the maximum allowed concentration of TPH in the subsoil is 2500 mg/kg for the beach area. So only point 6, 10, 13, 14 and 15 fulfil these requirements

Also some groundwater samples have been taken. The results of these measurements are presented in table 3.3.

Table 3.3: Measured concentrations of TPH in the groundwater

location	TPH (mg/l)
P1	3283,00
P2	14,20
P4	7,38
P5	4,10

The maximum concentration of TPH in the groundwater is 325 µg/l. So neither of the measured samples fulfils this requirement.

With this information the polluted area has to be determined. Figure 3.9 shows the area that certainly is contaminated.

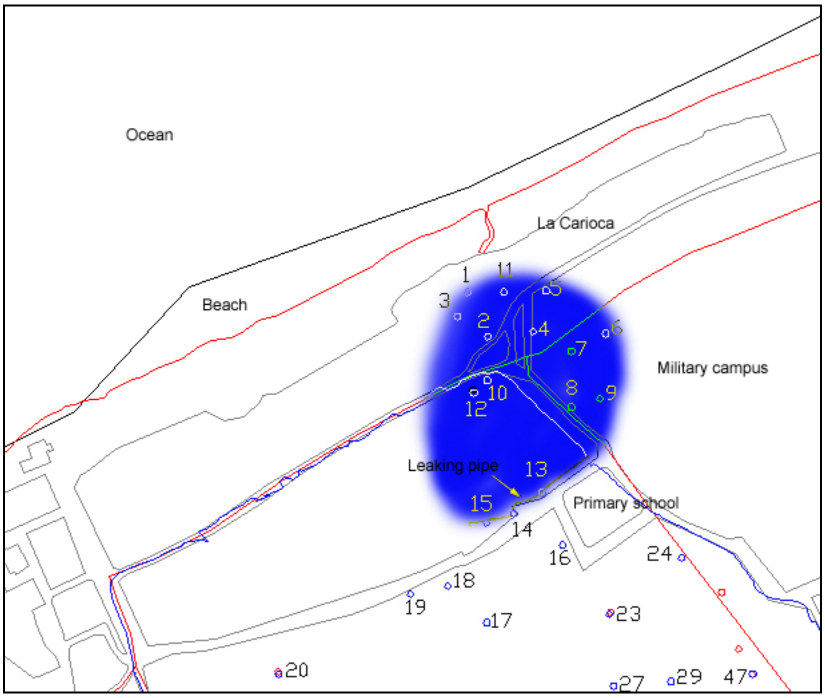


Figure 3.9: Outline of the contaminated area

This area has a maximum width (parallel to the shoreline) of about 160 meter and a maximum length (perpendicular to the shoreline) of about 170 meters. Since there are no more measurements, the polluted area could be larger and have a different shape. Therefore the polluted area will be schematised as a rectangular. To make sure that the entire pollution is captured in this rectangular, the length and the width are multiplied by a factor. This factor is of course arbitrary. The factor is chosen to be 1.5. This results in a rectangular area with a width of 240 meter and a length of 255 meter.

Now the contaminated groundwater plume has to be determined. Since only four measurements of the water quality have been done in the beach area, it is very hard to say anything about the extend of the contaminated groundwater plume. Given that all the measured concentrations of TPH in the water are much higher than the allowed concentration, it can be expected that the groundwater outside the measured locations is also contaminated. For this study, it is assumed that the contaminated ground water plume has the same dimensions as the contaminated area. The distance between the shoreline and the schematised plume is around 35 meters. Figure 3.10 shows the schematisation of the contaminated ground water plume.

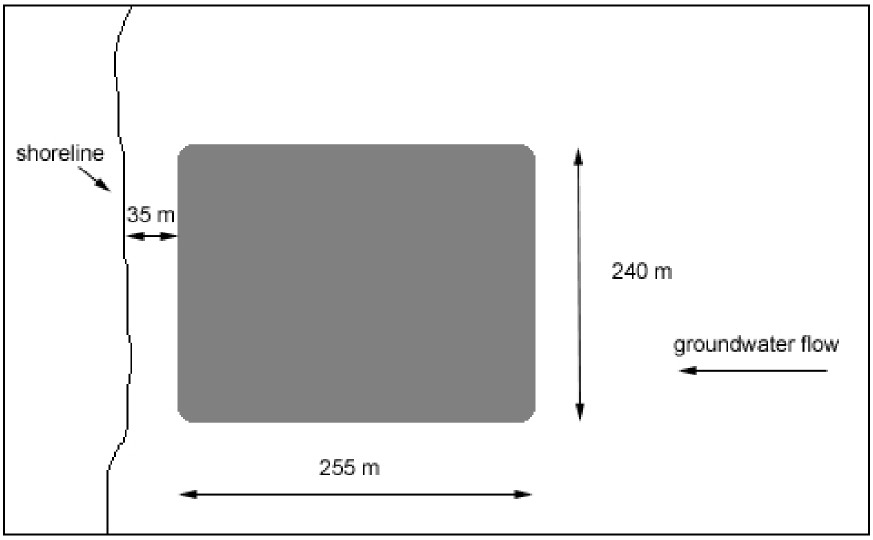


Figure 3.10: Schematisation of the plume

The study area can be schematised as an unconfined aquifer. The average saturated aquifer thickness in the study area is 8 meter. Beneath this aquifer there is a layer of clay (see [3.6]), which is assumed to be impermeable. The hydraulic conductivity of the aquifer itself still has to be determined. The granular distribution of most of the taken soil samples has been determined (see [3.7]). With these distributions the hydraulic conductivity can be calculated with the formula of Hazen, see [3.8]:

$$K = c * d_{10}{}^{2}$$

with:

K = hydraulic conductivity (cm/s)

c = a constant (-) = 150

d_{10} = the 10% percentile of the grain distribution (cm)

The hydraulic conductivity is determined for the soils samples for which the granular distribution has been determined. Table 3.4 shows the results of these calculations. This formula calculates the hydraulic conductivity in cm/s, but for further calculations m/d will be used. The table shows the calculated hydraulic conductivity in cm/s and the converted hydraulic conductivity in m/d.

Table 3.4: Calculated hydraulic conductivity

location	depth (m)	D10 (cm)	K (cm/s)	K (m/d)
P01	1,5	1,50E-02	3,38E-02	2,92E+01
P02	1	1,00E-05	1,50E-08	1,30E-05
P02	2	1,90E-02	5,42E-02	4,68E+01
P02	3	1,00E-04	1,50E-06	1,30E-03
P03	1	1,00E-02	1,50E-02	1,30E+01
P03	2	4,00E-02	2,40E-01	2,07E+02
P03	3	2,00E-02	6,00E-02	5,18E+01
P04	1	1,00E-02	1,50E-02	1,30E+01
P04	2	1,00E-02	1,50E-02	1,30E+01
P05	1	1,00E-05	1,50E-08	1,30E-05
P05	2	1,00E-05	1,50E-08	1,30E-05
P07	1	1,00E-02	1,50E-02	1,30E+01
P07	2	1,00E-02	1,50E-02	1,30E+01
P08	1	1,80E-02	4,86E-02	4,20E+01
P08	2	1,20E-02	2,16E-02	1,87E+01
P09	1	1,50E-04	3,38E-06	2,92E-03
P10	1	1,50E-02	3,38E-02	2,92E+01
P11	1	1,10E-02	1,82E-02	1,57E+01
P11	2	1,00E-05	1,50E-08	1,30E-05
P12	1	1,80E-02	4,86E-02	4,20E+01
P12	2	1,80E-02	4,86E-02	4,20E+01
P13	1	1,40E-04	2,94E-06	2,54E-03
P13	2	1,00E-02	1,50E-02	1,30E+01
P13	3	1,00E-02	1,50E-02	1,30E+01
P15	1	1,00E-05	1,50E-08	1,30E-05
P15	2	1,00E-05	1,50E-08	1,30E-05

These calculations show that the subsoil of the study area is not homogeneous. However for the further calculations homogeneity will be assumed. So an overall hydraulic conductivity has to be determined. It is very hard to compare the calculated results, since the altitude of the measured locations are not exactly known. So the depths at which the grain size distributions are determined are not correlated. And the samples only have been taken for the top three meters of the aquifer.

According to Breddin, see [3.8], subsoils that have a hydraulic conductivity smaller then $7 * 10^{-5}$ cm/s, or $6.05 * 10^{-2}$m/d, are impermeable. So some of the locations in the subsoil are impermeable, while the rest of the samples have according to Breddin, a medium permeability or, P03 at a depth of two meter, even a high permeability.

The only way to come to a overall hydraulic conductivity for the entire aquifer is to take the average value of the calculated permeabilities. The average hydraulic conductivity that will be used for further calculations is 23.7 m/d.

Some other parameters that need to be known are the effective porosity of the soil, the storativity of the soil and the hydraulic gradient. These can not be calculated from the taken samples, so these parameters have to be assumed.

The effective porosity is chosen based on standard values given in literature. Given that the mayor part of the soil samples show large amounts of sand, and the calculated hydraulic conductivity corresponds with this, the effective porosity is chosen based on a sandy soil. According to [3.8] a sandy soil has an effective porosity of 25%.

For the storativity the same method is applied. The assumed storativity is also based on standard values given in literature. According to [3.9] a sandy soil has a storativity of $4*10^{-5}$.

The determination of the hydraulic gradient is more complicated, since there are no standard values for the hydraulic gradient. The slope of the surface of the study area is approximately between 0.03 and 0.07. So the hydraulic gradient will be less then 0.03. Since the precipitation is only 300 mm per year, the hydraulic gradient will probably be much smaller than this 0.03. Based on an expert opinion the hydraulic gradient is chosen to be 0.007.
Besides this regional hydraulic gradient, the tide causes the groundwater to fluctuate. The head fluctuation due to the tide can, according to [3.10], be calculated with the following formula:

$$h = Ae^{-\alpha x}\sin(\omega t - \alpha x)$$
$$with: \alpha = \sqrt{\frac{S\omega}{2KB}}$$

with:

h	= head (m)	x	= location (m)
A	= amplitude of the tide (m)	S	= storativity (-)
ω	= angle velocity (radians/d)	K	= hydraulic conductivity (m/d)
t	= time (d)	B	= aquifer thickness (m)

The tide has a frequency of about 2 times a day, which results in a ω of 12.6 radians/day. The maximum amplitude of the tide in La Libertad is 2.5 m. The storativity is $4*10^{-5}$, the hydraulic conductivity is 23.7 m/d and the aquifer thickness is 8 m. Calculations with these parameters result in figure 3.11 (see also [3.10]):

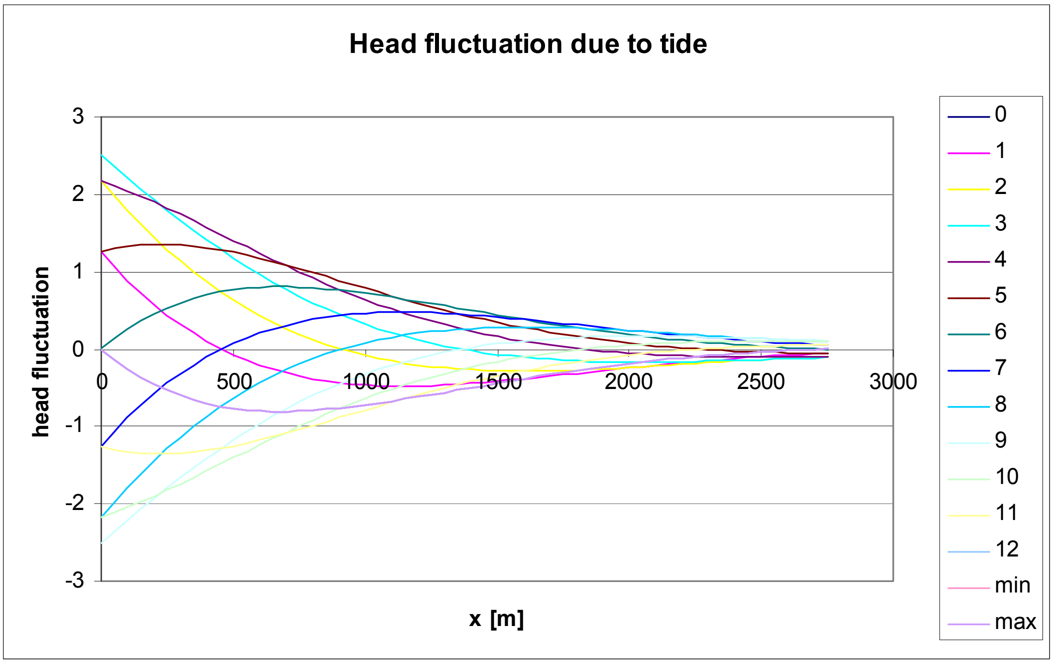


Figure 3.11: Head fluctuation due to tide, each curve represents an hour

For calculating the hydraulic gradient in the study area, the hydraulic gradient due to the tide has to be added to the regional hydraulic head. The hydraulic gradient due to the tide is determined by dividing the difference in head by the distance:

$$i_{tide} = \frac{\Delta h}{\Delta x}$$

with:

i_{tide} = hydraulic gradient due to tide (-)

h = head (m)

x = location (m)

In this case the hydraulic gradient is calculated over the first 500 meter, since this is approximately the study area. The maximum difference in head of these first 500 meter is ± 1.33 m. This results in a maximum hydraulic gradient due to the tide of $\frac{\pm 1.33}{500} = \pm 0.0027$. The maximum (and minimum) total hydraulic gradient can now be calculated with the following formula:

$$i_{\max/\min} = i_{average} \pm i_{tide}$$

with:

$i_{max/min}$ = the maximum/minimum hydraulic gradient (-)

$i_{average}$ = the average/regional hydraulic gradient (-)

i_{tide} = the hydraulic gradient due to tide (-)

This results in a hydraulic gradient with an average of 0.007 and a maximum value of 0.0097 and a minimum value of 0.0043.

3.5.3 Measures

Now the actual situation is known, possible measures can be proposed. First the control and containment measures will be discussed and after that measures to remove the plume by hydraulic means will be discussed.

3.5.3.1 Control and containment

The groundwater flow, and thus the movement of the plume, can be controlled by physical means Physical barriers that are used to prevent ground water flow are slurry walls, grout curtains, sheet piling and geomembranes. These physical barriers will prevent the contaminated plume to spread and will make sure that clean ground water is not contaminated.
Slurry walls, grout curtains and sheet piling all form a wall around the contaminated area (see figure 3.12).

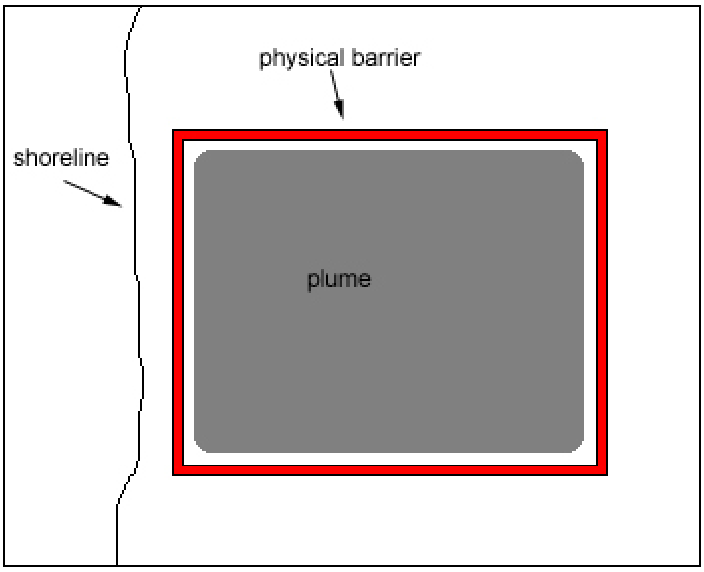


Figure 3.12: Physical barrier surrounding the plume

For constructing slurry walls a narrow trench, 0.5 to 1.5 m wide, has to be excavated, surrounding the contaminated plume. During the excavation, the trench is filled with slurry to keep the trench open. The trench has to be dug until the impermeable clay layer. The trenches can be constructed with a cement-bentonite mixture or with a soil-bentonite mixture. Trenches filled with a cement-bentonite mixture are allowed to set, but trenches filled with a soil-bentonite mixture have to be backfilled with other materials, like soil, cement, concrete and asphaltic emulsions. Depending on the backfill material, the impermeability of the resulting barrier ranges from $8 * 10^{-4}$ to $8 * 10^{-6}$ m/d. According to Breddin, the barrier is impermeable.

Grout curtains are constructed by injecting grout under pressure into the ground. The grout solidifies in the interstitial pore space. By injecting the grout into staggered well points, the curtain is made contiguous. The locations of the injection wells and the injection rate are critical. If the injection rate is too slow, the grout solidifies prematurely. If the injection wells are to far apart the curtain will not be continuous. It is also hard to determine if the grout curtain reaches until the impermeable layer.

Sheet piling involves driving interlocking sections of steel piling into the ground. The sheet piles are connected to each other by slotted or ball-and-socket type connections. The piles are driven into the ground and into the impermeable layer by a pile driver. The connections between the piles are initially not watertight. However, the gaps will eventually be filled with fine grained soil particles. The parts of the piles that remain above the ground are usually cut off.

A geomembrane can be used to seal the walls, so to make sure that they are impermeable.

For the beach area of La Libertad the slurry wall method seems to be the most appropriate physical barrier method. It is easy to construct, since the aquifer thickness is only eight meter. A grout curtain implies the risk of leakage, and sheet piling is not entirely water tight.

To reduce the cost, there can also be chosen for a wall downstream of the plume. In this case the slurry wall would only be constructed between the plume and the ocean (see figure 3.13).

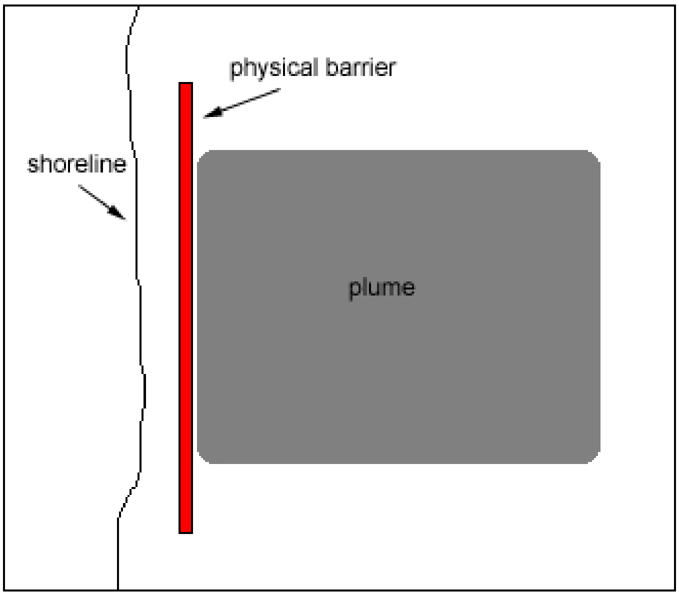


Figure 3.13: A wall downstream of the plume

This prevents the direct contact between the groundwater and the ocean. So it will prevent the plume to directly flow towards the ocean. However, eventually the plume will flow around the barrier. To prevent this, a barrier like this should be used in combination with a pumping system, this will be discussed later in this paragraph.

The mayor disadvantage of these methods is that the contamination is not removed from the site, so only the environmental problems are reduced. The ocean and clean groundwater will not be contaminated any more. The contamination is kept to the already polluted area. But the humane risks stay the same. The air in the ground can still be contaminated, flow out of the ground and be inhaled by people. The smell, that inhabitants complaint about, will not be gone, since the amount of pollution stays the same.

3.5.3.2 (Partial)removal by hydraulic means

The (partial) removal of the contaminated plume using hydraulic means can be done by interceptor systems or by pumping wells.

An interceptor system consists of a trench downstream of the polluted area (see figure 3.14).

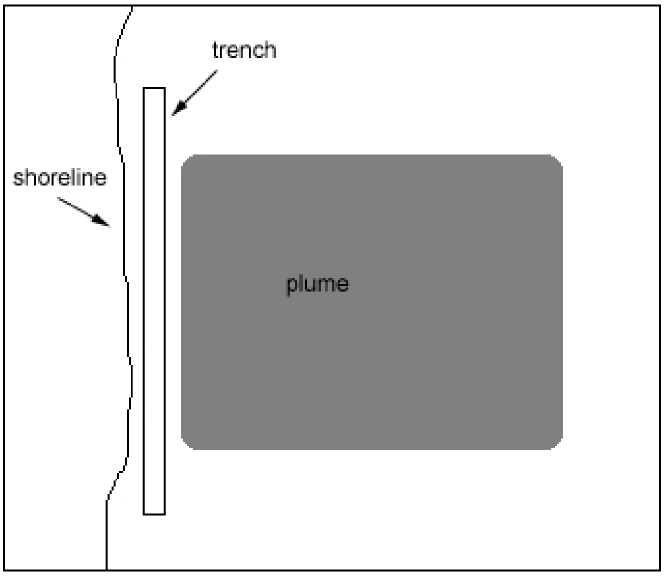
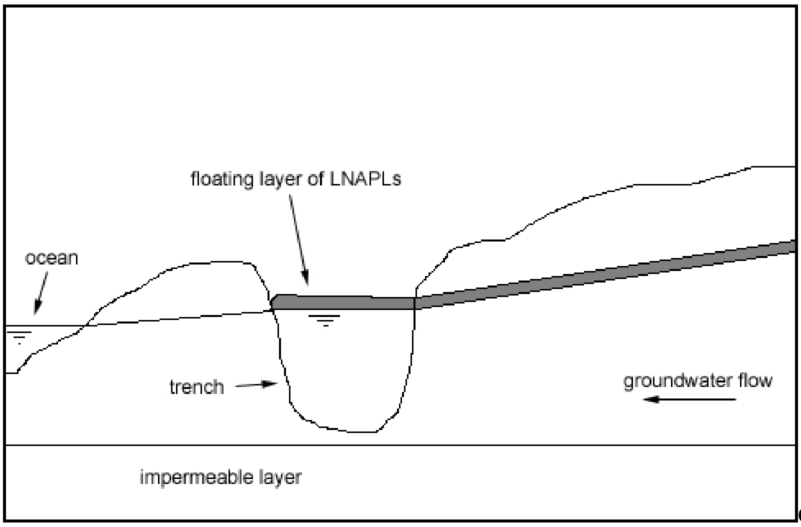


Figure 3.14: A trench downstream of the plume

The plume flows towards the trench. The trench has to be long enough to make sure that the plume will not flow around it. The contaminated groundwater can be pumped out of the trench. Since the LNAPLs float on top of the water table (see figure 3.15), they can be removed separately from the water by using a skimmer pump.



q

Figure 3.15: Floating layer of LNAPLs in the trench

According to Darcy (see [3.11]) the specific discharge can be calculated using the following formula.

$$q = Ki$$

with:

q = specific discharge (m/d)

K = hydraulic conductivity (m/d)

I = hydraulic gradient (-)

With a hydraulic conductivity of 23.7 m/d and a hydraulic gradient of 0.007, this results in a specific discharge of 0.17 m/d The average hydraulic gradient is used, since due to the trench with a fixed water level the ocean will have less influence on the hydraulic gradient. The average velocity at which the water is moving is given by the following formula (see [3.11]):

$$V_{avg} = \frac{q}{n_e}$$

with:

V_{avg} = average velocity (m/d)

q = specific discharge (m/d)

n_e = effective porosity (-)

With an effective porosity of 0.25 the average speed is 0.66 m/d. Since the plume is has a length of 255 meter, it will take about $\frac{250m}{0.66m/d} = 384d$ before the end of the plume reaches the trench.

In this case an interceptor system is probably not a good solution, since downstream of the trench the groundwater flow is still in the direction of the ocean. So, the contamination can still proceed downstream of the trench. An other disadvantage of the fact that the ocean is close by, is that during storms the ocean can get so high that it will overflow the trench. The LNAPLs and the contaminated groundwater that are in the trench when this happens will be taken into the ocean.
Pumping wells are used to extract groundwater from the soil. By doing this the contaminated groundwater plume is extracted from the ground. Since the water table is now sloping in the direction of the pumping well, the floating layer of LNAPLs will flow towards the pumping well. So the floating layer of LNAPLs can also be captured. A single or a two-pump system can be used. A single pump system pumps both the water and the LNAPLs. This is a relatively inexpensive and easy to use system. However, the water and the oil will probably be mixed by the pump, so an API-separator will be needed so separate the water and the LNAPLs again. After this the water will still have to be treated because of the dissolved petroleum hydrocarbons.

A two-pump system extracts the water and the LNAPLs separately. One pump is set just below the water level, so it will only extract water. The other pump is set just at the water level, so it will only extract the LNAPLs. This two-pump system is more expensive then a single pump system but it makes sure that the water and the LNAPLs are not mixed, so an API-separator is not necessary. The extracted groundwater of course still has to be treated.

When using pumping wells it can be necessary to install injection wells as well to make sure that there is enough water to be extracted from the ground. Especially in this case since there is only 300 mm of precipitation a year, and the mayor part of this precipitation falls in the rainy season, which only lasts three months. Sometimes the extracted groundwater can, after treatment, be used for injection again.

When using pumping wells it is important to make sure that the entire contaminated groundwater plume is in the capture zone of the well. If the water table is flat, the capture zone of a well is radially symmetrical. Since there is a regional hydraulic gradient, the area that is influenced by the well is not circular. When water is extracted from the ground, the streamlines bent towards the well. The outer envelope of the streamlines that converge to the well is called the capture zone curve. All the water that is inside this capture zone curve will eventually be extracted by the well. Flow lines outside the capture zone curve may bend towards the well but they will flow past it. Figure 3.16 shows the capture zone of a well located at the origin.

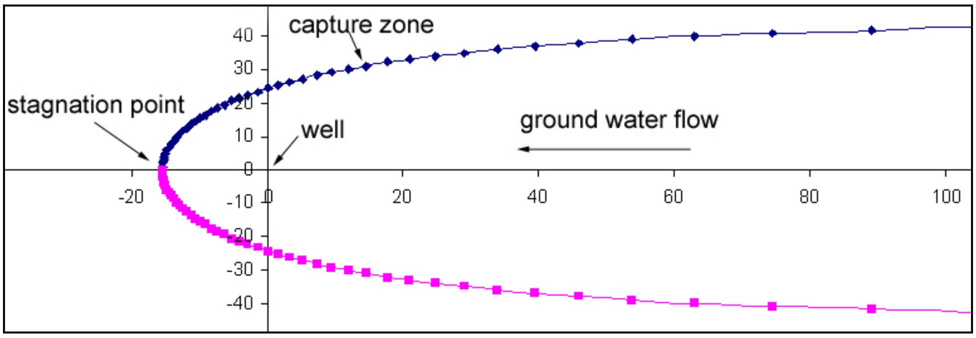


Figure 3.16: Capture zone of a well located at the origin.

The capture zone depends on the soil parameters and the pumping rate. The soil parameters are as given above, so only the pumping rate can be changed. If the capture zone is not large enough for the plume, the pumping rate can be enlarged, until the maximum pumping rate, or until the maximum allowed drawdown is exceeded. If it is not possible to enlarge the pumping rate, more wells will have to be used.

Javendel and Tsang derived (see [3.12]) the following formula that describes the relationship between x and y of the capture zone curve for a single well located at the origin:

$$y = \pm \frac{Q}{2KBi} - \frac{Q}{2\pi KBi} \tan^{-1} \frac{y}{x}$$

with:

B = aquifer thickness (m)

K = hydraulic conductivity (m/d)

i = hydraulic gradient (-)

Q = pumping rate (m^3/d)

This formula can also (see figure 3.17) be written like:

$$y = \frac{Q}{2KBi}\left(1 - \frac{\varphi}{\pi}\right)$$

with: $\tan\varphi = \frac{y}{x}$

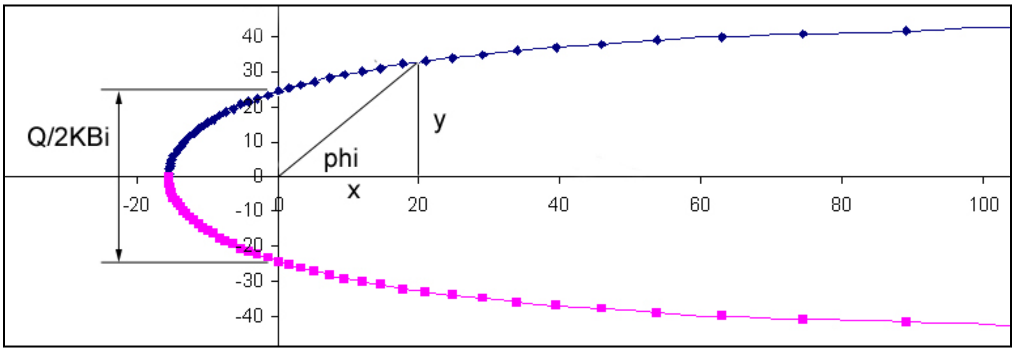


Figure 3.17: Capture zone with φ

With this formula some important measures of the capture zone can be determined.

If $x \to \infty$, $\varphi = 0$, then $y = \frac{Q}{2KBi}$, so the maximum total width of the capture zone is $\frac{Q}{KBi}$.

For $x = 0$, $\varphi = \frac{\pi}{2}$, then $y = \frac{Q}{4KBi}$, so the width of the capture zone along the y-axis is $\frac{Q}{2KBi}$, see figure 3.17.

Javendel and Tsang (see [3.12]) also derived a formula that describes the relationship between x and y of the capture zone curve for n optimally placed wells arranged symmetrically along the y-axis, all pumping at the same pumping rate:

$$y = \frac{Q}{2KBi}\left(n - \frac{1}{\pi}\sum_{i=1}^{n}\varphi_i\right)$$

where φ is the angle between a horizontal line through the *i*th well and a spot on the capture-zone curve.

Optimally placed wells means that the wells are places with the maximum distance between that that does not allow any flow to pass between them.

For two wells the optimal distance between them is: $\frac{Q}{\pi KBi}$, the maximum total width of the capture zone is: $\frac{2Q}{KBi}$, and the with of the capture zone along the y-axis is: $\frac{Q}{KBi}$.

For three wells the optimal distance between them is: $\frac{\sqrt[3]{2}Q}{\pi KBi}$, the maximum total width of the capture zone is: $\frac{3Q}{KBi}$, and the width of the capture zone along the y-axis is: $\frac{3Q}{2KBi}$.

The formulas derived by Javendel and Tsang are valid for confined aquifers, but they can also be used for unconfined aquifers if the drawdown is small relative to the total saturated thickness of the aquifer.

The well should be placed as close as possible to the plume; otherwise a lot of water would have to be extracted from the ground before the actual plume will be extracted. Figure 3.18 shows the best position of the well considering the amount of water that has to be extracted.

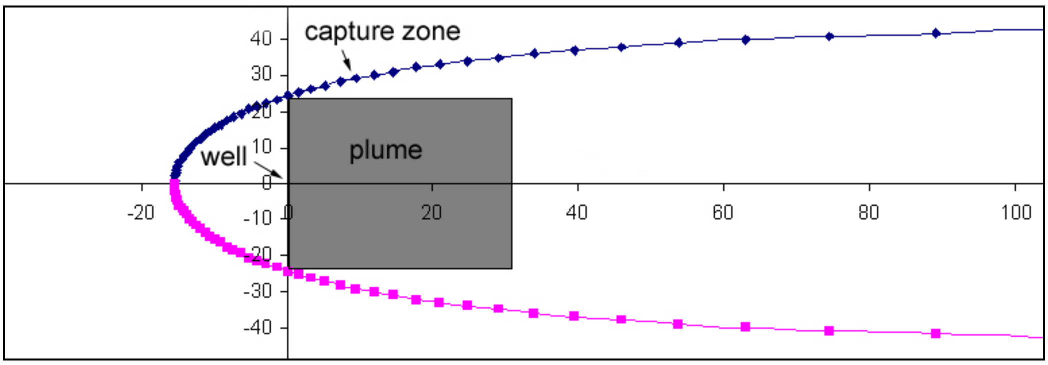


Figure 3.18: Location of the well relative to the plume

Figure 3.18 shows that for the best positioning of the well the width of the capture zone along the y-axis should match the width of the plume. The pumping rate that corresponds with this width along the

y-axis can be calculated with the above given formulas. It should be checked if this pumping rate does not exceed the maximum pumping rate. If so multiple wells should be used.
Furthermore it should be checked if the maximum drawdown is not exceeded. The drawdown can be calculated with the following formula (see [3.13]):

$$s = \frac{2.3Q}{4\pi KB}\log\left(\frac{2.25KBt}{r^2 S}\right)$$

with:

s	= drawdown (m)	t	= pumping period (d)
Q	= pumping rate (m^3/d)	r	= distance from well (m)
K	= hydraulic conductivity (m/d)	S	= storativity (-)
B	= aquifer thickness (m)		

This formula is valid for small values of r and large values of t. Since t is equal to the pumping period, it will be large. And r will not be larger than the distance between the wells.

The drawdown at the location of a well will be estimated with $r = 0{,}2m$. This is about half the diameter of the borehole. If more pumps are used, the drawdown will be calculated by super positioning the draw downs from the different wells. If there are two wells, the maximum drawdown will occur at the locations of the wells. This drawdown can be determined by calculating the drawdown at the first well, so $r = 0{,}2m$ and adding the drawdown caused by the second well, so $r = \frac{Q}{\pi KBi}$. For three wells the maximum drawdown occurs at the location of the middle well. So the drawdown is calculated using $r = 0{,}2m$ for the middle well and using $r = \frac{\sqrt[3]{2}Q}{\pi KBi}$ for the other two wells.

The pumping period is estimated by calculating the time that is needed to extract the volume of the plume. The volume of the plume is calculated with the following formula:

$$V_p = L_p W_p B n_e$$

with:

V_p	= volume of the plume (m^3)	B	= aquifer thickness (m)
L_p	= length of the plume (m)	n_e	= effective porosity (-)
W_p	= width of the plume (m)		

Now the pumping period can be calculated with the following formula:

$$t = a\frac{V_p}{Q}$$

with:

t = pumping period (d)

a = excess water delay factor (-) = 1.5

V_p = volume of the plume (m^3)

Q = pumping rate (m^3/d)

The factor a is used to compensate for the fact that not only the water from the plume is extracted from the ground but also the not contaminated groundwater that is within the capture zone. This pumping period is only an indication of the time needed for the total remediation of the site. When the polluted water is extracted, clean groundwater will get in contact with the contaminated soil and will, according to the formula of Raoult be contaminated too. So this water will have to be extracted too. The pumping period is only an indication, so different alternatives can be compared with each other.

<u>Calculations</u>

The following parameters are used to calculate the capture zones:

The soil parameters that will be used are shown in table 3.5:

Table 3.5: Soil parameters

hydraulic conductivity	K	23,7	m/d
aquifer thickness	B	8	m
hydraulic gradient	i	0,0097	-
storativity	S	$4*10^{-5}$	-
effective porosity	n_e	0,25	-

For the hydraulic gradient the maximum value is used in stead of the average value since with a higher hydraulic gradient, the necessary pumping rate is larger, and, as a result of this higher pumping rate, the drawdown will be larger.

The dimensions of the plume that will be used are shown in table 3.6:

Table 3.6: Dimensions of the plume

width of the plume	W_p	240	m
length of the plume	L_p	255	m
volume of the plume	V_p	122400	m^3

The restrictions are shown in table 3.7:

Table 3.7: Restrictions

maximum pumping rate	Q_{max}	2000	m^3/d
maximum drawdown	s_{max}	8	m

The drawdown can not be larger then 8 meter since this is the aquifer depth.

The necessary pumping rates are calculated for a single well, two wells and three wells. The wells are located on the y-axis with optimal separation; these optimal separations are also calculated. Furthermore, the estimated pumping period and the drawdown are determined. The results of these calculations are shown in table 3.8:

Table 3.8: Results of calculations

	1 well		2 wells		3 wells	
pumping rate	882,78	m^3/d	441,39	m^3/d	294,26	m^3/d
optimal separation	-		76,39	m	64,17	m
pumping period	207,98	d	207,98	d	207,98	d
drawdown	9,16	m	6,96	m	6,31	m

These calculations show that it is not possible to use only one well, since the drawdown is too large, it exceeds the aquifer thickness. Therefore a system with two wells will be used. A system with three wells can also be used, but the only advantage of this would be the reduction of the drawdown with 65 cm. Figure 3.19 shows the locations of the wells.

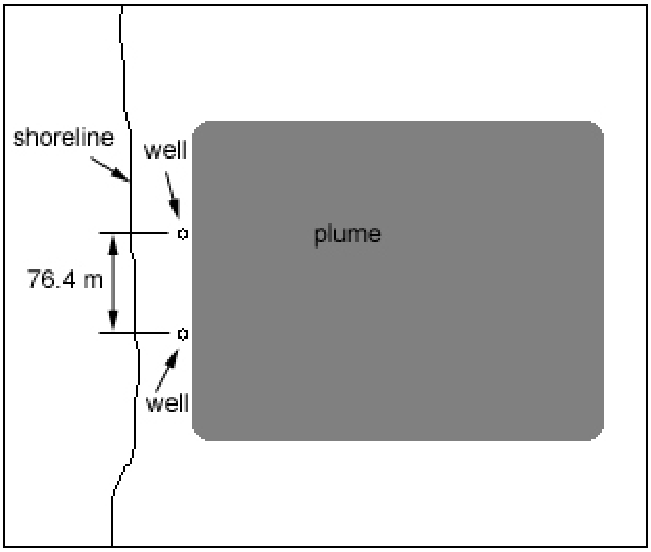


Figure 3.19: Locations of the two wells

The two wells both having a pumping rate of 441.39 m^3/d, are located 35 meter from the shoreline and the distance between them is 76.4 meter. They are located symmetrically towards the plume. The maximum with of the capture zone of the two wells is 480 m. It will take about 208 days to totally extract the contaminated plume and the drawdown will be 6.96 meter. During the pumping period in total 183600 m^3 water is extracted from the ground. Figure 3.20 shows the locations of the wells on the map of the ocean side of the refinery.

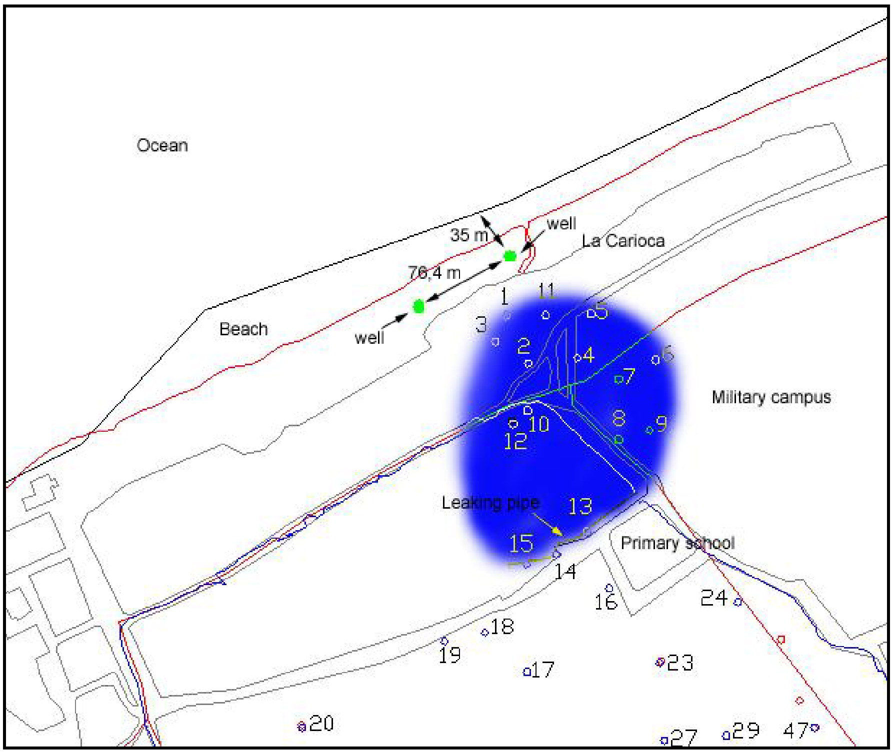


Figure 3.20: Locations of the two wells

The wells are designed for the maximum hydraulic gradient. Since this maximum hydraulic gradient will not occur often, the pumping rate will most of the time be too large, so the capture zone will be too large. But it is better to have a too large pumping rate then having a too wide capture zone. The pumping period however will be larger then calculated since the capture zone is wider then necessary.

If a single pump system is used, about 880 m^3 of water per day will have to be treated. First the water and the LNAPLs have to be separated by an API-separator. After this the water will have to be treated, due to the dissolved petroleum hydrocarbons. The LNAPLs possibly can be used or will have to be disposed. If a two-pump system is used the water and the LNAPLs are extracted separately, so an API-separator is not necessary.

As describes before it is also possible to combine a physical barrier downstream of the plume with a pumping well. Because of the physical barrier there is less influence of the tide on the hydraulic gradient, so in this case the hydraulic gradient that is used for the calculations is chosen to be the average value of 0.007. The other parameters all stay the same. For calculating the capture zones of the wells it is assumed that the physical barrier has no further influence, except the influence of the tide. In reality the barrier will have some influence on the capture zone. The capture zone will be wider, since there is less water to be extracted on the downstream side of the well. Due to this, the width of the capture zone along the y-axis will be larger then calculated. Since it is important that the entire plume is inside the capture zone, it is safe to calculate the capture zone neglecting the influence of the barrier on the capture zone.

Again the necessary pumping rates, optimal separations, pumping periods and draw downs are calculated for one, two and three wells. The results of these calculations are shown in table 3.9:

Table 3.9: Results of calculations

	1 well		2 wells		3 wells	
pumping rate	637,06	m³/d	318,53	m³/d	212,35	m³/d
optimal separation	-		76,39	m	64,17	m
pumping period	288,20	d	288,20	d	288,20	d
drawdown	6,69	m	5,11	m	4,64	m

In this case it will be possible to use only one well, since the drawdown is now 6.69 meter, which is less then the maximum drawdown. The well with a pumping rate of 637.06 m^3/d, will be located 35 meter from the shoreline. The physical barrier will be placed between the well and the shoreline. See figure 3.21 for the locations of the well and the barrier.

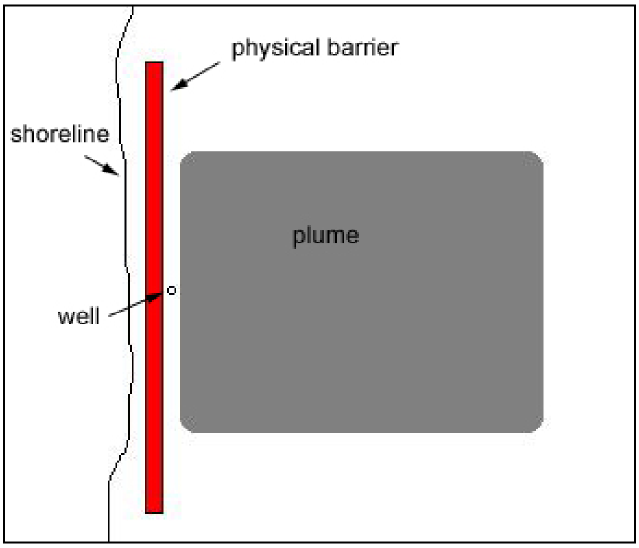


Figure 3.21: Locations of the barrier and the well

It will approximately take 290 days to extract the entire plume of contaminated groundwater and the floating layer of LNAPLs. During the pumping period an amount of about 183600 m^3 is extracted from the ground. The capture zone of the well with a pumping rate of 637.06 m^3/d is shown in figure 3.22; the influence of the wall is neglected in this figure.

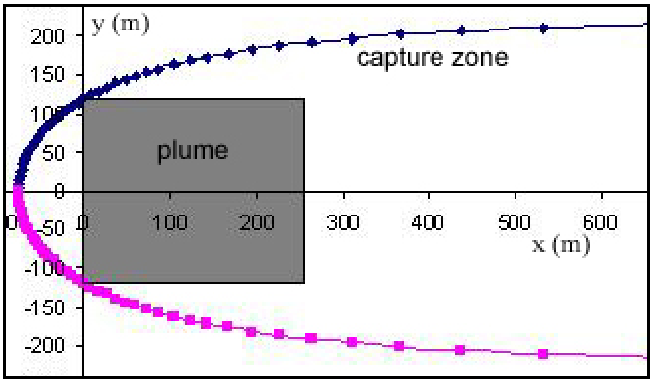


Figure 3.22: Capture zone of the well with a pumping rate of 637,06 m³/d

The maximum width of the capture zone of a well with a pumping rate of 637.06 m^3/d is 480 meter. Figure 3.22 shows that the plume just fits inside the capture zone if the well is located as close as possible to the plume. The influence of the barrier on the capture zone is neglected. Since the capture zone will be wider, due to the barrier, the plume will still be totally in the real capture zone of the well.

If a single pump system is used, about 640 m^3 of water per day will have to be treated. First the water and the LNAPLs have to be separated by an API-separator. After this the water will have to be treated, due to the dissolved petroleum hydrocarbons. The LNAPLs perhaps can be used or will have to be

disposed. If a two-pump system is used the water and the LNAPLs are extracted separately, so an API-separator is not necessary.
Figure 3.23 shows the locations of the well and the physical barrier on the map of the ocean side of the refinery.

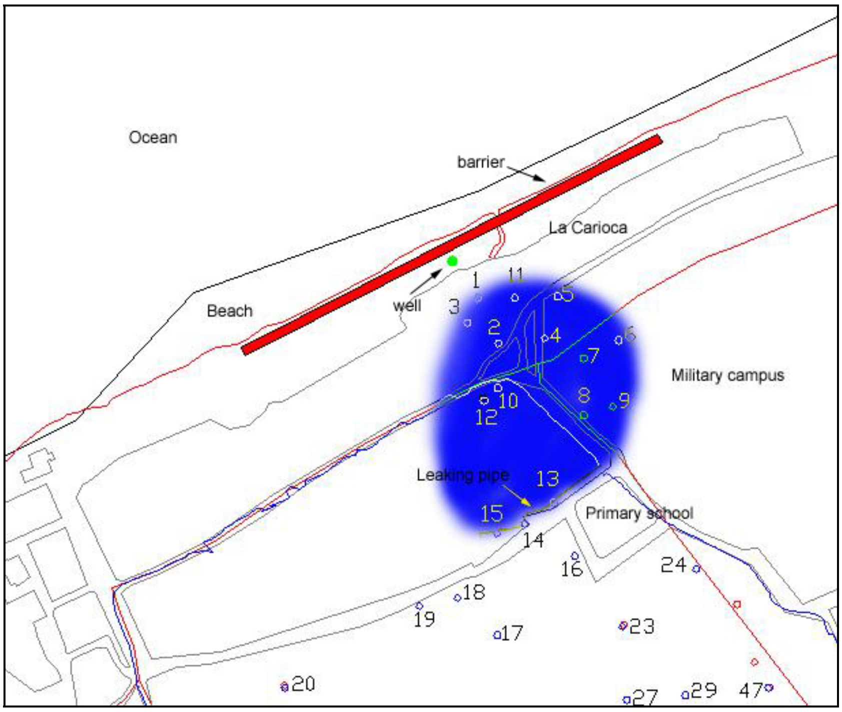


Figure 3.23: Locations of the well and the physical barrier

Besides only extraction wells, also a combination of extraction and injection wells can be used. The injection well will be located upstream of the contaminated plume. The injection well will increase the hydraulic gradient so the plume can be extracted more quickly from the aquifer. An injection well will also make sure that there is more water available in the aquifer. Since there is only 300 mm of precipitation, and the mayor part of this precipitation falls in the rainy season, which only lasts three months, the water table will drop during the dry season.

An injection well will influence the hydraulic gradient. The capture zone of an extraction well is influenced by the regional hydraulic gradient. The difference in head caused by an injection well can be calculated with the following formula (see [3.11]):

$$(h_2 - h_1) = \frac{Q \ln(\frac{r_2}{r_1})}{2\pi KB}$$

with:

r_1	= distance from well (m)	Q	= injection rate (m^3/d)
r_2	= distance from well (m)	K	= hydraulic conductivity (m/d)
h_1	= head at r_1 (m)	B	= aquifer thickness (m)
h_2	= head at r_2 (m)		

In this case $r_1 = 0.2m$ and $r_2 = 500m$. So the difference in hydraulic head is calculated over a distance of 500 meter from the injection well. The hydraulic gradient caused by the injection well can be calculated with this formula:

$$i_{injection} = \frac{(h_2 - h_1)}{(r_2 - r_1)}$$

By superposing this hydraulic gradient on the regional hydraulic gradient the total hydraulic gradient can be determined:

$$i_{total} = i_{average} + i_{injection}$$

with:

i_{total} = the total hydraulic gradient (-)

$i_{average}$ = the average/regional hydraulic gradient (-)

$i_{injection}$ = the hydraulic gradient due to the injection well (-)

In case no physical barrier is used the regional hydraulic gradient is 0,0097 and if the physical barrier is used the regional hydraulic gradient that is used is 0,007.

It is assumed that the rise of the water table due to an injection well can be calculated as a negative extraction well. So the formula for the rise of the water table is as follows (see [3.13]):

$$s^* = \frac{2.3Q^*}{4\pi KB}\log\left(\frac{2.25KBt}{r^2 S}\right)$$

with:

s*	= rise of the water table (m)	t	= pumping period (d)
Q*	= injection rate (m^3/d)	r	= distance from well (m)
K	= hydraulic conductivity (m/d)	S	= storativity (-)
B	= aquifer thickness (m)		

The extraction well is located at the upstream boundary of the contaminated plume, since the plume has a length of 255 meter, the injection well is located 255 meter upstream of the extraction well(s). So in the formula for the rise of the water table due to the injection well $r = 255m$. The total drawdown of the water table can be calculated by subtracting the rise due to the injection well from the drawdown due to the extraction well. This drawdown should again be less then 8 meter.

Now the pumping rate of the extraction well and the pumping rate of the injection well can be determined by iteration. First the pumping rate of the injection well is chosen and the new total hydraulic gradient is calculated with the above formulas. Now the needed pumping rate for the extraction well can be calculated like before. Furthermore the total pumping period and the total drawdown are calculated. Due to the higher hydraulic gradient the pumping rate will be higher than without the injection well, so the drawdown is also larger. This process is repeated until the drawdown is small enough. An other aspect is the pumping period. The less time it takes to remove the contaminated plume, the better.

First the calculations are made for the situation without a physical barrier, so the initial regional hydraulic gradient is 0.0097. Since it was not possible to use only one well without an injection well, it is also not possible to use it with an injection well, since the hydraulic gradient is larger and thus the pumping rate and the drawdown will be larger. For two wells the following results are obtained, see table 3.10, with an injection rate of only 227 m^3/d, which results in a total hydraulic gradient of 0.13. The drawdown is normative in this case.

Table 3.10: Results for two wells with an injection rate of 227 m^3/d

	2 wells	
pumping rate	577,07	m^3/d
optimal separation	76,39	m
pumping period	159,08	d
drawdown	8,00	m

The same calculations have been made for three wells. The drawdown was again normative. The results for three wells are given in table 3.11. The injection rate that followed out of the calculations was 454 m^3/d, which results in a total hydraulic gradient of 0.16.

Table 3.11: Results for three wells with an injection rate of 454 m^3/d

	3 wells	
pumping rate	475,17	m³/d
optimal separation	64,17	m
pumping period	128,80	d
drawdown	8,00	m

These results show that a higher injection rate can be obtained when using three extraction wells and the pumping period is about 30 days shorter. Since one of the purposes of an injection well is to reduce the pumping period, it is better to use three extraction wells. It now takes about 129 days to extract the contaminated plume. Figure 3.24 shows the schematisation of this situation.

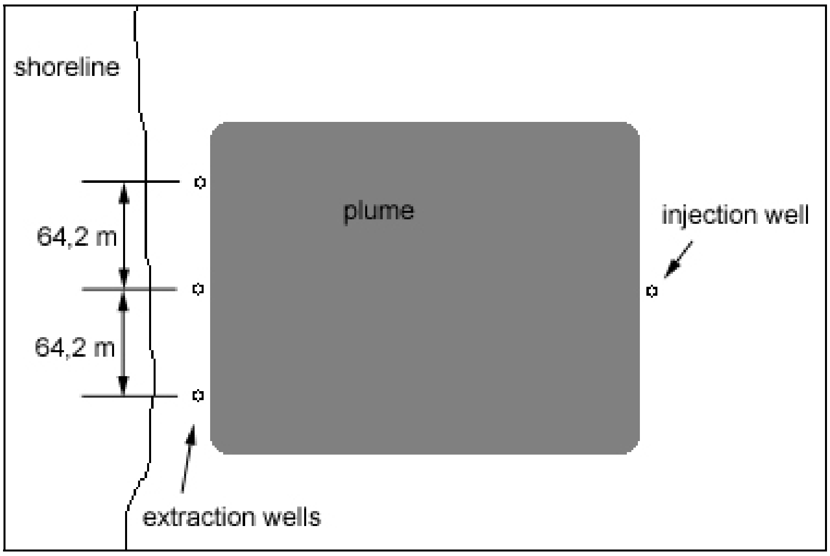


Figure 3.24: Locations of the extraction and injection wells

The extraction wells are located 35 m from the shoreline. Figure 3.25 shows the locations of the extraction and injection wells on the map of the ocean side of the refinery. The total amount of water extracted from the ground, 1425 m^3/d, should be treated. If it is treated it could perhaps, if it is clean enough, be used to be injected in the aquifer again. The amount of water extracted from the ground minus the injected water is about 970 m^3/d. So, during the pumping period an amount of 125130 m^3 water is extracted. So the injection well makes sure that the pollution is removed more quickly and there is less water extracted from the ground compared to the situation without an injection well. The injection rate and thus the extraction rate could be higher if more extraction wells would be used, but those calculations are not in the scope of this study.

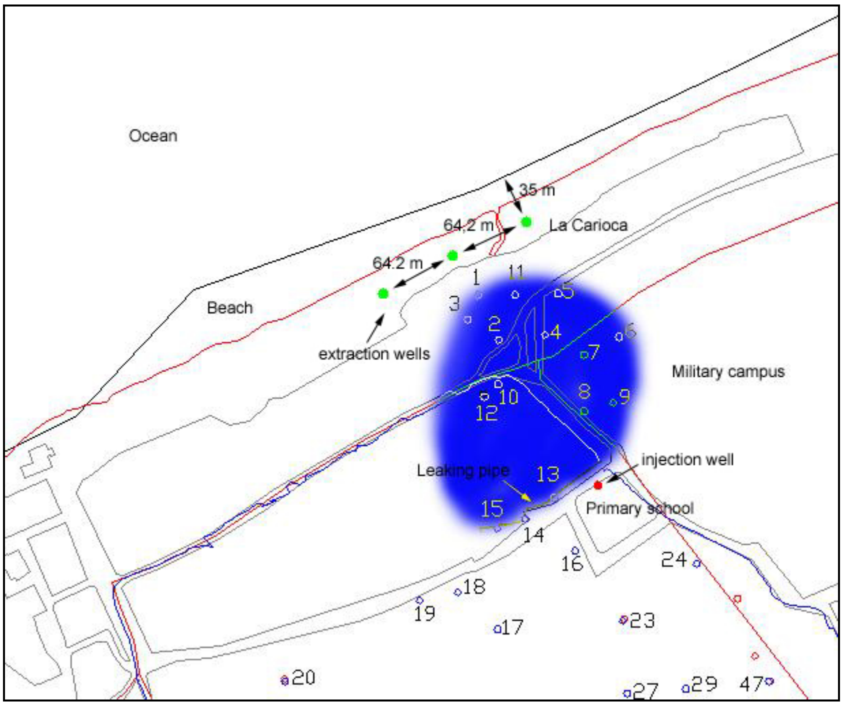


Figure 3.25: Locations of the extraction and injection wells

The same iteration has been done for the situation where a physical barrier is used, so the initial regional hydraulic gradient is 0.007. In this case it is possible to use one, two or three wells. Since the injection well is used to accelerate the process, the solution with three extraction wells is chosen, since it has the shortest pumping period. The results are shown in table 3.12. These results were obtained with an injection rate of 880 m^3/d, which results in a total hydraulic gradient of 0.19.

Table 3.12: *Results for three wells and a physical barrier with an injection rate of 880 m^3/d*

	3 wells	
pumping rate	563,01	m^3/d
optimal separation	64,17	m
pumping period	108,70	d
drawdown	8,00	m

Figure 3.26 shows the schematisation of this situation. The extraction wells are again at 35 meter from the shoreline. The injection well is located upstream of the plume. The extraction wells have a pumping rate of 563 m^3/d, so the total amount of water extracted from the ground minus the injected water is about 809 m^3/d. It takes about 109 days to totally extract the contaminated plume, so the total amount of water extracted during this pumping period is 87940 m^3. The amount of water that has to be treated is about 1689 m^3/d.

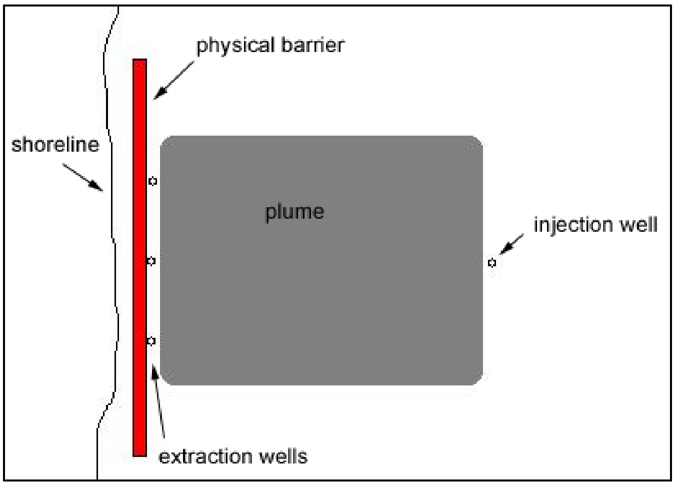


Figure 3.26: Locations of the extraction and injection wells and the physical barrier

Figure 3.27 shows the locations of the extraction and injection wells and the physical barrier on the map of the ocean side of the refinery.

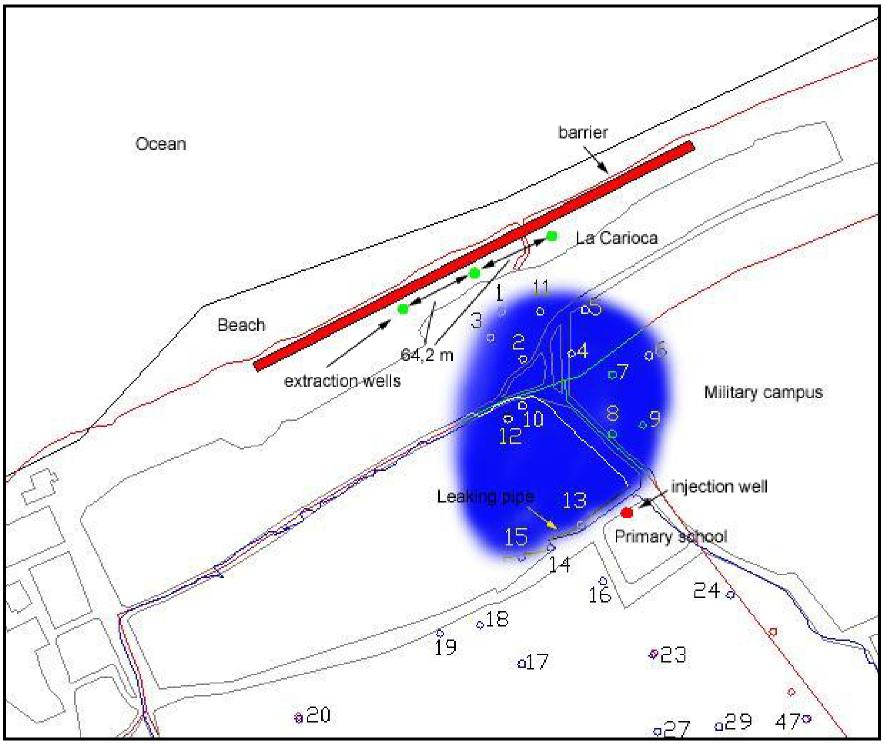


Figure 3.27: Locations of the extraction and injection wells and the physical barrier

The draw downs that are calculated are fairly large compared to saturated aquifer thickness. The formulas of Javendel and Tsang are only valid for small draw downs relative to the saturated aquifer thickness, so the results will not be entirely accurate.

Both the interceptor system method and the pumping well method are based on dissolving components of the oil mixture. The components dissolve into the groundwater according to the formula of Raoult, as described before:

$$M \cdot S_w = C_w$$

with:

M	= mass fraction of the component in the oil mixture (-)
S_w	= solvability in water of the component in its pure form (µg/l)
C_w	= concentration of the component in water (µg/l)

Since only the concentration of total petroleum hydrocarbons is know, the exact components of the oil mixture and their concentrations are not know. So it can not be determined what the resulting concentrations of the oil mixture will be after the treatment. Consequently, it can not be determined if the concentration of total petroleum hydrocarbons in the groundwater and the subsoil eventually will meet the requirements.

The amount of total petroleum hydrocarbons will change. Since the contaminated plume is extracted, clean water will take the place of the contaminated water. This changes the equilibrium situation, so a new equilibrium is reached by components of the oil mixture dissolving in the groundwater. This process proceeds until only the components are left in the subsoil that do not, or very hardly, dissolve in water. This shows that it takes much more time to remove all the contamination from the polluted area then only the time to extract the contaminated plume. So the treatment time will be several times the pumping period.

Both the interception system method and the pumping well method are only able to remove the components of the oil mixture that dissolve in the groundwater and the floating layer on the water level. The components that are adsorbed to the ground are not removed.

To improve the solvability of the components of the oil mixture or to cause the unsolvable components to dissolve in the water, additives can be put into the groundwater. As additives usually surfactants or solvents are used. Surfactants are surface-active agents; they are also used in soap and detergent. Solvents are substances that are capable of dissolving another substance to form a solution.

The additives can be injected into the groundwater via the injection well. This will increase the removal of the contamination, since the components dissolve more easily and components that would not dissolve at all without the additives will dissolve too.

3.6 Multi criteria analysis

Six alternatives to reduce the impact of the pollution with LNAPLs in the contaminated area on the ocean side of the refinery of La Libertad, based on groundwater and groundwater flow have been determined:

- a slurry wall around the plume
- an interceptor system
- a pumping system with two extraction wells
- a slurry wall with an extraction well
- a pumping system with three extraction wells and an injection well
- a slurry wall with three extraction wells and an injection well

Instead of a slurry wall entirely around the plume, also only a wall on the downstream side of the wall could be constructed. This will reduce the cost, but will be less effective in containing the plume.

If an injection well is used, additives can be added to the injected water to improve the solvability of the components of the oil mixture.

These alternatives will be compared based on the following criteria:

- reduction of human and environmental risks
- risk of further spreading of the contamination
- amount of groundwater needed
- treatment time
- costs

<u>Reduction of human and environmental risks</u>

An important criterion is the reduction of human and environmental risks. The more LNAPLs are removed, the more the impact on the human health and the environment is reduced. Since there is not enough information, the amount of removal can not be determined exactly.

If a slurry wall around the plume is used, none of the LNAPLs is removed, so the impact on the human and environmental risks will not decrease.

An interceptor system will remove the floating layer of LNAPLs and the dissolved components in the groundwater, so it will reduce the human and environmental risks.

A pumping system without injection wells will also remove the floating layer of LNAPLS and the dissolved components in the groundwater. A pumping system with an injection well will not only remove the floating layer of LNAPLS and the dissolved components in the groundwater, it will also remove less solvable components of the oil mixture.

Risk of further spreading of the contamination

It is important to make sure that the contamination can not spread any further.

A slurry wall around the plume will, if constructed correctly, always makes sure that the contamination is contained.

The trench of the interceptor system has to be long enough to prevent flow around it. In this case downstream of the trench the groundwater flow is still in the direction of the ocean. So, the contamination can still proceed downstream of the trench. An other disadvantage of the fact that the ocean is close by, is that during storms the ocean can get so high that it will overflow the trench. The LNAPLs and the contaminated groundwater that are in the trench when this happens will be taken into the ocean. So there is a high risk of further spreading of the contamination if an interceptor system is used.

The plume is within the capture zones of the pumping systems. The only risk of further spreading of the contamination happens if a pump of an extraction well stops functioning. Due to groundwater flow the plume can flow past the capture zone of the well. If an injection well is used, the speed of the groundwater flow will be higher, so the risk is even higher.

Amount of groundwater needed

Since there is already water shortage on the Santa Elena Peninsula, the less water there is necessary the better. The amount of water needed to remove the entire plume will be used to compare the alternatives. The slurry wall around the plume of course uses no water at all. The interceptor system only uses natural groundwater flow, so this is no extra discharge compared to the normal situation.

A pumping system with two extraction wells uses 183600 m^3. An extraction well with a slurry wall also uses 183600 m^3.

The pumping system with three extraction wells and an injection well uses 125130 m^3 without a slurry wall, and 87940 m^3 with a slurry wall.

Treatment time

The treatment time is an important criterion. The less time it takes to remediate the area, the better it is. The time to remove the entire plume will be used to compare the different alternatives. The actual treatment time will be much longer. The alternative with the slurry wall of course has no treatment time since the pollution is not removed.

The interceptor system removes the plume by natural groundwater flow. It takes about 384 days before the entire plume has reached the trench.

A pumping system with two extraction wells has a pumping period of about 208 days. An extraction well with a slurry wall has a pumping period of about 288 days.

The pumping system with three extraction wells and an injection well has a pumping period of about 129 days without a slurry wall, and a pumping period of about 109 days with a slurry wall.

<u>Costs</u>

The costs are also an important factor. The costs include the construction costs as well as the exploitation and maintenance costs. The construction of the trench for the interceptor system is probably the cheapest. The construction of slurry walls is probably more expensive then the construction of the wells. However the pumps will have exploitation and maintenance cost. Furthermore the extracted water will have to be treated somewhere, this will also have costs.
So the slurry wall around the plume is the cheapest alternative. The interceptor system is a little bit more expensive. The pumping systems with a slurry wall will be more expensive then the pumping systems without a slurry wall.

Table 3.13 shows a score card with the above given information.

Table 3.13: Score card

	Slurry wall around the plume	Interceptor system	Two extraction wells	Slurry wall with an extraction well	Three extraction wells and an injection well	Slurry wall and three extraction wells and an injection well
reduction of human and environmental risks	0	+	++	++	+++	+++
risk of further spreading of the contamination	+++	0	++	++	+	+
amount of grounwater needed (m^3)	-	-	183600	183600	125130	87940
treatment time (d)	∞	384	208	288	129	109
costs	+++	++	+	0	+	0

The more plusses the better the alternative scores on that criterion. The alternatives that score the best on a criterion are marked with green; the alternatives that score the worst on a criterion are marked with red.

The slurry wall around the plume has the best score on three of the criteria, but it has the worst scores on the reduction of the human and environmental risks and on the treatment time. These two criteria are the most important criteria, so the slurry wall around the plume is not the best option to take. The slurry wall with three extraction wells and an injection well has the best scores on these two criteria. It also has one of the worst scores on the cost. Still this alternative is the best to be used, since the reduction of the environmental risks is the main goal of this study. And the less time it takes to remediate the site the better it is.

So the best way to reduce the impact of the contamination is to construct a slurry wall between the ocean and the contaminated plume. On the downstream side of the contaminated groundwater plume three extraction wells are located. The plume is entirely captured in the combined capture zone of these wells so the contamination can not spread any further. On the upstream side of the contaminated groundwater plume an injection well is located. This injection well makes sure that the plume can be removed more quickly. Additives can be put in the water that is injected through the injection well. This will cause more components of the oil mixture to dissolve in the water so more of the pollution can be removed from the soil.

If the cost of this alternative would be too high, the pumping system with three extraction wells and an injection well could be used.

3.7 Sensitivity analysis

Since al lot of the parameters used for the calculations had to be assumed, a sensitivity analysis is carried out. The soil parameters that had to be assumed are the effective porosity, the storativity and the hydraulic gradient. Furthermore the dimensions of the plume could not be determined exactly.

The influence of a different hydraulic gradient has already been shown by the difference in the results of the calculations for the situation with and without a physical barrier. Table 3.14 shows the results for two wells with a hydraulic gradient of 0.007 and one that is 20% larger, so a hydraulic gradient of 0.0084. The other parameters are the same as used before.

Table 3.14: Results for different values of the hydraulic gradient

	i = 0,007		i = 0,0084	
pumping rate	318,53	m³/d	382,23	m³/d
optimal separation	76,39		76,39	m
pumping period	288,20	d	240,17	d
drawdown	5,11	m	6,07	m

The optimal separation is not influenced by the hydraulic gradient. All the other calculated values are 20% larger or smaller. So the hydraulic gradient has a lot of influence on the calculated design factors. Thus, it is important to make sure that the hydraulic gradient is correct.

Also calculations have been made with two different values for the effective porosity. The assumed value of 0.25 is used and one that is 20% larger, so an effective porosity of 0.30. Table 3.15 shows the results for two wells with these values for the effective porosity. The hydraulic gradient is now 0.007 again .

Table 3.15: Results for different values of the effective porosity

	Ne = 0,25		Ne = 0,30	
pumping rate	318,53	m³/d	318,53	m³/d
optimal separation	76,39		76,39	m
pumping period	288,20	d	345,84	d
drawdown	5,11	m	5,16	m

The effective porosity only influences the pumping period and the drawdown. The pumping period increases in the same order of magnitude as the effective porosity. The drawdown is only minimally influenced by the effective porosity.

Calculations have been made with two different values of the storativity. The assumed value of $4.0 * 10^{-5}$ is used and one that is 20% larger, so a storativity of $4.8 * 10^{-5}$. Table 3.16 shows the results for two wells with these values of the storativity.

Table 3.16: Results for different values of the storativity

	S = 0,00004		S = 0,000048	
pumping rate	318,53	m³/d	318,53	m³/d
optimal separation	76,39		76,39	m
pumping period	288,20	d	288,20	d
drawdown	5,11	m	5,06	m

The difference in the values of the storativity has almost no influence on the calculated values, so it is quite save to assume a value for the storativity.
A change in the value of the effective porosity has only some influence on the pumping period. A change in the value of the hydraulic gradient has a lot of influence on the calculated values. It has a lot of influence on the pumping rate and the drawdown, which are important factors for choosing a design. Thus, it is very important to determine the hydraulic gradient.

The optimal separation was not influenced by any of the changes of the values of the parameters above. The optimal separation is only influenced by the width of the plume. The dimensions of the plume are of course important factors for the calculations, especially the width of the plume since it determines the width of the capture zone. Table 3.17 shows the results of the calculations made for two wells. For the width of the plume the normal value of 240 meter is used and one that is 20% larger, so a width of the plume of 288 meter.

Table 3.17: Results for different values of the width of the plume

	Wp = 240 m		Wp = 288 m	
pumping rate	318,53	m³/d	382,23	m³/d
optimal separation	76,39		91,67	m
pumping period	288,20	d	288,20	d
drawdown	5,11	m	6,07	m

The pumping rate and the optimal separation change proportionally to the width of the plume, the drawdown changes almost proportionally to the width of the plume.

The same kind of calculations have been done for different values of the length of the plume. The normal value of 255 meter is used, and one that is 20% larger, so 306 meter. The results of these calculations are shown in table 3.18.

Table 3.18: Results for different values of the length of the plume

	Lp = 255 m		Lp = 306 m	
pumping rate	318,53	m³/d	318,53	m³/d
optimal separation	76,39		76,39	m
pumping period	288,20	d	345,84	d
drawdown	5,11	m	5,16	m

The pumping period changes proportionally to the length of the plume. The drawdown is also a little bit affected by the change of the length of the plume.
Thus, it is very important to determine the exact dimensions of the contaminated plume.

Some important parameters like the hydraulic gradient and the length of the plume are not exactly known, although they do have a rather large influence on the calculated design factors.
The formulas of Javendel and Tsang are only valid for small draw downs relative to the saturated aquifer thickness. Since the calculated draw downs are fairly large relative to the saturated aquifer thickness, the values of the pumping rate and the optimal separation, calculated with these formulas of Javendel and Tsang, will not be entirely correct.
So the calculated pumping rates, optimal separations, pumping periods and draw downs are more indications of to be expected values than exact values.

3.8 Conclusions

On the ocean side of the refinery of la Libertad petrochemical substances, petroleum hydrocarbons, have been leaking into the subsoil. This was caused by a leaking pipe. This pipe has been located and replaced by a new one. So the source of the contamination is removed.
To reduce the impact of the pollution with LNAPLs in the contaminated area on the ocean side of the refinery of La Libertad different alternatives have been generated. The alternatives are designed to make sure that the risks for the human health and the environment are reduced, and to make sure that the contamination does not spread any further.

In this study only methods are used that involve groundwater or groundwater flow. The best way to reduce the impact of the contamination using these methods, is to construct a slurry wall between the ocean and the contaminated plume. On the downstream side of the contaminated groundwater plume three extraction wells are located. The plume is entirely captured in the combined capture zone of these wells so the contamination can not spread any further. On the upstream side of the contaminated groundwater plume an injection well is placed. This injection well makes sure that the plume can be removed more quickly. Additives can be put in the water that is injected through the injection well. This will cause more components of the oil mixture to dissolve in the water so more of the pollution can be removed from the soil.

Some important parameters like the hydraulic gradient and the length of the plume were not exactly known, although they do have a rather large influence on the calculated design factors.
The formulas of Javendel and Tsang are only valid for small draw downs relative to the saturated aquifer thickness. Since the calculated draw downs are fairly large relative to the saturated aquifer thickness, the values of the pumping rate and the optimal separation, calculated with these formulas of Javendel and Tsang, will not be entirely correct.
So the calculated pumping rates, optimal separations, pumping periods and draw downs are more indications of to be expected values than exact values.

In this report only methods that involve groundwater and groundwater flow are considered. There are a lot of other methods that could be used to remediate the site, like vapour extraction, biological or chemical breakdown, removal and disposal of the contaminated ground or land farming. These methods should be investigated too to determine what the best option is.

3.9 Recommendations

- To get a better idea about the extend of the pollution a roster should be used for taking the soil samples, instead of taken samples where it looks polluted. If a roster is used the dimensions of the plume can be determined more accurately, so the results of the calculations will be more precise;
- To get more accurate results the hydraulic gradient should be determined more precisely. The hydraulic gradient has quite some influence on the results;
- To get more accurate results the effective porosity of the soil should be determined;
- To get more accurate results the storativity of the soil should be determined;
- A water balance should be made of the region to check if there is enough water available for the extraction of the plume. If there is not enough water available for the pumping systems, the slurry wall around the plume or other methods will have to be used.
- Other methods that do not involve groundwater or groundwater flow have to be investigated too and they should be compared with the methods described in this study.
- More specific measurements, like the density of the LNAPL, should be done to be able to determine saturation with the LNAPL.
- Calculations should be made to be able to determine how much LNAPLs are removed from the soil.

3.10 References

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4. Wastewater treatment at the refinery in La Libertad

4.1 Introduction

4.1.1 Background

In order to make La Libertad more appealing to tourists the city wants to diminish sources of pollution near the beaches and in the ocean. One of these sources most likely is the wastewater discharged by the refinery.

In 1993 the final results of a study about the quality of the water discharged by the refinery in La Libertad has been published. The goal of this study was to investigate the impact of the wastewater on the northern coastal zone of the Santa Elena Peninsula (see the map in chapter 1). During the measuring period of one and a half year water quality samples have been taken not only at the discharge points but also at several points in the ocean. This implies that the results of this study were influenced by other activities, such as domestical wastewater discharges, fishery and the transport of oil from oil tankers to the pipelines of the refinery. The main results showed that the discharged water contained a lot of hydrocarbons but the impact of this pollution only stretched out for a few miles. The recommendations of the report mentioned that wastewater treatment facilities at the refinery were not functioning properly, resulting in very poor treatment of the wastewater before it was being discharged.

As mentioned above, the most up-to-date investigations known are twelve years old. Although many things could have changed in that period, recent inspections at the refinery still show oil stains on the water being discharged on the beach. Before jumping to conclusions however, a thorough investigation of the present situation has to be made.

4.1.2 Present situation

The assessment of the present situation starts with visual inspections on the beach near the pipes discharging the refinery's wastewater. As can be seen in Figure 4.1 the water is discharged directly onto the beach where it percolates into the sand or runs off directly into the ocean. Although the sand naturally contains some dark coloured elements, the third picture shows that the water being discharged contains some dirty substances that contaminate the beach. As for the smell of the water, chemicals are clearly present.

Figure 4.1: Wastewater discharge on the beach

Inside the refinery the water is being used in a variety of processes before it is discharged into the ocean. Ocean water enters the refinery in a desalinisation plant where after it will be used as either process water or cooling water. An entire process scheme can be found in paragraph 4.4.1

Two API (American Petroleum Institute) -Separators are installed in order to separate hydrocarbons from the water before the water is being discharged. One, API-Separator 1 (see Figure 4), is being used to treat the process water.

Figure 4.2: API-Separator 1 viewed from different angles

API-Separator 2, the smaller one (see Figure 4.1), treats drainage water coming out of tanks in which refined oils are stored.

Figure 4.1: API-Separator 2 viewed from different angles

The cooling water leaves the refinery untreated. The theoretical background of the API-Separators is discussed in chapter 4.4.2.

The report of 1993 shortly describes the way the API-Separators were operated then. It looks as though a few things are not quite the same as they were then.

After having read the report it appeared as though nobody in particular was responsible for the operation of the API-Separators and that maintenance wasn't a part of the operation. Nowadays an operator is employed and both basins of the large API-Separator are being cleaned once a year, for the smaller API-Separator the cleaning frequency is unknown.

It also appeared that the moving parts of both separators, e.g. the wheels to move the baffles at the inlet of the separator, were stuck. They do function properly at this time.

As will be discussed in 4.4.3 it was clearly visible that the API-Separators weren't designed conform the standards. The malfunctioning of the separators was most visible at the outlets where oil stains were clearly visible as can be seen in Figure 4.2. This oil is floating on the water and it should be possible to remove this oil by a well-designed API-Separator.

Figure 4.2: Oil stains at the outlet of the separators

4.2 Objective of this subproject

In the previous paragraphs it has become clear that the refinery shows not much consideration towards the environment. It is strongly suspected that the API-Separators are not functioning in an optimal way. Therefore the water discharged on the beach is much more polluted than it could be. Moreover, the cooling water is not treated at all before being discharged although it is suspected that the water is polluted.

The main goal of this subject is to reduce the pollution of the beach and the ocean near La Libertad caused by the discharge of partially untreated wastewater produced by the oil refinery.

4.3 Requirements

4.3.1 Boundary Conditions

BC1 The quality of the water discharged should meet the standards set by the Ecuadorian Law: Reglamento Ambiental para las Operaciones Hidrocarburiferas (RAOH) en el Ecuador.

BC2 Any measure taken should not interfere with the refinery processes.

BC3 All necessary measuring methods should be known and available in Ecuador.

BC4 The design of the baffles has to be based on demands found in City of Tacoma Surface Water Management Manual - Volume V Runoff Treatment BMPs, chapter 11.

4.3.2 Constraints

Co1 The API-Separators should meet de design criteria described in API Publication 421 -*Design and Operation of Oil-Water Separators.*

Co2 Every offered solution should be possible to implement in Ecuador.

Co3 The solution should be sustainable.

Co4 The solution should be easy to use.

Co5 The solution should be easy to maintain.

Co6 Cost should be taken into account when making a design or choosing the final solution.

4.3.3 Assumptions

As1 Quantity measurements of in- and outflows of the API-Separators (Q) are not available. Because it was not possible to measure the water flow manually these discharges have to be assumed. The assumptions are based on the amount of water flowing through the cooling water channel and on data found in a report called Diagnóstico y Plan de Manejo Ambiental de las Refinerías La Libertad y Cautivo. This report was published in 2004. The flow rates measured at that time can be found in Table 4.1.

Table 4.1: Flow rates at the refinery in 2004

	Cooling water near API-Separator 1	**Effluent of API-Separator 1**	**Effluent of API-Separator 2**
Flow rate [m3/s]	0,066	0,033	0,011

At the investigation of the refinery for this report, a flow rate of 0,13 m^3/s was measured and calculated for the cooling water, this will be discussed in paragraph 4.5 Measurements. Assuming that the amount of water being used has increased due to rise of production and again assuming that the ratio between the flow rates of cooling water, and the effluent flows from API-Separator 1 and 2 stays the same, an assumption can be made about the influent flow rate of the two API-Separators (see Table 4.2).

Table 4.2: Flow rates at the refinery in 2005

	Cooling water near API-Separator 1	Influent of API-Separator 1	Influent of API-Separator 2
Flow rate [m3/s]	0,13	0,07	0,02

The values found for the influent flow rates of API-Separators 1 and 2 look plausible compared to the water flows present at the site.

As2 The sizes of the holes in the baffles in API-Separator 1 and 2 are assumed to be as indicated in the pictures below.

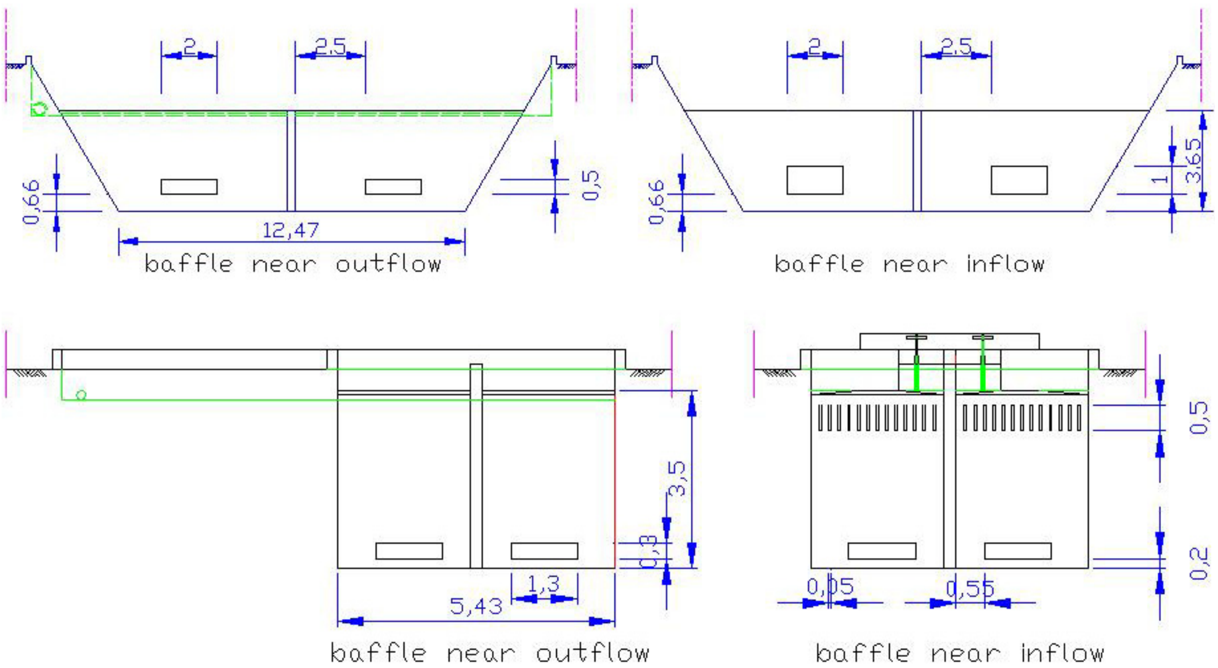

Figure 4.3: Assumed sizes of the baffles in API-Separator 1 (above) and 2 (below).

As3 It is assumed that no other water flows are present in the water discharged onto the beach than the cooling water and the effluent of the two API-Separators.

As4 The exact composition of the oil entering the API-Separators is unknown. Therefore the density of the substance (ρ_o) has to be assumed.

The density of petroleum (the more technical word for crude oil) generally is between 750 and 1000 kg/m^3.[1] This range is divided in two at 860 kg/m^3. Oil mixtures with higher densities are considered heavy petrols, the ones with a lower density are lighter petrols. USFilter[2], one of the companies selling API-Separators, states that 30 or 40 years ago crude oils used in refineries where much lighter then they are now. This is confirmed by one of the operators at the refinery of La Libertad. He mentioned that until 10 or 15 years ago the separators had relatively clean basin sides. This changed and now the basin sides are covered with a sticky and lumpy oil cake when they are cleaned.

This information supports the belief that the type of oil entering the separators is the heavier type (> 860 kg/m^3). After questioning though, operators at the refinery reported a density of 850 kg/m^3. The design density of oil (ρ_o) is assumed to be 900 kg/m^3. This value is taken higher than the found value because lower densities will only have a positive effect the efficiency of the separator. The density is a very important factor for the design of an API-Separator and it is expected that the density of petroleum at the refinery will become even higher in the future.

[1] http://www.wocb.nl/wijzer/wijz7ohc.htm

[2] http://www.usfilter.com/water/Industries/Hydrocarbon_Processing/Solutions_Newsletter/

4.3.4 Program of demands

PoD1 According to Boundary Condition 1; RAOH's Decreto 1215, Anexo 2 states the following about the effluent quality of a discharge location:

a) Effluent at the discharge location[3]					
Parameter	**Expressed in**	**Unit**	**Permitted maximum value** 1)	**Mean annual value** 2)	**Discharge location**
pH-value	pH	---	5,0 < pH < 9,0	5,0 < pH < 9,0	All
Electrical conductivity	EC	µS/cm	<2500	<2000	Land
Total Hydrocarbon Compounds	THC	mg/l	<20	<15	Land
Total Hydrocarbon Compounds	THC	mg/l	<30	<20	Open sea
Chemical Oxygen Demand	COD	mg/l	<120	<80	Land
Chemical Oxygen Demand	COD	mg/l	<350	<300	Open sea
Total Suspended Solids	TSS	mg/l	<1700	<1500	All
Barium	Ba	mg/l	<5	<3	All
Chromium (total)	Cr	mg/l	<0,5	<0,4	All
Lead	Pb	mg/l	<0,5	<0,4	All
Vanadium	V	mg/l	<1	<0,8	All
Total nitrogen 3)	NH_4-N	mg/l	<20	<15	All
Fenoles 3)	mg/l		<0,15	<0,10	All

1) **Measured at any moment**

2) **Mean value of the measurements done in one year conform the monitoring standards in article 11 of this regulation**

3) **Parameters only demanded for refineries that are part of a environmental monitoring program**

[3] Translated from: RAOH Decreto 1215, Anexo 2, Tabla 4a.: Parámetros, valores máximos referenciales y límites permisibles para el monitoreo ambiental interno rutinario y control ambiental

PoD1a Because the regulations are not clear about the difference between discharging at open sea or on land, we state that discharging on the beach is not discharging at open sea. Therefore the water quality has to meet the standards set for discharging on land.

PoD1b For every parameter there are two different norms, a maximum value and an average yearly value. For the design of a treatment step the design value will be set at the average yearly value. In order to cope with variations during the year, the permitted higher maximum can function as a buffer.

PoD1c The regulation sets demands for a control point in the receiving water body as well, but fails to specify the exact location of this point. These demands are probably set for cases when the receiving water body has a low flow rate compared to the flow rate of the discharged water. In this case the receiving water body is the ocean, which is very large compared to any kind of wastewater flow, therefore the regulations concerning the control point are not taken into account.

PoD2 Constraint 1 states that an API-Separator has to meet the standards set in API Publication 421. These standards are as follows.

Design Criteria for an API-Separator according to API Publication 421 - Design and Operation of Oil Water Separators (first edition, February 1990)[4]. The complete version of the guidelines can be found in appendix 4.4.

General

The following parameters are required for the design of an oil-water separator:

a) Design flow (Q_m), the maximum wastewater flow. The design flow should include allowance for plant expansion and storm water runoff, if applicable.

b) Wastewater temperature. Lower temperatures are used for conservative design.

c) Wastewater specific gravity (S_w).

d) Wastewater absolute (dynamic) viscosity (μ). Note: Kinematic viscosity (v) of a fluid of density (ρ) is $v = \mu / \rho$.

e) Wastewater oil-fraction specific gravity (S_o). Higher values are used for conservative design.

f) Globule size to be removed. The nominal size is 0,15 millimeters (150 micrometers), although other values can be used if indicated by specific data.

[4] Adapted from: Environmental Guidelines for Water Discharges from Petroleum Industry Sites in New Zealand, Ministry for the Environment, December 1998

The design of conventional separators is subject to the following constraints:

a) Horizontal velocity (V_H) through the separator should be less than or equal to 15 mm/s (0,015 m/s) or equal to 15 times the rise rate of the oil globules (V_t), whichever is smaller.

b) Separator water depth (d) should not be less than 1 m, to minimize turbulence caused by oil/sludge flight scrapers and high flows. Additional depth may be necessary for installations equipped with flight scrapers. It is usually not common practice to exceed a water depth of 2,4 m.

c) The ratio of separator depth to separator width (d/B) typically ranges from 0,3 to 0,5 in refinery services.

d) Separator width (B) is typically between 1,8 and 6 m in refinery services.

e) By providing two separator channels, one channel is available for use when it becomes necessary to exclude the other from service for repair or cleaning.

f) The amount of freeboard specified should be based on consideration of the type of cover to be installed and the maximum hydraulic surge used for design.

g) A length-to-width ratio (L/B) of at least 5 is suggested to provide more uniform flow distribution and to minimize the effects of inlet and outlet turbulence on the main separator channel.

Figure 4.4 shows a typical oil-water separator and depicts the design variables listed above.

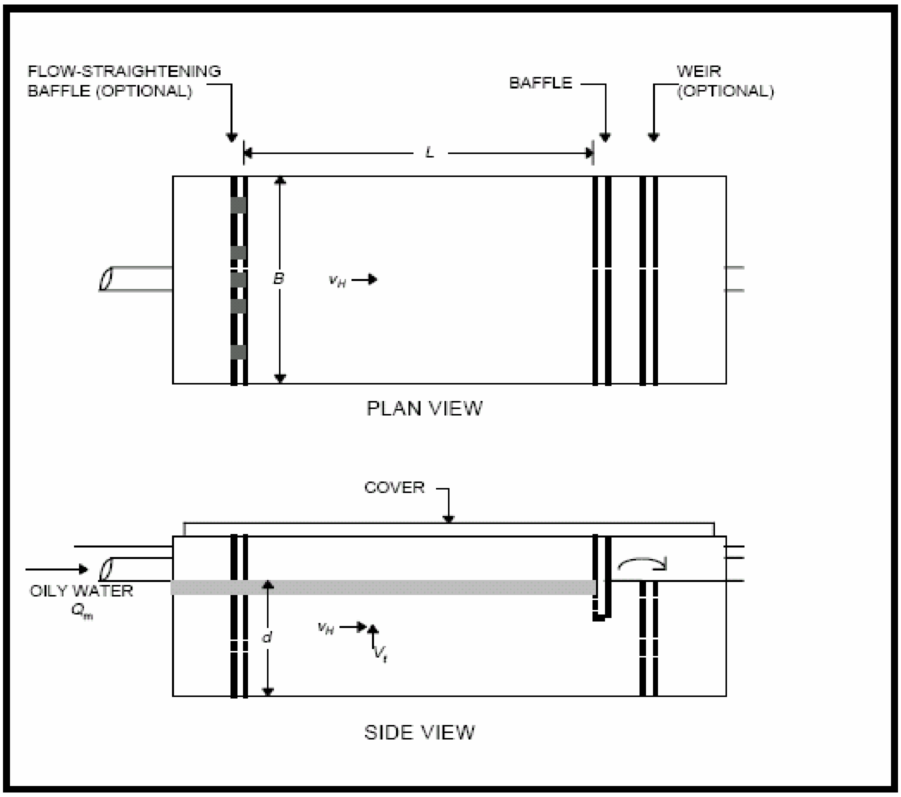

Figure 4.4: Design variables for oil interceptors

PoD3 As mentioned in Boundary Condition 4 the baffles have to be designed conform the criteria found in City of Tacoma Surface Water Management Manual - Volume V Runoff Treatment BMPs, chapter 11.

The criteria in this document state that a baffle has to be at least ½ d high in order to keep the oil layer in the correct basin. It also states that the opening underneath the baffle should be at least 0,3 meters high.

4.4 Important processes in the oil refinery

This chapter contains all basic information about the refinery in La Libertad and the way the wastewater is treated at present.

4.4.1 Water flows in the refinery

To determine the origin of the wastewater, the water flows inside the refinery have to be analysed. At the refinery the number of pipes is enormous, above the ground as well as underneath. Therefore it was not possible to tract down the origin of the wastewater by following the pipes. Since no process schemes were available at the refinery the scheme in Figure 4.5 is constructed after questioning. In appendix 4.1 a more illustrated version is added.

As can be seen in the figure the water is abstracted from the ocean. This salt water is first desalinised in order to prevent the pipes from getting corroded and scaled. After desalinisation the water is used either as process water or cooling water.
The water used for cooling applications is not treated after usage and flows directly to the ocean. The largest API-Separator present, API-Separator 1, treats the water used in the refinery as process water. The water flow leaving the API-Separator is combined with the cooling water and is discharged into the ocean.
The oil abstracted from API-Separator 1 is temporarily stored in three slop tanks that are situated near the separator. Because the oil is stored in the slop tanks for some time, the oil gets more concentrated and can be reused in the refinery. The watery sludge that remains is discharged into a storage basin. At the construction of this storage basin no attention is given to the prevention of the percolation of the oil in the ground. Because this temporary solution is already in use for over 20 years most likely the ground is contaminated. At this moment a design for the treatment of this wastewater is being finished and the devices are ordered.
When API-Separator 1 is cleaned the sludge is scooped out and mixed with wooden chips. This mixture is transported out of the refinery and used as landfill in order to stimulate microorganisms to use the oil as a carbon source.

Besides the process water a second source of wastewater is present at the refinery. The tanks that store the final products of the refinery produce drainage water. A much smaller API-Separator, API-Separator 2, treats this drainage water. The water leaving this separator runs directly to the ocean, the abstracted oil is discharged to API-Separator 1.

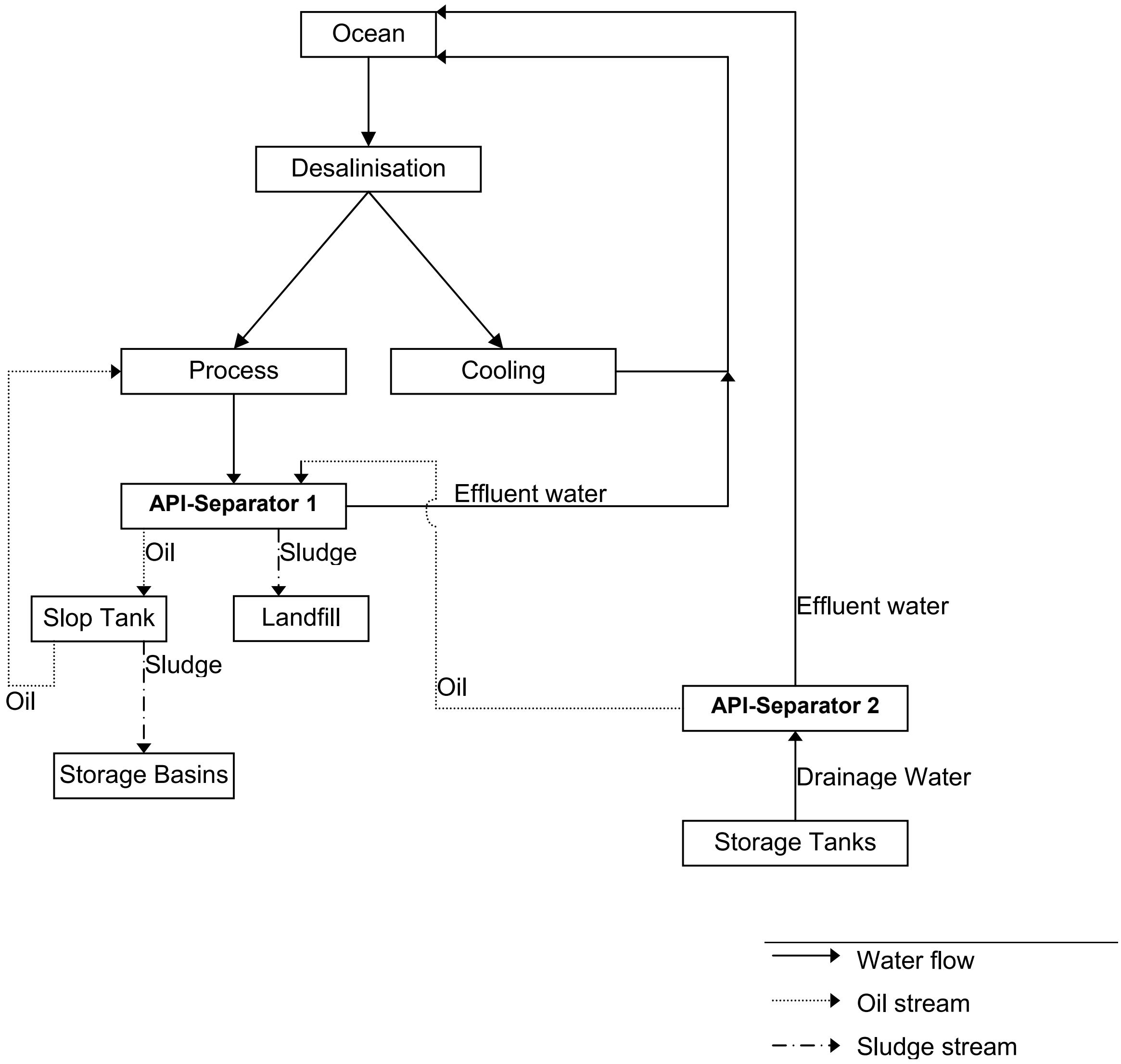

Figure 4.5: Process scheme Water flows in the oil refinery

4.4.2 API-Separator, technical background

API-Separators are widely used as first step in treating industrial wastewater originating from oil refineries. Using gravity as driving force, it is possible to separate substances with different specific weights.

The separator is basically a large basin where conditions are such that a clear separation is established between the water and the oil layer. Three forces are acting on the oil droplets, influencing the vertical velocity V_r (see Figure 4.6), a gravitational force F_g, a sheer force F_s and a floatation force F_f.

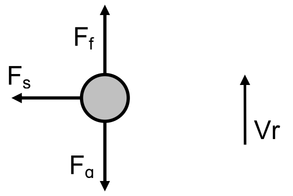

Figure 4.6: Forces on an oil droplet

The floatation force being larger than the gravitational force the oil droplets are slowly moving to the surface. Meanwhile suspended solids will settle at the bottom of the basin because their specific weight is larger than the specific weight of water. Because sheer forces can disturb these processes, turbulence must be prohibited. In order to guarantee the oil to surface short-circuits must be prohibited as well.
The above-described theories are combined in Stokes' law.

Stokes' law:

$$V_r = \frac{g(\rho_w - \rho_0)d_p^2}{18\mu} \qquad (1)$$

Where:
V_r = Vertical velocity of a droplet of hydrocarbon [m/s]
g = Gravity acceleration [m/s^2]
ρ_w = water density [kg/m^3]
ρ_0 = hydrocarbon density [kg/m^3]
d_p = diameter of the particle [m]
μ = dynamic viscosity [Ns/m^2]

Whereby, in this case, the positive direction of the vertical velocity is upward.

Main goal of the design is that the time a droplet of oil needs to reach the surface is shorter than the time needed to reach the end of the basin. To assure the goal is reached, design parameters of an API-Separator have to meet some well-tested and proven standards. These standards are published in API specification 421.

Since an API-Separator uses differences in specific weight to separate different substances to it possible to separate both suspended solids and oil from water. Since the difference in the specific weight between water and the substance that has to be removed is the least for oil and water, this separation is normative.

A schematic cross section and top view of an API-Separator can be found in Figure 4.7. At the end of the basin a baffle construction is placed in order to retain the oil and abstract the cleaned water. A skimmer, a suction device floating parallel to the baffle construction, can remove the oil at the effluent (downstream) end of the basin. A scraper will remove suspended solids at the influent (upstream) end of the basin.

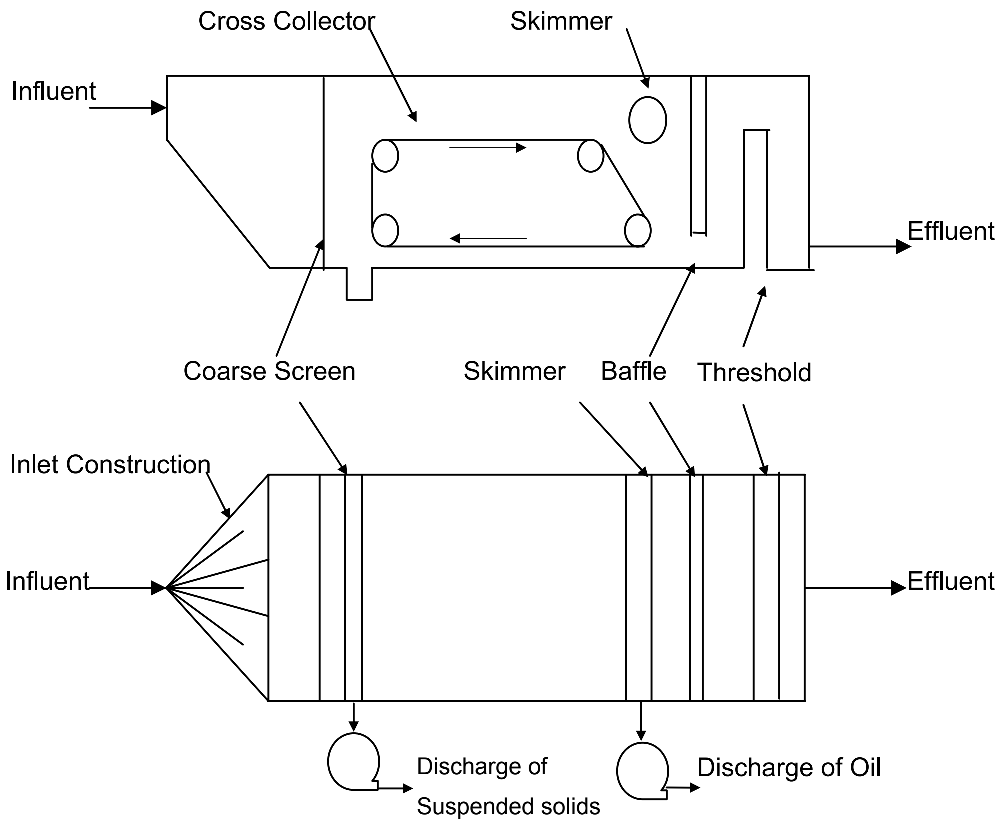

Figure 4.7: Schematic Drawing API-Separator

As described in the program of demands (PoD2) a well-designed API-Separator has to meet specific standards. Table 4.3 below shows a brief overview of these demands.

Table 4.3: Design criteria API-Separator

B	d	d/B	L/B	V_H
m	m	-	-	m/s
1,8 - 6	1 - 2,4	0,3 - 0,5	> 5	< 0,015 or 15 Vt

Removal efficiency

Long time experience with API-Separators has proved that their sensitivity towards changes of influent quality or influent flow rates is very low. The effluent quality will vary a little, but much less than the varieties in influent quality and flow. General efficiencies of 80 - 100% are found at well-designed API-Separators.

4.4.3 Visual analysis of the API-Separators in La Libertad

In order to analyse the API-Separators technical drawings of the construction need to be gathered. Unfortunately these drawings are not available at the refinery, therefore the measures had to be taken at the site. This excursion also provided the possibility to do research about the operation of the API-Separators. One of the streets of API-Separator 1 was being cleaned at the moment so that the generally submerged construction could also be examined. Unfortunately the water samples to analyse the water quality had to be taken at the same excursion. These water samples will be influenced by the cleaning activities. The results of these samples will be discussed in paragraph 4.5, the AutoCAD-drawings of the API-Separators can be found in appendix 4.2 and 4.3.

Analysis of API-Separator 1

Inlet construction

The inlet constructions, see Figure 4.8, consist of an adjustable baffle to control the divergence of the water flow over the two process streets. At the moment of inspection one baffle was closed in order to clean one of the streets, therefore these baffles do seem to work properly. In case a baffle is stuck, it is also possible to close the inlet by placing wooden logs in an iron frame. In case of malfunctioning of both streets of API-Separator 1 it is not possible to close both baffles because there is no possibility to store the wastewater temporarily or to by-pass the API-Separator.


Figure 4.8: Inlet construction API-Separator 1

After the baffle the flow is passing through a funnel shaped gutter, this decreases the flow velocity and divides the flow over the width of the street. In this way turbulence and short-circuiting are prevented. This part of the structure seems to be well designed and is working properly.

At the site it appeared that the basin is not covered by any kind of shield. This lack of cover proved to be a problem during heavy rainfall, which can occur during the rainy season. The meteorological phenomena El Niño can cause even more rainfall; in such an extreme event flotation of the separator has occurred because run off entered the basins. Therefore some special gutters are dug in order to discharge the rainwater directly to the ocean.

Removal of suspended solids

The wastewater stream entering the API-Separator seems quite clear and free of suspended solids. But because the separator is not covered suspended solids can enter the separator by wind. The separator is situated near the beach where wind can be strong. No large buildings are present in the surrounding of the separator; therefore the impact of the wind is not reduced. Because the ground is very dusty and no vegetation is present the soil can be eroded easily. A 30 cm high wall is present around the separator; this wall will prevent some sand from entering the separator.

As described in paragraph 4.4.2, skimmers in combination with a cross collector are the most frequently used devices to remove the oil and the suspended solids. This cross collector has two functions; scrape the suspended solids at the bottom of the tank to a submerged gutter, and moving the oil to the skimmers. At API-Separator 1 no cross collector is installed. The suspended solids are removed once a year by shutting down one street and empting the basin to remove the sludge and oilcake.

Removal of oil

As can be seen in the AutoCAD drawing in appendix 4.2 three skimmers are installed; therefore it is probably not necessary to artificially move the oil to a skimmer.

The skimmers are connected to an oil collection tank, this tank was placed in the middle of the basin in front of the last baffle as can be seen in Figure 4.9. The reason for placing this tank as low as possible,

and therefore inside the basin, is that the oil can move under gravity from the skimmers to the tank. Placing this tank inside the basin has a negative influence on the flow conditions. In order to let the oil surface the flow must be as quiet as possible, obviously this is not the case near the tank. Because the tank is placed in front of the last baffle the flow is disturbed in a crucial part of the structure. After the last baffle no oil removal takes place and all the water is discharged.
At a visual inspection the skimmers seemed to work quite well, the slots were not blocked and the storage tank was filling and emptied every once in a while by a pump. As described in paragraph 4.4.1 the removed oil is pumped to a slop tank.

Figure 4.9: Oil collection tank API-Separator 1

Pumps
At the side of the basin two pumps are installed, one pump for every street. In case of pump failure no additional pumps are available. The pumps are controlled by an on/off mechanism detached to a floating device inside the oil collection tanks.

Baffles
As can be seen in the AutoCAD drawing in appendix 4.2 two baffles are placed. The first baffle is placed behind the inlet construction. The function of this baffle is to retain and remove the oil already floating on the surface. This baffle probably has a positive influence on the short-circuiting as well and prevents the water in the middle compartment to get disturbed by the inflowing water. The submerged holes in the baffle were unfortunately invisible, even in the street being cleaned. After questioning the baffles appear to have a hole just above the bottom of the basin. In the first baffle the hole is 2 meters wide and 1 meter high, in the second baffle near the outlet is less high and measures 2 meters wide and 0,5 meters high. In comparison to the total width of a process street, 9,20 meters, the holes are quite small.
The reason why the gap beneath the baffle doesn't stretch out over the whole width of the basin is unknown. Because all the water has to pass through the baffles the flow velocity will be larger due to a smaller cross-sectional area. This higher flow velocity creates sheer forces, which can negatively influence water-oil separation.

Outlet construction

At the outlet a weir is placed in order to maintain a certain water depth inside the basin. The weir is not adjustable and therefore neither is the water level. A weir could possible function as a structure to determine the flow. As can be seen in Figure 4.10 the weir is broad crested and the water level above the weir is too low to measure. No other flow measuring devices were installed.
In the discharge channel clear oil stains were visible. Near the end of the gutter, where a pipe discharges the water, a clothed hose floated at the surface. It may well be possible that this hose was placed in order to absorb some floating oil.


Figure 4.10: Outlet construction API-Separator 1

At the moment we were about to leave the site, the operator placed a hose in the still functioning street of API-Separator 1. As can be seen at Figure 4.11 the hose discharged wastewater into the basin. Due to this discharge turbulences are introduced in the basin and the oil water separation will be disturbed highly. It would be far better to discharge this wastewater at the inflow construction where the construction is designed to introduce a flow in the basin and not disturb the separation processes. This underlines the suspicion that the operator is not aware of the physical background of the processes inside the separator.


Figure 4.11: Discharge in separation basin

API-Separator 2

Inlet construction

API-Separator 2 is much smaller than API-Separator 1 as can be seen in the AutoCAD drawings in appendix 4.2 and appendix 4.3. API-Separator 2 like API-Separator 1 consists of two similar process streets. As can be seen in Figure 4.12, two adjustable baffles divide the total flow. The baffles seem to be working but no measures are taken in case that both baffles have to be closed. It is not possible to by-pass the separator and no storage facilities are present.

In order to equally divide the flow over the width of the street a flow diversion structure is present. As can be seen on the pictures this structure is not working properly because the iron frame, which is supposed to divide the flow, is not in place. Therefore short-circuits could be present. Because the problem can be solved easily by placing the iron frame at the entrance, this is an indication that there is no well-educated operator present.

Figure 4.12: Inlet construction API-Separator 2

Removal of suspended solids

In API-Separator 2 there are no measures taken to continually remove the suspended solids. The inflowing water seems quite clear but ground erosion is present at the separators surrounding. This separator is situated very close to the beach so the wind forces can be strong. Around this separator a wall is present with a height of 30 centimetres. This wall will prevent some sand from entering the separator. Probably this separator is also emptied every once in a while to remove the suspended solids and the sludge cake but there are no information sources confirming this.

Removal of oil

As can be seen in the AutoCAD drawing in appendix 4.3 and in Figure 4.13 a skimmer is installed near the last baffle. Unfortunately this skimmer is not working.

Figure 4.13: Skimmer API-Separator 2

As can be seen in Figure 4.14 oil collection tank is installed to remove some oil. Near the surface level a hole is made in the tank in order to skim the oil. In the other street no serious measures are taken in order to remove the oil continuously. After questioning it appeared that now and then the oil is gathered by some kind of manually operated gathering device. Because it was not possible to see this operation in action the exact figures and the operation of this device remain unknown.

Figure 4.14: Oil collection tank API-Separator 2

Pumps

At the side of the basin two pumps are installed. The function of one pump is to pump the skimmed oil out of the storage collection tank. A floating device inside the oil collection tank switches this pump on. As soon as this tank is emptied the pump shuts off again. The function of the other pump is not clear; probably it is installed to abstract the oil from the other process street.

Baffles

As can be seen on the AutoCAD drawing in appendix 4.3 two baffles are installed. Because the two process streets were both in use, it was not possible to measure the dimensions of the holes in the baffles. A few people were questioned but the exact dimensions of the holes remain unclear. Some people remembered that the holes don't stretch out over the entire width of the process street. The assumed sizes of the holes, as indicated in the AutoCAD drawing, are based on these memories.
The first baffle near the inflow structure divides the basin in a stilling basin for the inflowing water, and the actual separation basin. Because no oil-removing device is present in the stilling basin the floating oil has to pass the baffle. In order to do so slots are present at the surface level of the baffle. The hole in the first baffle is assumed to be 0,3 meter high and 1,3 meter wide. In comparison to the total with of a street, 2,6 meter, this is quite small. As discussed by the baffles of API-Separator 1 this induces a higher flow velocity and therefore sheer forces. These forces negatively influence the oil-water separation.
The second baffle is placed near the outflow structure; it prevents the oil layer from getting discharged. The hole in this baffle is assumed to have the same size as the hole in the first baffle. Obviously no slots are present.

Outlet construction

At the outlet construction a non-adjustable weir is placed, see Figure 4.15. Because the weir cannot be adjusted the water level in the separator is fixed. It is not possible to determine the flow velocity by measuring the water depth above the weir because this water level is to low. No other flow measuring devices are installed.
A gutter collects the water that is flowing over the weir; in this gutter oil stains are floating on the water.

Figure 4.15: Outlet construction API-Separator 2

4.4.4 Technical analysis of the API-separators in La Libertad

In this paragraph the design of the API-Separators in La Libertad will be compared with design criteria given in paragraph 4.3.4. In this analyse not only the measured sizes will be compared with the criteria (see Table 4.4) but also the reason for setting these criteria will be discussed. The reason for this is to justify the suggested adjustments to the API-Separators as will be discussed in the alternatives presented in paragraph 4.6 and further.

Table 4.4: Dimensions of the API-Separators compared with the criteria

	B	d	L	d/B	L/B	Vt	Vh	n
	m	m	m	-	-	m/s	m/s	-
						Stoke's	Q/(n*Av)	
Design criteria	*1,8 - 6*	*1 - 2,4*	-	*0,3 - 0,5*	*> 5*		*< 0,015 or 15 Vt*	*>1*
API-Separator 1	9,2	3,65	32	0,40	3,48	1,69E-03	1,16E-03	2
API-Separator 2	2,6	3,5	18,3	1,35	7,04	1,45E-03	1,19E-03	2

Depth (d)

To prohibit occurrence of short-circuiting and to allow oil droplets to reach the surface, boundaries are set for the depth of an API-Separator. These boundaries are set on depths between 1 and 2,4 meter. The lower boundary is set in order to reduce turbulence as result of high flow rates. The upper boundary is set because oil droplets need time to reach the surface. The deeper the separator, the more time is needed.

As can be seen in Table 4.4, the depth of API-Separator 1 does not meet the boundaries set. The basins are too deep. This results in a longer time needed to effectively separate oil and water. The actual impact on the process is also depending on the width to depth ratio and the length of the basin. These ratios will be discussed later.

API-Separator 2 also has a depth that falls outside the set range for depths. For this separator this is even a bigger performance risk because the separator is much smaller than API-Separator 1. Depending on the rise rate of the oil droplets and the horizontal velocity of the water a basin that is too deep can result in a very inefficient removal of oil. The droplets do not have enough time to rise and will be washed out of the basin with the water flow.

Width (B)

Typically well-designed API-Separators have a width between 1,8 and 6,0 meter. A wider basin can cause problems with distributing the influent over the total width; this might result in short-circuiting of flows, especially when the basin is very short. A less wide basin also needs a larger depth in order to create a sufficient vertical cross-section that reduces the horizontal velocity. The impact of a larger depth on the efficiency is discussed previously

With a width of 9,2 meter API-Separator 1 is very wide compared to the boundaries set; this can result in short-circuiting when the length is too short. The length to width ratio will be discussed later.

API-Separator 2 has a width of 2,6 meter. This is according to the standards. The depth to width ratio and the length to width ratio will be discussed hereafter.

Depth to width ratio (d/B)

This ratio is standardised because of the influence of the depth on the time a globule needs to surface. The vertical cross-section needs to be sufficiently large to minimize horizontal velocity, but as said the size of the depth has to be restricted. Since the width has its own limitations by reason of occurrence of short-circuiting, a ratio for width and depth is introduced. The ratio is set between 0,3 and 0,5.

Although both the depth and the width of API-Separator 1 are out of range of the standards, the depth to width ratio of 0,4 falls in perfectly. This implies that API-Separator 1 has a nice cross-sectional area compared to the time a globule has to rise. The final remark about the travelling time will be made in the section about the length.

The depth to width ratio of API-Separator 2 does not meet the standard at all. With a ratio of 1.35 the basin is designed too deep compared to the width. The horizontal velocity is reduced because of a fitting cross-sectional area, but the globules have to travel a long way to reach the surface. The length will determine whether they get enough time.

Length to width ratio (L/B)

API-Separators function best when the water to be treated can enter the basin en mass and flow through the basin in the same mass without being disturbed too much. This is called plug-flow. In order to maintain this plug-flow the length to width ratio is set at at least 5. A shorter and therefore wider basin disturbs the plug-flow because water moves sideways to fill the basin completely. This causes turbulences.

The length to width ratio of API-Separator 1 is 3,48. This is much less than the recommended 5. This most likely influences the efficiency of the separator in a negative way. If the flow velocity is on the high side, oil globules do not have enough time to reach the surface. Furthermore a low length to width ratio has a negative influence on the plug-flow system. Turbulences can occur, again negatively influencing the removal efficiency.

The length to width ratio of API-Separator 2 is 7,04. This is quite a bit larger than the stated minimum of 5. Short-circuits will be minimised, giving the oil globules a better but also a longer chance to reach the surface. This is positive influence is necessary, since it was just concluded that the separator actually is to deep compared to the width, which results in longer times needed to reach the surface.

Determination of the minimum necessary length

At a well-designed API-Separator the necessary length of the basin is determined using the formula, this formula is described in appendix 4.4.

$$L = F\left(\frac{V_H}{V_t}\right)d \qquad (2)$$

As can be seen in the formula the ratio of the horizontal and vertical velocity and the depth of the basin determine the length of the basin. The reason for this is that the effluent side of the basin the oil droplets must have reached the surface. The formula is derived from two formulas:

$$V_H \cdot t = L \qquad (3)$$

$$Vt \cdot t = d \qquad (4)$$

Where t is the time that the oil droplets are present in the basin, this time factor can be eliminated by combining the two formulas creating the following formula:

$$L = \left(\frac{V_H}{V_t}\right)d \qquad (5)$$

The factor F in the design formula is introduced because of turbulence and short-circuiting. In formula 5 it is assumed that the oil droplets have a constant vertical velocity, because of turbulence this is not the case. Short-circuiting causes that not all the oil droplets are present in the basin for the same period of

time. In order to be sure that the oil droplets with a shorter retention time reach the surface a safety factor is introduced.
As can be read in appendix 4.4 this safety factor is determined by the ratio V_H/V_t. For the present situation this ratio is presented in Table 4.5.
Because the horizontal velocity is very small compared to the vertical velocity the ratio is very small and no reduction for the turbulence has to be incorporated. The value of F is theoretically only determined by a short-circuiting reduction of safety factor of 1,21. The optimal length of the basin calculated with this formula also is presented in Table 4.5.

Table 4.5: Calculated Length versus actual Length

Separator	V_H/V_t ratio	F-value	d (m)	L (m) Calculated	L (m) Present
API 1	0,86	1,21	3,65	3,8	32,0
API 2	0,82	1,21	3,50	3,5	18,3

As can be seen in the table the length of the API-Separators at the refinery is much longer than the theoretical length necessary for the oil droplets to surface. The one criterion that doesn't meet the standards when using these values is the length to width ratio. This is set at at least 5 in order to create a plug-flow. The length to width ratio at this point is about 1. From this example it can be concluded that in this case the length to width ratio is determining the minimal length necessary.

Because the API-Separators at the refinery are not designed conform the norms, it is wise to introduce an extra safety factor. The reason for this is that the holes under the baffles are not stretching out over the whole length and in the process street is a tank present. These structures disturb the flow and cause extra turbulences. However the difference between calculated length and the present length is so large that can be concluded that the length of the basin is large enough for all the oil droplets to reach the surface.

4.5 Measurements

4.5.1 Measured data description

In order to find a suitable way to treat the wastewater of the refinery, information about the quality and quantity of the discharged water has to be known. Because this data was not readily available measurements had to be done.

As described previously, no data was available about the sizes of the API-Separators. Therefore we measured them ourselves. Besides that, the refinery doesn't have any data available on water quality or quantities. This data had to be gathered manually as well.

In the refinery three important wastewater flows are present; process water, treated by API-Separator 1; drainage water from oil tanks, treated by API-Separator 2; and cooling water, which leaves the refinery untreated. It is important to measure the quantity as well as the quality of these water flows.

As described in paragraph 4.4.3 no flow measuring structures are present in the refinery, this implies that there are no records about the discharges of these three wastewater flows. As will be described in the next paragraph it was possible the measure the flow of the cooling water stream with the velocity-area method. In order to execute this method a straight gutter of reasonable length with a constant slope is necessary. Unfortunately these conditions where only present for the cooling water stream. Because it was not possible to visit the refinery a second time, we were not able to do more measurements at the wastewater flows treated by the API-Separators. Rough assumptions have been made about this.

Water quality analyses have been made for us. Unfortunately only TPH levels have been analysed. Total Suspended Solids levels would have been very interesting because the levels can influence the efficiency of API-Separators. The results of the analyses are presented in paragraph 4.5.3.

4.5.2 Water Quantity

As discussed above, the only flow rate that was possible to measure is the cooling water flow. This flow is passing through a straight gutter near API-Separator 1.

To calculate the flow rate by using the velocity-area method two dimensions have to be determined, the velocity (v) and the cross-sectional area (A). In order to determine the flow rate (Q) the following formula can be applied:

$$Q = v \cdot A \qquad (6)$$

In order to determine the flow velocity a floating device is gently placed in the water stream. A time measurement starts once the device has the same velocity as the water. This measurement ends as

soon as the float mark is passing an imaginary cross-sectional area at a measured distance from the point where the device was when the measurement started. This distance has to be large enough to exclude measuring errors, when the results of different measurements show large difference the distance has to increase to give more accurate results.
It is assumed that the water velocity is the same as the velocity of the floating device. Because the measurement is done over a distance the mean water velocity is calculated over this distance.
Two different floating devices have been used. Ping-Pong balls were considered a little light and easy to influence, therefore also tennis balls have been used. As can be seen in Table 4.6 the time measurements for the two different floating devices do not show any irregularities, therefore these values are accepted and no adjustments of the measuring method were considered necessary.
In ideal cases the cross-sectional area of the channel is constant over the length of the gutter that is used to determine the flow velocity. In order to minimize errors in our case the cross-sectional area is determined at three places, therefore the water depth and the width of the channel are measured at three points. These results are presented in Table 4.6. As can be seen in the table the water depth shows large differences. The reason for this is that the width of the channel is also changing and that the water was flowing quite rapidly, therefore the flow conditions were probably not laminar. In a not laminar flow the water level can change easily and the ruler was hard to read because of the beckoned water. One other reason for the differences in water level can be differences in the slope of the channel. It was not possible to check if the slope was constant.
All measurements have been done on September 30, 2005 except for the quality of the discharge water on the beach. This sample was taken on September 20, 2005.

Table 4.6: Physical properties of the cooling water channel

Location	Width (m)	Depth (m)	Cross-sectional Area (m^2)
begin	0,52	0,14	0,073
middle	0,67	0,10	0,067
end	0,64	0,18	0,115
Average	0,61	0,14	0,085

Table 4.7: Determination of Flow Rate

Measurement	Time	Type floating device	Distance	Velocity	av. Width	av. Depth	Discharge
	sec		m	m/s	m	m	m^3
1	23,8	Pingpong bal	36,35	1,53	0,61	0,14	0,13
2	22,8	Tennisbal	36,35	1,59	0,61	0,14	0,14
3	23,0	Tennisbal	36,35	1,58	0,61	0,14	0,13
4	23,6	Tennisbal	36,35	1,54	0,61	0,14	0,13
5	23,6	Pingpong bal	36,35	1,54	0,61	0,14	0,13
6	23,3	Pingpong bal	36,35	1,56	0,61	0,14	0,13
Average	23,4		36,35	1,56			0,13

After processing the data the flow rate of the cooling water is set to 0,13 m^3 per second.

4.5.3 Water Quality

Since only TPH values have been measured conclusions can only been drawn about the efficiency of the API-Separators and whether the effluent quality meets the standards set in Ecuadorian laws. As no other values are known, it is unknown whether the process water or the cooling water is containing any other impurity that might have an effect on the environment and which could probably be removed with sufficient treatment. This report will only focus on TPH. The results of the measurements are found in Table 4.8.

Because the efficiency of an API-Separator depends on the temperature of the water to be treated, the temperature is measured at the site; these results are presented in Table 4.8.

Table 4.8: TPH-levels

	API-Separator 1		API-Separator 2		Cooling Water	Water discharge at beach
	Influent	Effluent	Influent	Effluent		
TPH (mg/l)	926,1	40,7	479,3	25,6	69,4	28,1
Temperature (ºC)	37	35	29	28	42	

As can be seen in the table the differences in TPH level of the influent and effluent are quite different, the API-Separators are reducing the amount of hydrocarbons present in the water by an average of 95 percent. This is a good accomplishment, especially since the separators are not designed and used in a proper manner.

In the Program of demands it is stated that the amount of hydrocarbons in the effluent flows of the different wastewater sources has to be lower than 20 mg/l. Not one wastewater stream meets this demand.

At the point where the water is discharged on the beach a water sample is taken, this sample however was taken by other people on a different day. As presented in paragraph 4.1.2 Present situation, two pipes are discharging onto the beach. One pipe discharges the effluent of API-Separator 1 and the cooling water, the other the effluent of API-Separator 2. The sample taken probably is a sample of the combined wastewater flows. The measured TPH level at that day was 28,1 mg/l; this is quite low compared to the other samples, but even this value does not meet the standards.

4.6 Alternatives for the treatment of process and drainage water

Since API-Separators are the only wastewater treatment step available at the refinery they have to function properly in the first place. As presented in the technical analyses of the API-Separators in paragraph 4.4.4, the existing separators are not designed conform the norms. In the paragraph about the visual analyses (paragraph 4.4.3) of the separators it is also stated that the separators are not operated in a correct way. The measurements presented and discussed in paragraph 4.5.3 however show that a reasonable amount of TPH is removed. In literature[5] it was found that for an API-Separator an efficiency of 80 -100% should be possible. The present removal efficiency of about 95% is in this range, but still the water quality does not meet the standards set in Ecuadorian laws
Because the processes in the separator are too complicated to be calculated, it is not possible to determine the highest possible removal efficiency based on the dimensions of the existing separators and a rough analysis of the wastewater. Besides that, a lot of variable processes influence the efficiency, for example: inconstancies introduced by operation of the separator and weather influences.
What can be stated is that the outflow of the separators contains oil floating on the water. Because this oil is not dissolved in the water, it should be possible to extract this from the water by means of gravity. With a well-designed and properly operated API-Separator it should be possible to remove this oil as well. For that reason it is expected that with a better-designed separator that is operated more wisely a higher efficiency is feasible. Besides that, years of experience with API-Separators indicate that an API-Separator is the most efficient method to begin treatment of water contaminated with hydrocarbons.
As a result of this the alternatives will only focus on treatment with API-Separators.
The presence of API-Separators in Ecuador proves that the materials to construct a separator are available. Therefore Constraint 2 is met. Constraints 3, 4 and 5 state that the offered solution should be sustainable, uncomplicated to operate and easy maintain. Life cycles of existing API-Separators have shown that these treatment facilities are very sustainable. Operation times of 30 to 40 years are not uncommon. An API-Separator is a very stable treatment step and does not need much attention to function properly compared to other treatment facilities. This does not imply that it should not be looked after.
Two possible alternatives remain, maintaining the existing API-Separators with some adjustments or build a new separator based on the API regulations described in appendix 4.4. Suggestions for adjustments to the existing separator will be discussed in paragraph 4.6.1. The design of a new separator is presented in paragraph 4.6.2. After implementing one of these options a measurement program has to be set up in order to check if the effluent quality is meeting the standards. Suggestions for a possible program will be

[5] http://www.northshorecity.govt.nz/ISDM/4/App4D.htm#Figure2

discussed in paragraph 4.9.3. If the water quality does not meet the standards, additional treatment steps should be implemented, this is beyond the scope of this project and will not be discussed further.
The one pollution source that hasn't been discussed yet is the cooling water. Measurements show that the water contains more TPH than is allowed in the regulations. The cooling water will be discussed in paragraph 4.7

4.6.1 Maintaining the existing API-Separators

The alternative with the lowest impact for the refinery with respect to costs is maintaining the existing API-Separators. In order to have them functioning more effectively, some adjustments have to be made.
As mentioned before an API-Separator functions best when flows in the basins are disturbed the least. Disturbances can be caused by a structure that diverts flow directions; most likely resulting in turbulences and probably in short-circuits. Both the design of the baffles and the oil abstraction devices force the flows to change directions. Therefore these are the most important parts of the separator that have to be adjusted.

Oil abstraction system

The adjustments made for the oil abstraction devices are clear. The tanks have to be removed from the basins, giving the direction of the flow a chance to stay straight. There are several alternatives for the tanks, all based on the skimmer system. The skimmers installed at API-Separator 1 do function correctly at this moment, but they are connected to the tanks in the basin. This connection is based on gravity transportation; no pumps are installed.
When the tanks are removed of the basin a new way of removing the oil has to be implemented. This can be done either by installing a pump or by means of gravity by placing the oil collection tanks in the ground. If the tanks are places low enough no additional pump is needed.
At API-Separator 2 there is also an oil collection tank present in the basin, besides that the skimmers do not function anymore. One way to solve this problem is to remove the tank placed in one of the basins and install new skimmers. The better option has a more rough approach. Since the flow rate to API-Separator 2 is not that large and investigations show that API-Separator 1 is highly over dimensioned (also see paragraph 4.4.4 about the technical analyses of the API-Separators) there is a possibility to shut down API-Separator 2 and divert the water to API-Separator 1. The advantages are that there will remain only one separator that has to be maintained and no new purchases have to be done. In the following part it will be checked whether this option really is feasible.

Shutting down API-Separator 2

The flow rates of the water entering API-Separator 1 and API-Separator 2 are 0,07 and 0,02 m^3/s respectively. Combined this will result in a flow rate of 0,09 m^3/s. One of the demands in the API regulations is that when designing an API-Separator future expansions of processes should be taken into account. This means that the separator should be able to manage higher flow rates in the future. Therefore the flow rate to be used to check if it is possible to shut down API-Separator 2 will be set at 1,5 times the flow rate found at the refinery. The flow rate used for the calculation will be 1,5 * 0,09 = 0,135 m^3/s. The required design dimensions are given in Table 4.9.

Table 4.9: Dimensions for combining the flows of API-Separator 1 and 2

	Q	**B**	**d**	**L**	**d/B**	**L/B**	**Vt**	**15*Vt**	**Vh**	**n**
	m^3/s	m	m	m	-	-	m/s	m/s	m/s	-
							Stoke's		Q/(n*Av)	
Design criteria	-	*1,8 - 6*	*1 - 2,4*	-	*0,3 - 0,5*	*> 5*	-	-	*< 0,015 or 15* Vt*	*>1*
API 1+ 2 combined in 1	0,135	9,2	3,65	32	0,40	3,48	0,00130	0,01944	0,00240	2

The temperature used in these calculations is 23 ˚C because it can be cooler in Ecuador than it was on the day the water samples were taken. This influences the temperature of the water flows. A lower temperature means a higher viscosity and therefore a lower V_t. Lower temperatures result in more conservative designs.

The first criteria to check are the horizontal and vertical velocity (V_H and V_t). The criterion set states that V_H should be lower than 0,015 m/s or 15* V_t whichever is smaller. 15*V_t is 0,019 m/s. This is larger than the horizontal velocity is allowed to be, as a result the value of maximum 0,015 m/s will be normative. The V_H found with Q = 0,135 m^3/s is 0,0024 m/s. This is much lower than the limit, it is not likely problems will arise caused due to high flow velocities.

The other check that has to be made is about the length of the basin. The length of the basin has to be long enough for the oil droplets to reach the surface.

The minimum length necessary can be found by using formula (15) found in appendix 4.4.

The value used for F is the smallest one available in the list, because V_H/V_t = 1,8, which is smaller than the lowest value in the graph, so F = 1,28.

This results in a minimum length necessary of 8,6 meters. The length of API-Separator 1 is 32 meters, so the length of the separator is long enough.

The one thing that cannot be changed though, is that the existing API-Separator does not meet the criteria set for width, depth and length-to-width ratio. Nonetheless the removal efficiency is quite high, so combined with the suggested alterations to the separator it should be able to achieve a better effluent quality.

Baffles

The more difficult parts to adjust are the baffles. The sole purpose of the baffles is containing the oil in one part of the separator while the water can move to the next compartment through the gap under the baffle. No matter how the baffle is designed, it will always cause the flow to divert its direction downward. Unfortunately the baffles in the API-Separators at the refinery are not as wide as the basin. These holes divert the flow in two directions, this can cause turbulences, as is shown in figure 4.18.

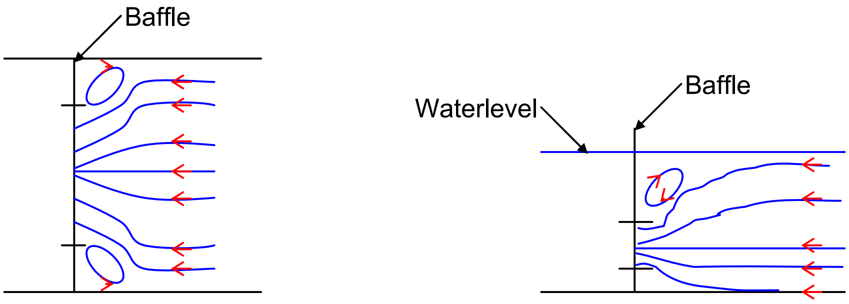

Figure 4.16: Topview (left) and sideview (right) of water flows near the gap in the baffle

This is not so much a problem for the oil droplets that are rising, investigations have shown that there is plenty of time for the droplets to ascend, see paragraph 4.4.4. Nevertheless it can cause problems because the oil layer on top of the water can be mixed up again.

The more efficient way to maintain the oil and let the water pass the barrier is making a baffle that sticks deep enough to retain the oil but that also leaves enough spaces for the water to flow through without the flows being diverted too much or the velocity being accelerated too much. A baffle with a height (length) of at least half the depth of the basin is necessary according to PoD3. For the opening at the bottom a height of at least 30 centimetres is being used in demands. The more ideal situation is found in Figure 4.17 below.

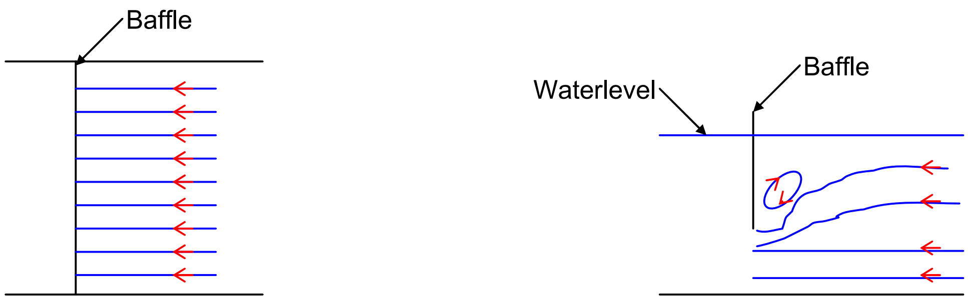

Figure 4.17: Topview (left) and sideview (right) of water flows underneath a baffle

The recommended alterations for the existing API-Separators can be summarised as follows:
- Shut down API-Separator 2 and divert the inflow to API-separator 1.
- Remove the oil collection tanks from the basins.
- Adjust the baffles conform the design criteria, the adjusted baffles are shown in Figure 4.18.

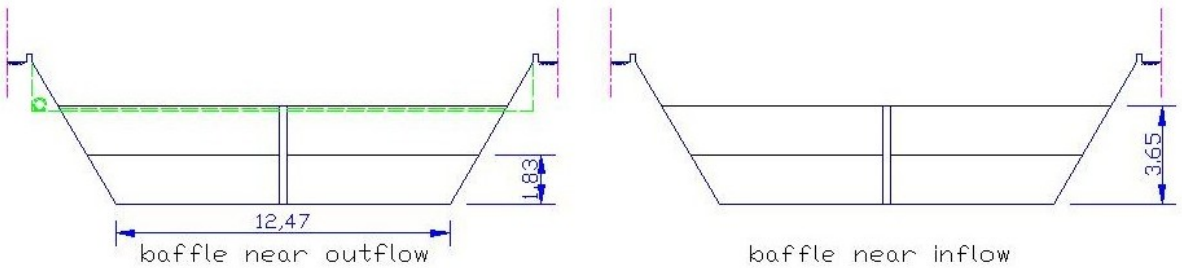

Figure 4.18: Cross-section of the adjusted baffles

4.6.2 Design of a new API-Separator

In this paragraph a new treatment plant will be designed for the treatment of the process water and the drainage water. In the present situation two API-Separators treat these water flows. At the new design this will be reduced to one treatment plant consisting of an API-Separator with multiple streets. The reason for this clustering is to get a better overview of the situation, the plant will be easier to operate and only one extra process street has to be designed in order to facilitate treatment in case of malfunctioning of one of the process streets.
The characteristics of the influent water are listed in the following table (Table 4.10). To determine the design flow rate the present influent flow rates of the two API-Separators are summed and multiplied by a factor 1,5 in order to cope with a possible rise in the wastewater flow rate in the future.

Table 4.10: Characteristics of the influent water

Characteristic	Flow rate (m^3/s)	Temperature (°C)	Oil global diameter (m)	Density of the oil (kg/m^3)	Density of the water (kg/m^3)	Vertical velocity, V_t (m/s)
Value	0,14	23	0,00015	900	1000	0,0013

The temperature is set quite low compared to the measured temperatures; a low temperature causes a slow rising velocity and therefore induces a more conservative design. A temperature of 23 degrees is chosen because the average air temperature in the cold season is about the same. The rise rate of the globules (V_t) is calculated using formula 8 as described in the design criteria in appendix 4.4.

Calculation of the dimensions of the separation basin

The first characteristic that can be calculated is the total cross-sectional area; the flow rate and the maximal horizontal velocity determine this characteristic. The maximal horizontal velocity should be less than or equal to 0,015 m/s or equal to 15 times the rise rate of the globules, whichever is smaller. Because 15 times the rise rate is equal to 0,020 m/s, the most stringent demand is 0,015 m/s. The total cross-sectional area can be calculated with the following formula:

$$A = \frac{Q}{v} = \frac{0{,}14}{0{,}015} = 9{,}4 m^2 \qquad (7)$$

Now the total cross-sectional area is known the dept and width of the separator can be determined. A number of demands determine the possible combinations of depth and width. These demands are listed in Table 4.11. In this table the possible combinations of widths and depths are listed and it is checked whether they meet the demands.

The demand for the width has to be checked using the width of one process street. In the table it can be seen that the total necessary width is divided by two, this implies that there are two process streets present. The reason for this is that in case of one process street the d/B demand is not met. In case of three process street the maximal width is out-limited.

Table 4.11: Demands for the width and the depth

Demands								
1 < d > 2,4		1,8 < B > 6			A > 9,4		0,3 < d/B > 0,5	
d (m)	check	total B (m)	B of one street	check	A (m^2)	check	d/B (-)	check
1,0	o.k.	9,33	4,67	o.k.	9,3	o.k.	0,11	not o.k.
1,2	o.k.	7,78	3,89	o.k.	9,3	o.k.	0,15	not o.k.
1,4	o.k.	6,67	3,33	o.k.	9,3	o.k.	0,21	not o.k.
1,6	o.k.	5,83	2,92	o.k.	9,3	o.k.	0,27	not o.k.
1,7	o.k.	5,49	2,75	o.k.	9,3	o.k.	0,31	o.k.
1,8	o.k.	5,19	2,59	o.k.	9,3	o.k.	0,35	o.k.
2,0	o.k.	4,67	2,33	o.k.	9,3	o.k.	0,43	o.k.
2,1	o.k.	4,44	2,22	o.k.	9,3	o.k.	0,47	o.k.
2,2	o.k.	4,24	2,12	o.k.	9,3	o.k.	0,52	not o.k.
2,4	o.k.	3,89	1,94	o.k.	9,3	o.k.	0,62	not o.k.

As can be seen in the table a depth of 1, 9 meters to 2,1 meters is conform the standards, the total width varies between 4,48 and 4,95 meters.

The last characteristic of the separator that needs to be determined is the length of the separation basin. The length should be at least 5 times the width of the process street or the value calculated with formula 15 in appendix 4.4, whichever is larger. The formula is based on the demand that the oil globules have to have reached the surface at the effluent side of the basin. In Table 4.12 the minimal lengths of the basin for the different dept-width ratios are listed.

Table 4.12: Minimal length by different depth-width combinations

d (m)	B (m) One street	L		
		L > (5*B)	L > F*(V_h/V_t)*d	
		L (m)	F (-)	L (m)
1,9	2,47	12,37	1,56	34,2
2,0	2,35	11,75	1,56	36,0
2,1	2,24	11,19	1,56	37,8

In the table can be seen that the demand based on the formula is the most stringent. In order to decrease the minimal length necessary, the horizontal velocity has to decrease. A number of similar calculations as the one explained above has been made; the results are listed in Table 4.13.

Table 4.13: Dimensions of the basin for different horizontal velocities

V_h (m/s)	d (m)	B (m) one street	L			V (m^3)
			L > (5*B)	L > F*(V_h/V_t)*d		
			L (m)	F (-)	L (m)	
0,015	1,7	4,12	21	1,56	31	214
	2,1	3,33	17	1,56	38	265
0,013	1,8	2,99	15	1,52	27	147
	2,3	2,34	12	1,52	35	188
0,012	1,9	3,07	15	1,24	22	127
	2,4	2,43	12	1,24	27	160
0,01	2,1	3,33	17	1,19	23	161
	2,4	2,92	15	1,19	26	185

In the table a column is added where the total volume of the separator is calculated, this characteristic is an indicator for the costs of the basin. As can be seen in the table the separator with the smallest volume has the shortest length as well, the depth and the width are not considerably larger than those of the other possible separators. Therefore this alternative is chosen.

In the total design of the treatment plant there will be three process streets available. The reason for this is to provide enough capacity in case one street has to be cleaned or maintained. A detailed drawing of this alternative is presented in appendix 4.5.

Design of the in- and outflow structures

The design criteria as listed in appendix 4.4 do not set any demands for the in in-and outflow structures other then these have to be designed properly. Especially the design of the inflow structure is of great importance, the oil globules present in the wastewater must not be fractionated due to turbulences caused by the inflow structure. Besides that, the inflowing water must not disturb the separation-process taking place in the middle compartment of the separator. The outflow structure is easier to design and installing a fixed weir is sufficient.

Because the inflow structure of API-Separator 1 is working properly, this design is used for the layout of the inflow structure of the new API-Separator. As can be seen in the AutoCAD drawing in appendix 4.5

the water flow is first distributed over the total width of the separator by a funnel shaped gutter. The next step is dividing the water over the total depth; a slope in the bottom of the process street accomplishes this. In the drawing it can be seen that the slope is designed at an angle of 45˚. This seems rather steep but since the flow velocities are small, it is not expected that flow disturbances are induced.
A baffle is placed after the inflow structure in order to minimise the influences of the inflowing water on the separation process in the middle compartment of the separator. At the water surface level slots are present in the baffles in order to let the oil pass to the separation basin.
The outflow construction is designed as a weir connected to a gutter in order to discharge the effluent.

Design of the baffles

As mentioned in the program of demands (PoD3) the baffles have to reach at least to half the depth of the basin, the opening under the baffle has to have a height of at least 30 cm and stretch over the whole width of the basin.
In the AutoCAD drawing in appendix 4.5 the size of the baffles is indicated, as can be seen the opening in the baffle has a height of 0,8 meters, no threshold is present under the baffle. This design is chosen in order to minimize flow deviations. By making the opening under the baffle as large as possible deviations of flow direction are minimized. As discussed in paragraph 4.6.1 a threshold under the baffles causes extra turbulences, therefore these are not present in the design.

Design of the oil removal devices

Every company that designs and sells API-Separators has its own oil abstraction devices. Since these devices do not treat the water, but only remove floating oil, not much attention is given to them in documentation about design criteria. The only information available is the type of oil removal device the different companies use.
There are different types of devices that can be applied. These vary from the floating oil skimmers as used at the refinery to rope and belt chains. There are basic designs of these types but also enhanced versions are available.
The best way to make a choice between the different types of oil removal devices is ask for more information at a few of the companies which sell API-Separators. They can tell what the pros and cons of their devices are and whether the company has a service program for the equipment they sell. Based on that information and the availability of a service/maintenance program a choice can be made.

Covering the basin

In order to control the emission of volatile organic compounds (VOC) and protect the separator from weather influences it is possible to cover the separators. A considerable drawback of covering the separator is that visual inspection becomes impossible threatening the proper operation of the separator.

At the refinery several storage basins are present full of oil, which are all uncovered. Since the surface of the separator is much smaller than these basins the covering of the separators is not considered necessary based on controlling the emission of VOC’s.
The weather influences however are of importance for a properly functioning separator. As was mentioned in paragraph 4.4.3 about the visual inspection of the separators, the amount of rain can be so high that even flooding can occur. In order to minimize the amount of rain entering the separator a roof will be constructed.
Another weather related problem is the wind. To be more specific, the suspended solids that enter the basin due to wind. In order to reduce this source of suspended solids a one-meter high wall will be constructed around the separator.

Removal of suspended solids

The removal of suspended solids can be accomplished continuously or discontinuously. In order to facilitate a continuous removal, a scraping device has to be installed in the basin. This scraping device needs adequate maintenance and operation to function properly. Because the analyses of the present situation made it clear that good maintenance and operation are not readily available, it is not wise to install such an installation.
Therefore it is advised to remove the suspended solids discontinuously by emptying a process street and cleaning the basin. After installing the treatment plant it is wise to investigate the amount of suspended solids after a period of half a year to determine the cleaning frequency. Because an extra process street is installed the effluent quality will not deteriorate due to shutting down one process street for cleaning purposes.

4.6.3 Decision about the alternatives

In the previous paragraphs two possible alternatives for treatment of the process water and the drainage water have been discussed. These alternatives are based on two principles. One principle is that the existing treatment facilities will be maintained; the other is that a complete new treatment facility will be implemented.
As for the design of the first alternative, it is impossible to meet all the demands set in the program of demands. As a result of that making a decision about the implementation of one of the alternatives based on technical reasons is easy. The design criteria are based on years of experience with API-Separators and therefore a part of the program of demands. Since it is not possible to adjust the existing separators in such a way that the demands are met, this alternative is rejected.
A new treatment plant is the only alternative left.

As for the operation and management of the separators, at present not very much is done about the separators at the refinery. They still have a reasonably high efficiency. Therefore it can be stated that with a little bit of attention and knowledge about the process, an API-Separator can be operated and managed very well.

The analysing of the water quality samples has proven that complicated analysing methods are available in Ecuador as well, so Boundary Condition 3 (BC3) will be met.

Since it is possible to first build a new separator, then redirect the flows and shut down the existing separators the new treatment facility will not interfere with the refinery processes (BC2).

4.7 Cooling water

Besides the water samples of the process and the drainage water a sample of the cooling water was taken, this was described in paragraph 4.5.3.The TPH-value found after analysing the sample is much higher than allowed in the regulations (69,4 mg/l compared to the 15 mg/l found in PoD2), therefore the cooling water needs to be treated.

In the gutter where the sample was taken, the water is flowing in a turbulent way. These turbulent flow conditions caused that the water and the oil were mixed up. The sample did not show oil separating from the water at the moment of sampling. Since the sample went straight to the laboratory it is not sure if the type of petrohydrocarbons present in the cooling water is separable from the water by means of gravity. It can be that only non-soluble petrohydrocarbons are present but other more soluble types are possible as well. Extensive investigations have to indicate what type of petrohydrocarbons are present in the cooling water. When this is known a pilot plant can be set up in order to determine the most effective treatment method.

4.8 Conclusions

The main goal of this part of the project is to reduce the pollution of the beach and the ocean due to discharges of partially untreated wastewater of the refinery of La Libertad.
The main conclusion after examining the processes of the refinery is that the API-Separators do function, but not as efficient as they could be. Furthermore their design does not meet the standards set by API publication 421.
It is possible to reduce the pollution of the beach and the ocean by taking several measures.
Since it is not possible to adjust the existing API-Separators in such a way that the standards will be met, the alternative to build a new separator system is chosen. This will give the refinery an opportunity to update their treatment facilities to the latest standards. A full design of this alternative is presented in appendix 4.5.

4.9 Recommendations

Although it is concluded that a new treatment facility will clearly be the best solution it is highly suspected that this facility will not be implemented in the near future. Awareness that environment is vulnerable but valuable and should be protected is not common in Ecuador.
The Ecuadorian government has made laws about pollution but they have not included an inspection system. So if any person or company violates these laws, most likely there will be no punishment. This means that there is no extra stimulant for oil companies to meet the standards set in the environmental laws, if it is cheaper to violate them. Only a national mentality change can raise enough awareness to alter this situation. This will probably take a long time.
This paragraph will recommend some steps to be taken in case a new treatment facility will not be implemented in the near future. Furthermore a measuring program will be discussed, as well as a recommendation to check the soil for pollution.

4.9.1 Adjustments to the constructions

Nevertheless quick adjustments of the existing facilities are wishful because of the impact on the environment by the polluted discharges. Therefore if no new system is built, at least adjustments made to the existing treatment plant are recommended. As explained at the description of the first alternative in paragraph 4.6.1, the tanks have to be placed outside the basins of API-Separator 1. The gaps in the baffles should be made larger and should reach the bottom of the basin. The drainage water of the storage tanks should be rerouted to API-Separator 1 where after API-Separator 2 can be shut down. These adjustments are all relatively cheap and easy to accomplish.

4.9.2 Operational changes

Besides these structural adjustments the management of the separators has to be altered as well. Inspections have shown that the API-Separators are not operated the way they should be. In order to operate the treatment facility wisely the operator should be educated in the processes that take place in the separators.

4.9.3 Measuring program

In a water treatment facility it is of fundamental importance to measure physical and chemical properties of the wastewater. These measurements provide a lot of knowledge about the performance of the treatment facility and can serve as a basis upon which proposed new structures can be designed. Without measurements no check can be made on whether the demands in the regulations are met.

Physical properties

The two most important physical properties that have to be measured are the water temperature and the flow rate. For measuring the temperature no additional structures have to be built, a regularly checked thermometer is sufficient. Care should be taken that the thermometer is regularly cleaned.
The flow rate can be measured with two different methods. The first is based on a structure that imposes a certain water depth for different flow rates, this water level has to be measured. The second is the velocity area method, where the velocity and the water depth need to be measured.
The velocity area method is not considered feasible mainly because of the difficulties in measuring the water velocity. It is possible to measure velocity both automatic or by hand. Measuring the velocity by hand requires a well-trained operator and is even then not very accurate and reliable. Measuring the velocity automatically is costly and sensitive since a reflective sonic Doppler is necessary. These measuring devices are not accurate at low flow rates and require a channel of considerable width.
A structure on the other hand is very robust and easy to measure. The principal of the measuring structure is based on introducing a transition from sub-critical flow to supercritical flow. A weir or a flume induces this transition. A weir has the major drawback that transport of sediment is not possible. The sediment that accumulates in front of the weir needs to be removed regularly. Therefore the flume is the best option.
A lot of companies offer well-designed and easy to implement flumes. The company Plasti-Fab, Inc. in Tualatin, Oregon, USA designs flumes like the one in Figure 4.19.

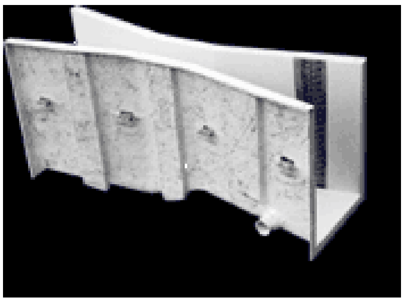

Figure 4.19: Parshall flume

The size of the flume is based on the measuring range of the flow rate, in Table 4.14 the possible sizes and measuring ranges are listed.

Table 4.14: Available Parshall flumes[6]

Parshall Flume	Flow Range - /1000 m^3/s
1"	0,13-5,68
2"	0,32-11,04
3"	0,95-52
6"	1,58-110
9"	2,8-250
12"	3,5-450
18"	5,0-690
24"	12-930
30"	15-1180

Parshall Flume	Flow Range - /1000 m^3/s
36"	18-1400
48"	36-1900
60"	44-2400
72"	74-2900
84"	86-3400
96"	98-3900
120"	170-9200
144"	190-16000

At the refinery three flow measuring structures have to be installed, one for each wastewater flow. The suggested flume sizes for the different wastewater flows are indicated in Table 4.15. The suggested flume has to have a measuring range large enough to measure lower as well as higher flow rates than the expected one.

Table 4.15: Suggested flume sizes

Waste water flow	Flow rate /1000 m^3/s	Suggested Parshall Flume
Process water	70	6"
Drainage water	20	3"
Cooling water	130	9"

At the installation of the flume one has to bear in mind that a supercritical flow is present in the measuring structure. This implies that the water is well mixed and oil globules can fractionate, this is not positive for the oil-water separation. Therefore the measuring instrument has to be installed as far from the API-Separators as possible.

[6] Data from: http://www.plasti-fab.com/company/company.html

Chemical properties

Besides physical properties, chemical properties need to be determined as well. Given that it is not possible to determine these properties at the site samples have to be taken. Samples can be taken either manually or automatically. Since neither of these sampling methods requires specific structures at the treatment plant no extra adjustments to the design of the structure have to be made.

Quality assurance project plan:

Before starting the measurements a quality assurance project plan (QAPP) has to be set up. An accurate QAPP insures the collection of data that is representative, reproducible, defensible and useful. Therefore a QAPP is of primary importance. The following items must be specified:

- Sampling plan:
The sampling plan must contain a detailed description of the measuring location and the moments the samples have to be taken.
- Sampling types and sizes:
In the plan it must be specified what type of samples have to be taken. Differences are made between a catch sample and a grab sample. Besides the type of sample the size of the sample has to be specified as well.
- Sample labelling and chain of custody:
In order to compare the different samples all the samples must be handled in the same way. To accomplish this, a detailed description about sample labels, sample seals, logbooks, chain of custody record, sample delivery to the laboratory, receipt and logging of sample and an assignment of sample analysis must be present in the plan.
- Sampling method:
The sampling techniques and the equipment used must be described with an explanation of the sampling method. It must be specified if the samples are taken manually or automatic.
- Sampling storage and preservation:
Also the type of containers, preservation method and maximum allowable holding times must be described for the correspondence of the samples.
- Sample constituents and analytical methods
The plan must contain a description of the parameters to be measured. In order to gather information that is reproducible all the field and laboratory test methods and procedures must be described. The detection limits for the individual methods must be indicated.

4.9.4 Soil pollution

The existing API-Separators are about 20 to 30 years old. In that time they have treated billions of litres of water polluted with oil. Because the separators have been used for such a long time the concrete structure could well be show signs of age. It is well possible that after 20 years there might have appeared cracks or porous spots where polluted water can enter the soil. Because the separators are designed to reduce the impact of the wastewater on the environment it is not desirable to pollute the soil. Therefore it is strongly advised to investigate whether the concrete structures of the API-Separators are permeable.

4.10 References

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Website

All websites mentioned are visited in the period of September and November 2005

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- www.ultraspin.com.au
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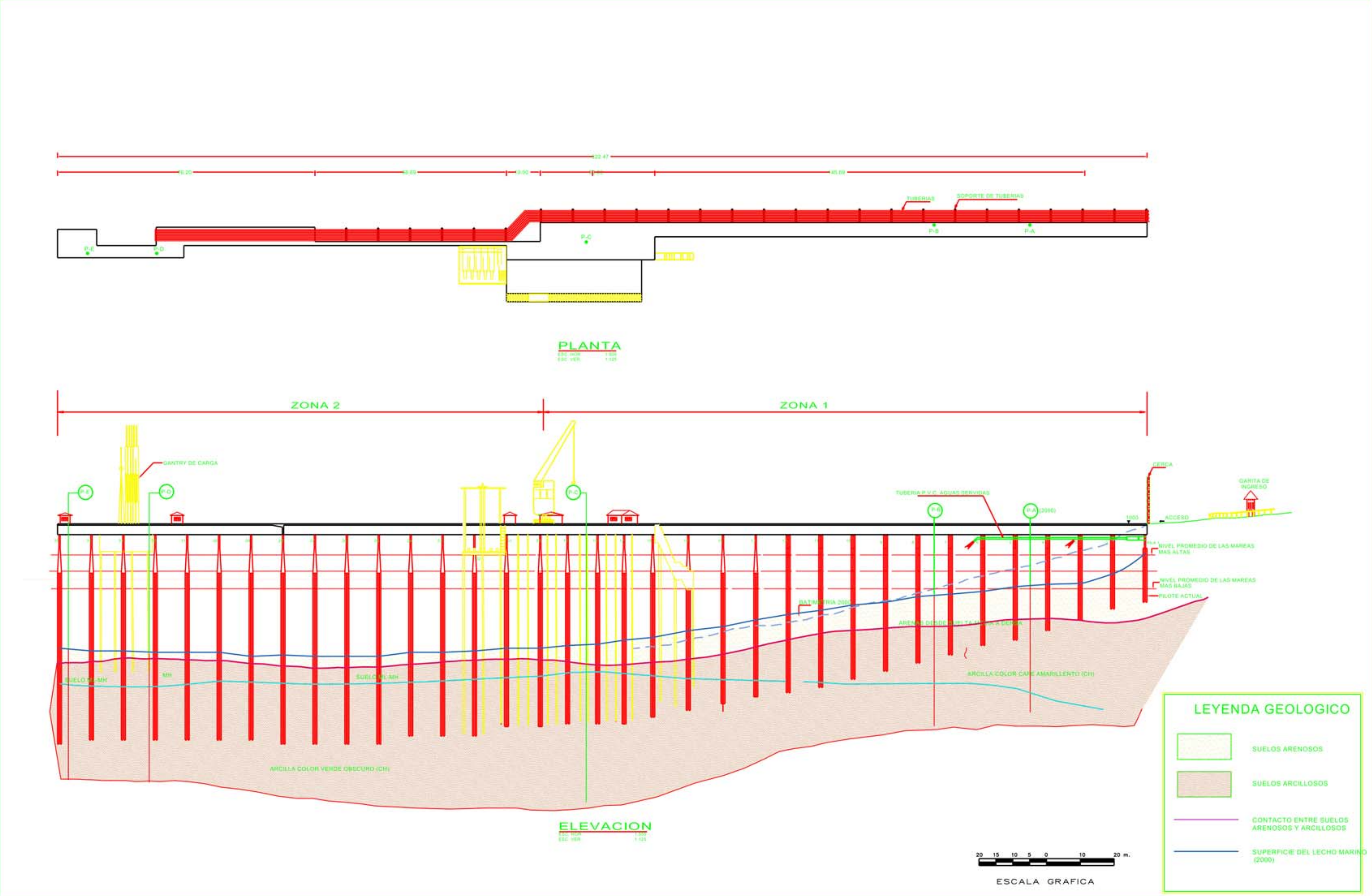
- www.ecologixsystems.com/w_owsmain.shtml



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PROYECTO: PROYECTO MUELLE LA LIBERTAD				ESCALA: INDICADAS
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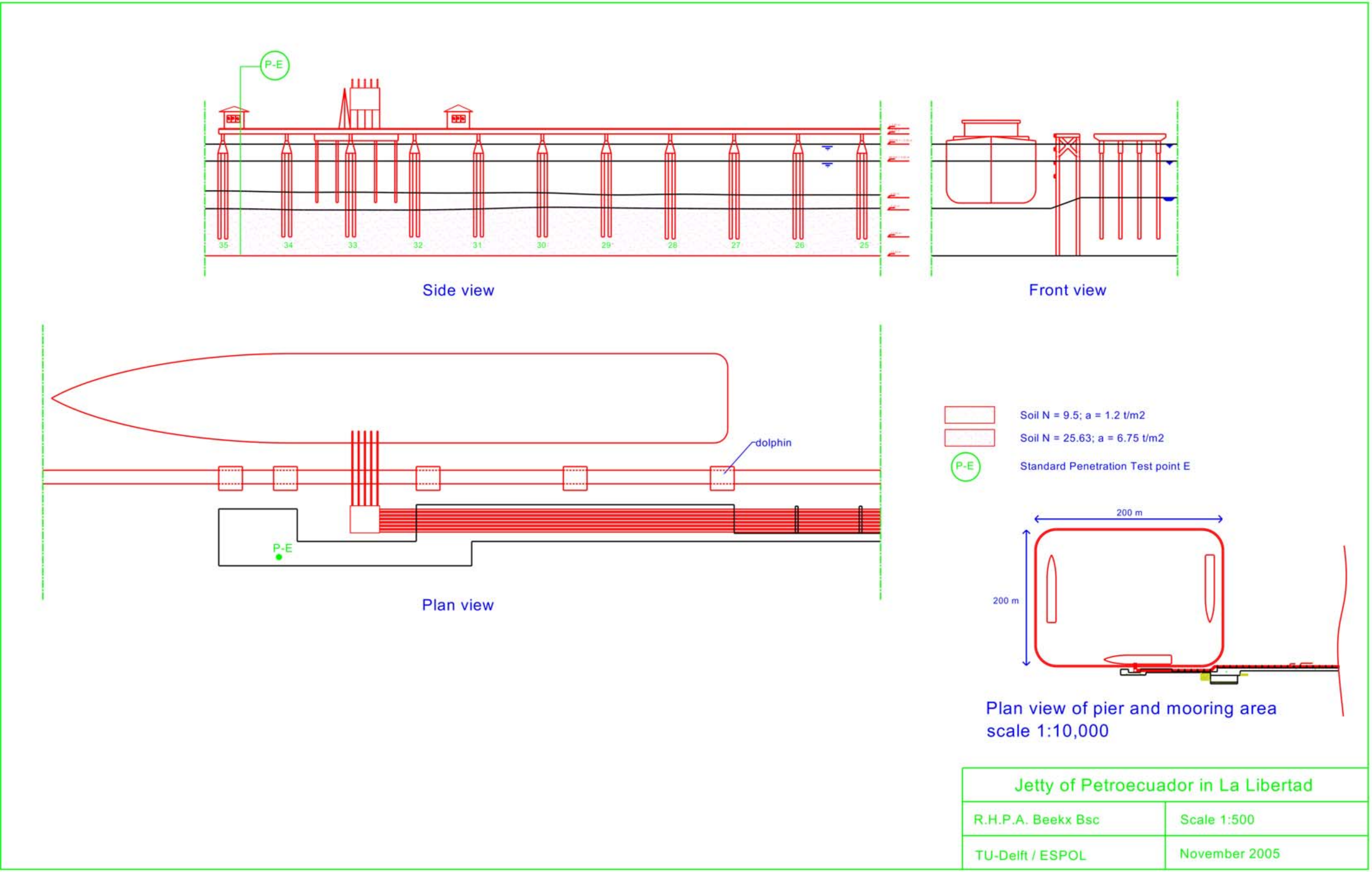
				No. PROYECTO: 166
				HOJA No.: 1 DE: 1


P-E
35
34
33
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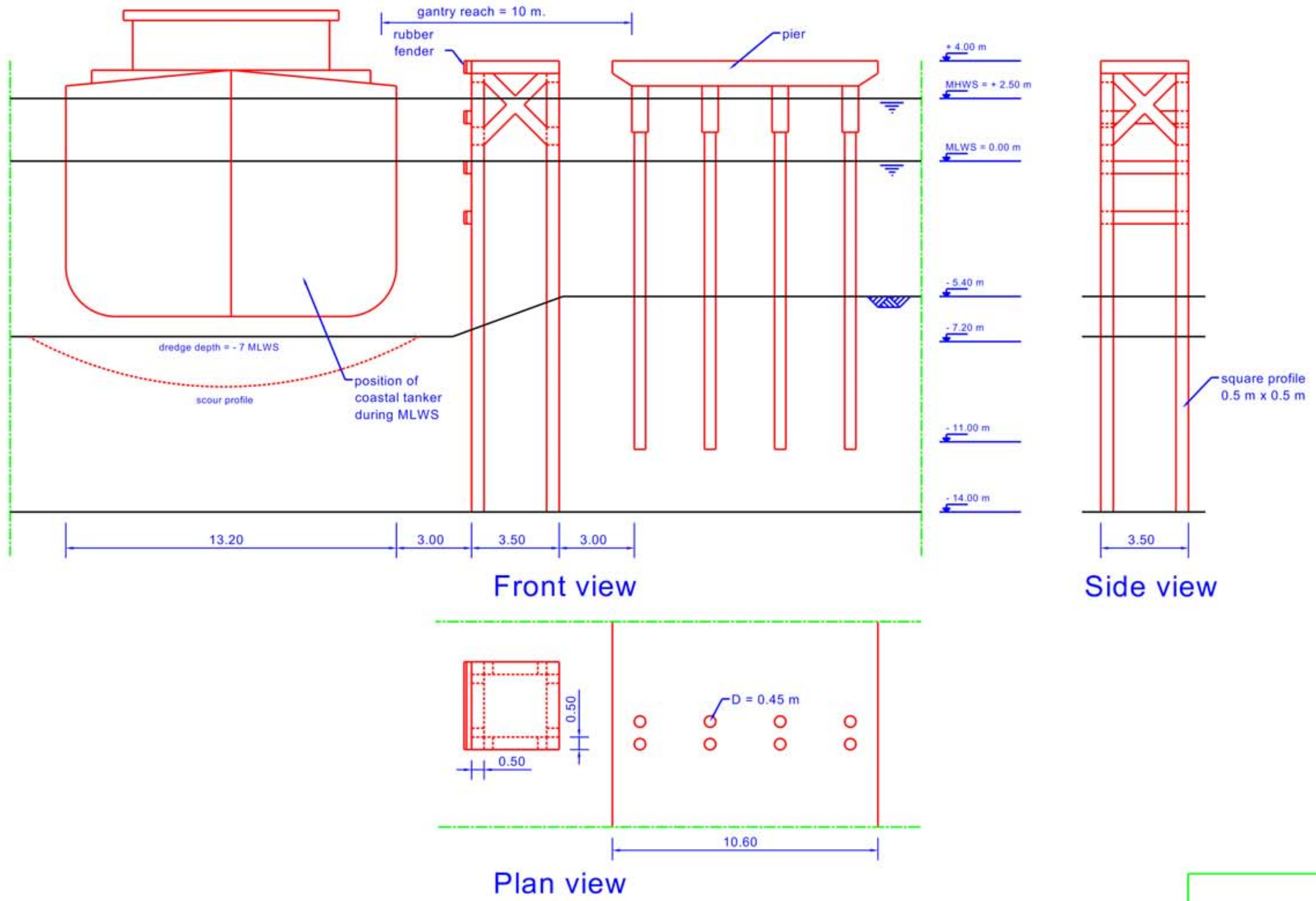
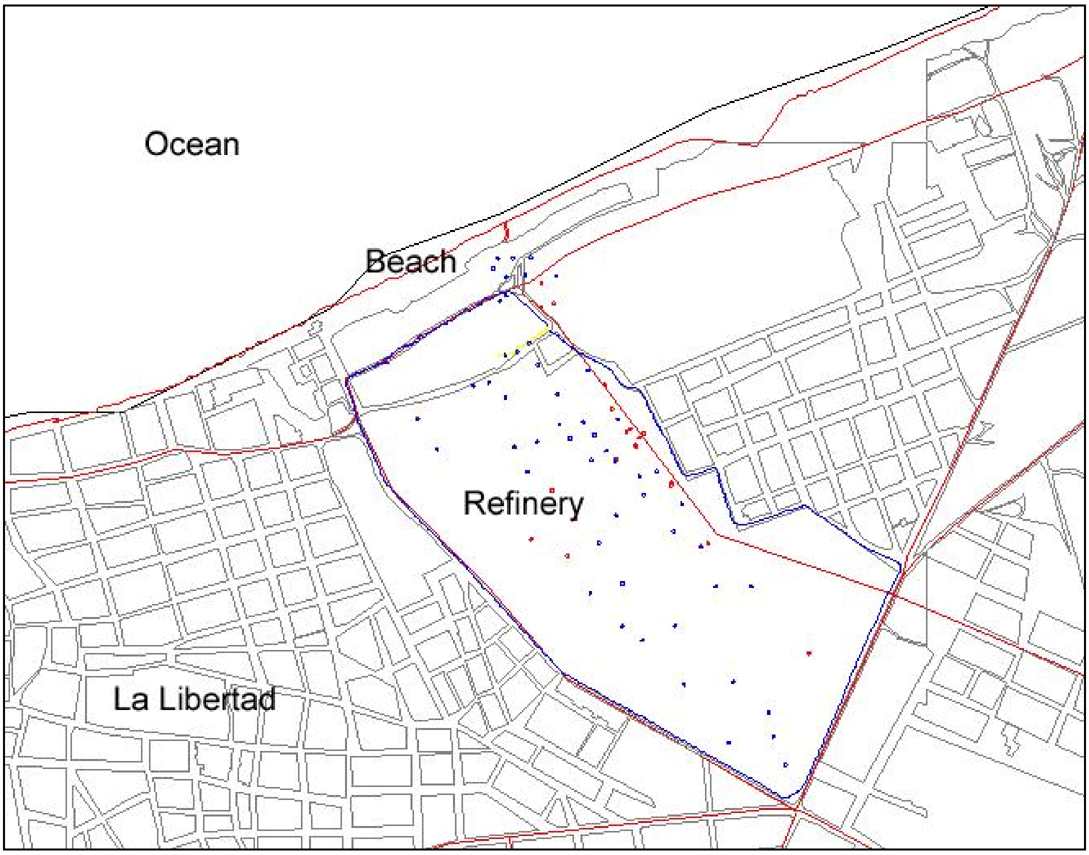
25
Side view
Front view
dolphin
P-E
Plan view
Soil N = 9.5; a = 1.2 t/m2
Soil N = 25.63; a = 6.75 t/m2
P-E
Standard Penetration Test point E
200 m
200 m
Plan view of pier and mooring area
scale 1:10,000
Jetty of Petroecuador in La Libertad
R.H.P.A. Beekx Bsc
Scale 1:500
TU-Delft / ESPOL
November 2005

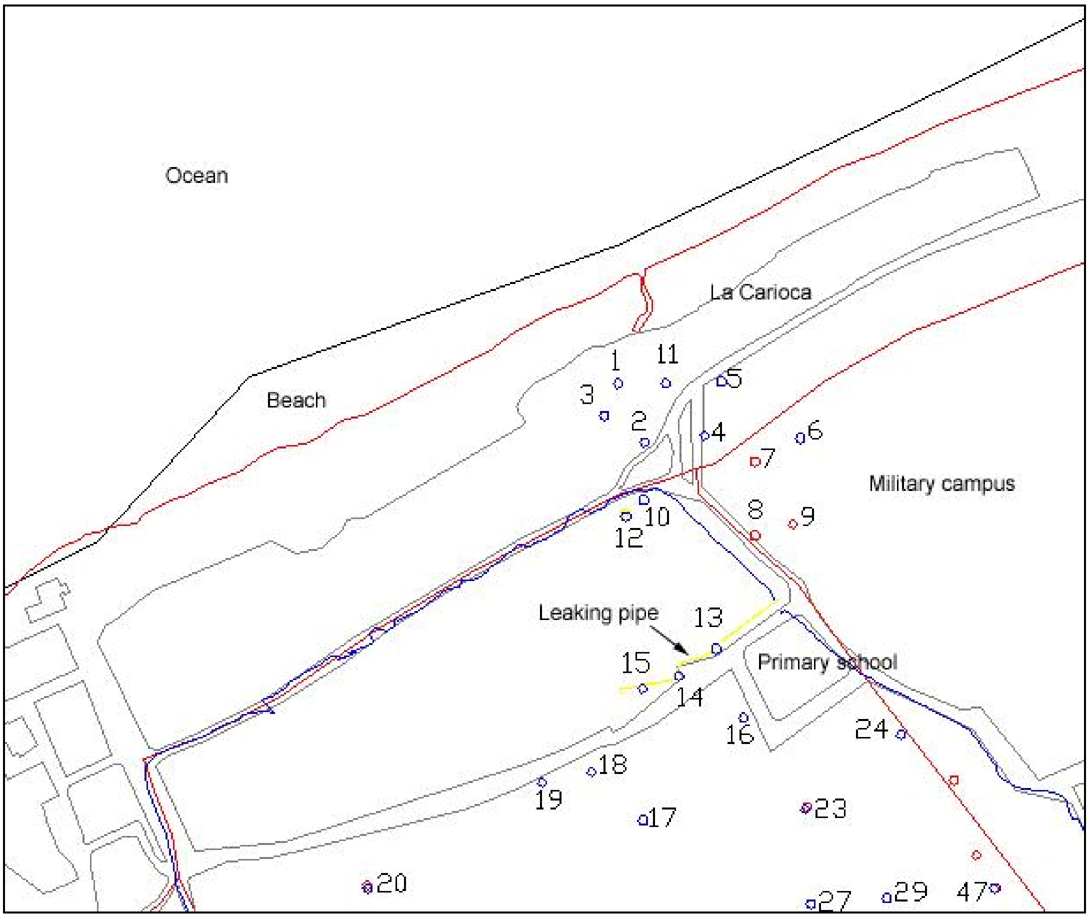


Dolphin design	
R.H.P.A. Beekx Bsc	Scale 1:5000
TU-Delft / ESPOL	November 2005

Annex 3.1: Map of the refinery of La Libertad



Annex 3.2: Locations of the soil samples



Annex 3.3: Soil samples

Soil samples

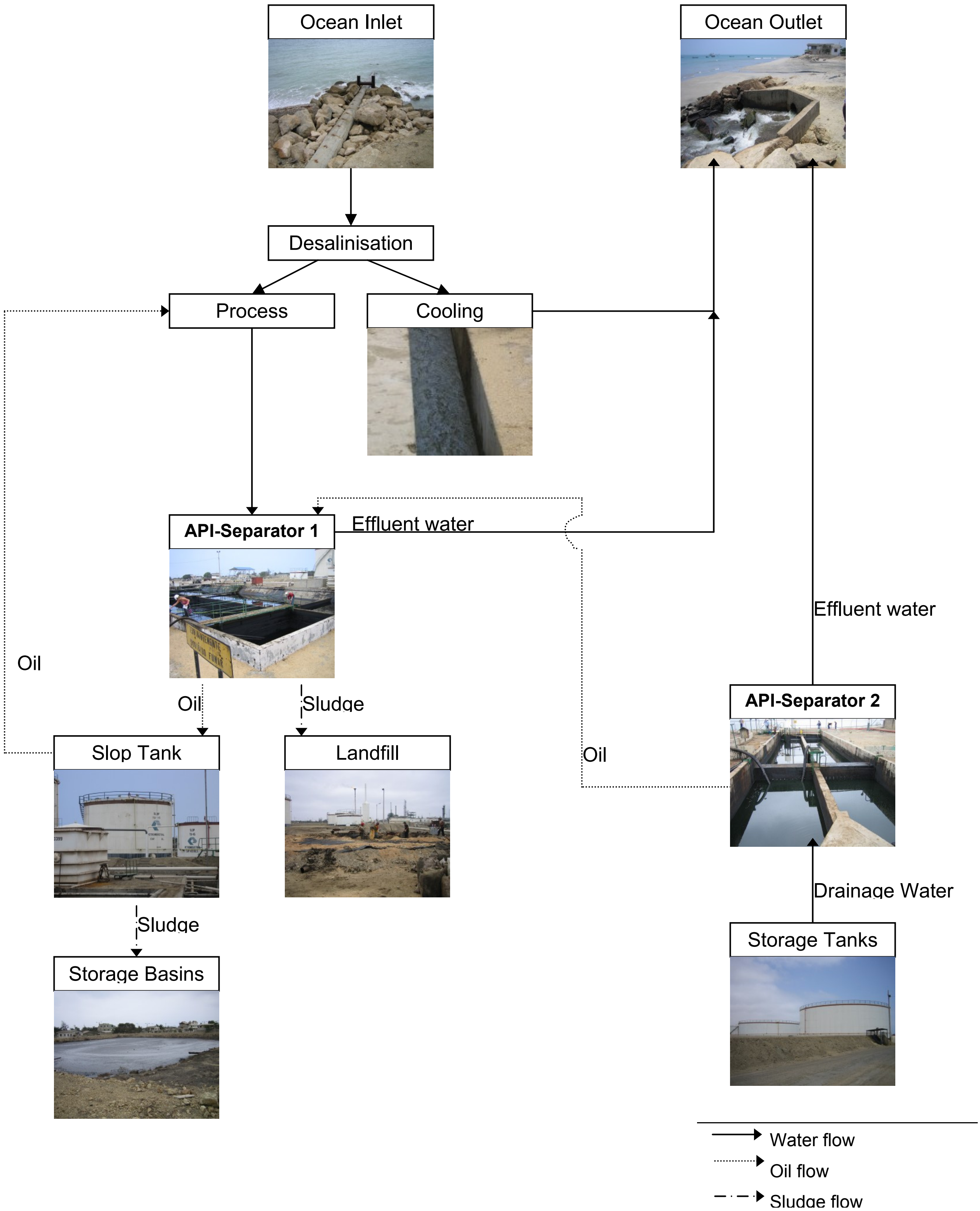
location	depth (m)	TPH (mg/l)
P01	1,00	857,40
P01	2,00	10200,00
P01	2,65	42203,00
P02	1,90	279,64
P02	3,00	4212,80
P02	4,00	3758,80
P03	1,00	1082,60
P03	2,00	6721,00
P03	3,00	4611,00
P04	1,00	470,30
P04	2,00	13200,00
P04	3,00	140,33
P05	1,00	110,30
P05	2,00	5270,40
P05	2,60	748,40
P06	1,00	265,70
P06	1,40	101,20
P07	1,00	4557,00
P07	2,00	10961,00

location	depth (m)	TPH (mg/l)
P08	1,00	11354,00
P08	2,00	29762,00
P09	0,60	26827,00
P10	1,00	81,40
P10	2,00	24,20
P11	1,00	13028,00
P11	2,00	3218,90
P11	2,90	4191,00
P12	1,00	9892,00
P12	2,00	5647,50
P13	1,00	57,00
P13	2,00	18,30
P13	2,50	16,10
P14	1,00	134,90
P14	2,00	29,90
P15	1,00	142,90
P15	2,00	231,20

Water samples

location	TPH (mg/l)
P1	3283,00
P2	14,20
P4	7,38
P5	4,10

Appendix 4.1: Process schedule with pictures



Appendix 4.2: AutoCAD drawing API-Separator 1

Appendix 4.3: AutoCAD drawing API-Separator 2

Appendix 4.4: Design criteria API-Separator

Design Criteria for an API-Separator according to API Publication 421 - Design and Operation of Oil-Water Separators (first edition, February 1990)[1]

General

The following parameters are required for the design of an oil-water separator:

a) Design flow (Q_m), the maximum wastewater flow. The design flow should include allowance for plant expansion and storm water runoff, if applicable.

b) Wastewater temperature. Lower temperatures are used for conservative design.

c) Wastewater specific gravity (S_w).

d) Wastewater absolute (dynamic) viscosity (μ). Note: Kinematic viscosity (v) of a fluid of density (ρ) is $v = \mu / \rho$.

e) Wastewater oil-fraction specific gravity (S_o). Higher values are used for conservative design.

f) Globule size to be removed. The nominal size is 0,15 millimeters (150 micrometers), although other values can be used if indicated by specific data.

The design of conventional separators is subject to the following constraints:

a) Horizontal velocity (V_H) through the separator should be less than or equal to 15 mm/s (0,015 m/s) or equal to 15 times the rise rate of the oil globules (V_t), whichever is smaller.

b) Separator water depth (d) should not be less than 1 m, to minimize turbulence caused by oil/sludge flight scrapers and high flows. Additional depth may be necessary for installations equipped with flight scrapers. It is usually not common practice to exceed a water depth of 2,4 m.

c) The ratio of separator depth to separator width (d/B) typically ranges from 0,3 to 0,5 in refinery services.

d) Separator width (B) is typically between 1,8 and 6 m in refinery services.

e) By providing two separator channels, one channel is available for use when it becomes necessary to remove the other from service for repair or cleaning.

f) The amount of freeboard specified should be based on consideration of the type of cover to be installed and the maximum hydraulic surge used for design.

g) A length-to-width ratio (L/B) of at least 5 is suggested to provide more uniform flow distribution and to minimize the effects of inlet and outlet turbulence on the main separator channel.

[1] Adapted from: Environmental Guidelines for Water Discharges from Petroleum Industry Sites in New Zealand, Ministry for the Environment, December 1998

Figure 4.22 shows a typical oil-water separator and depicts the design variables listed above.

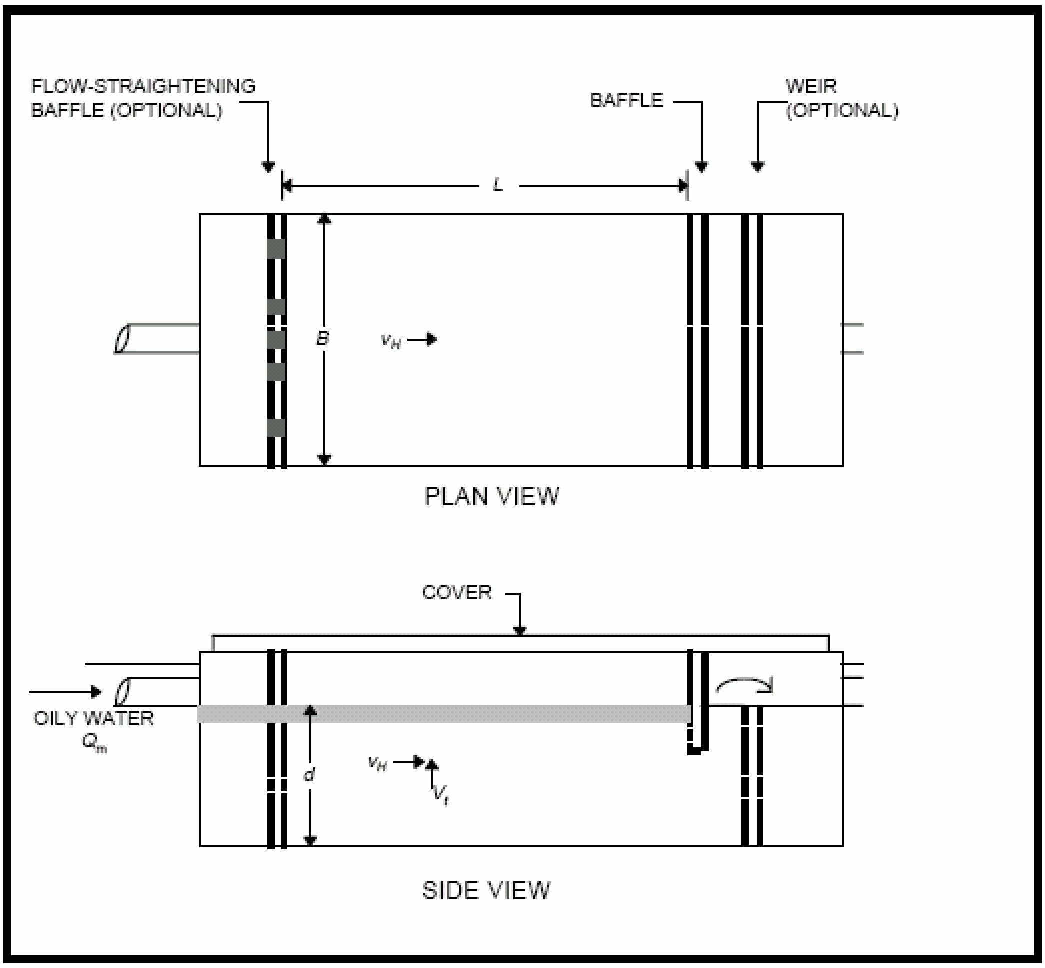


Figure 4.22: Design variables for oil interceptors

The oil-globule rise rate (V_t) can be calculated by Equation 8 or 9 shown below. Equation 8 should be used when the target diameter of the oil globules to be removed is known to be other than 0,15 mm and represents a typical design approach. Equation 9 assumes an oil globule size of 0,15 mm.

$$V_t = \frac{g(\rho_w - \rho_0)d_p^2}{18\mu} \qquad (8)$$

$$V_t = 0{,}0123\left(\frac{\rho_w - \rho_o}{\mu}\right) \qquad \text{(where } d_p = 0{,}15 \text{ mm)} \qquad (9)$$

where:

V_t = vertical velocity, or rise rate, of the design oil globule [m/s].

g = acceleration due to gravity (9,81 m/s^2).

μ = dynamic viscosity of wastewater at the design temperature [N.s/m^2]

ρ_w = density of water at the design temperature [kg/m^3]

ρ_o = density of oil at the design temperature [kg/m^3].

d_p = diameter of the oil globule to be removed [m].

S_w = specific gravity of the wastewater at the design temperature [-], which is equal to

$$S_w = \frac{\rho_{afvalwater}}{\rho_w} = 1 \tag{10}$$

S_o = specific gravity of the oil present in the wastewater ([-], not degrees API), which is equal to

$$S_o = \frac{\rho_0}{\rho_w} \tag{11}$$

Alternatively, if using kinematic viscosity, Equations 8 and 9 may be rearranged as follows:

$$V_t = \frac{g}{18\nu}\left(1 - S_o\right)d_p^2 \tag{8a}$$

With g is equal to 9,81 and d_p is equal to 0,00015 m this formula becomes:

$$V_t = 0{,}0123\left(\frac{1 - S_o}{\nu}\right) \tag{9a}$$

where:

v = kinematic viscosity of the wastewater at design temperature [m^2/s].

Once the oil-globule rise rate (V_t) has been obtained from Equation 8 or 9, the remaining design calculations may be carried out as described in the following sections.

Horizontal Velocity (V_H)

The design mean horizontal velocity is defined by the smaller of the values for V_H in m/s obtained from the following two constraints:

$$V_H = 15V_t < 1.5 \tag{12}$$

These constraints have been established based on operating experience with oil-water separators. Although some separators may be able to operate at higher velocities, 15 mm/s has been selected as a recommended upper limit for conventional refinery oil-water separators. Most refinery process-water separators operate at horizontal velocities much less than 15 mm/s at average flow. All separators surveyed by the API in 1985 had average horizontal velocities of less than 10 mm/s, and more than half had average velocities less than 5 mm/s, based on typical or average flow rates. Maximum flow rates were not reported in the survey; however, design flow rates were typically 1,5 - 3 times the typical average flow rates.

Minimum Vertical Cross-Sectional Area (Ac)

Using the design flow to the separator (Q_m) and the selected value for horizontal velocity (V_H), the minimum total cross-sectional area of the separator (A_c) can be determined from the following equation:

$$A_c = \frac{Q_m}{V_H} \tag{13}$$

Where:

A_c = minimum vertical cross-sectional area [m^2].

Q_m = design flow to the separator, in [m^3/s].

V_H = horizontal velocity [m/s].

Channel Width and Depth (B and d)

Given the total cross-sectional area of the channels (A_c) and the number of channels desired (n), the width and depth of each channel can be determined. A channel width (B), generally between 1,8 - 6 m, should be substituted into the following equation, solving for depth (d):

$$d = \frac{A_c}{Bn} \tag{14}$$

where:
d = depth of channel [m].
A_c = minimum vertical cross-sectional area [m^2].
B = width of channel [m].
n = number of channels [-].

The channel depth obtained should conform to the accepted ranges for depth (1-2,4 m) and for the depth to width ratio (0,3 - 0,5).

Separator Length (L)

Once the separator depth and width have been determined, the final dimension, the channel length (L), is found using the following equation:

$$L = F\left(\frac{V_H}{V_t}\right)d \tag{15}$$

where:
L = length of channel [m].
F = turbulence and short-circuiting factor [-], see figure 4.23.
V_H = horizontal velocity [m/s].
V_t = vertical velocity of the design oil globule [m/s].
d = depth of channel [m].

If necessary, the separator's length should be adjusted to be at least five times its width, to minimize the disturbing effects of the inlet and outlet zones.

Equation 15 is derived from several basic separator relations:

a. The equation for horizontal velocity ($V_H = Q_m/A_c$), where Ac is the minimum total cross-sectional area of the separator.
b. The equation for surface loading rate ($V_t = Q_m/A_H$), where A_H is the minimum total surface area of the separator.
c. Two geometrical relations for separator surface and cross-section area (A_H = LBn and A_c= dBn), where n is the number of separator channels.

The turbulence and short-circuiting factor (F) is a composite of an experimentally determined short-cutting factor of 1,21 and a turbulence factor whose value depends on the ratio of mean horizontal velocity (V_H) to the rise rate of the oil globules (V_t). A graph of F versus the ratio V_H/V_t is given in Figure; the data used to generate the graph are also given below.

v_H/V_t	Turbulence factor (F_t)	F=1.2F_t
3	1.07	1.28
6	1.14	1.37
10	1.27	1.52
15	1.37	1.64
20	1.45	1.74

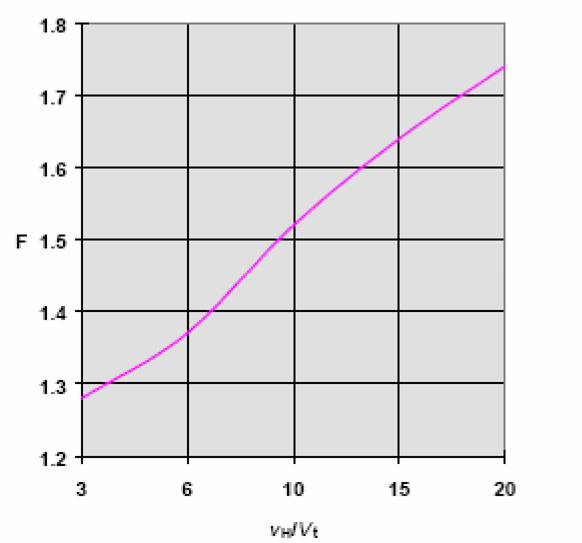


Figure 4.23: Recommended values of F for various values of V_H/V_t

Minimum Horizontal Area (A_H)

In an ideal separator - one in which there is no short-circuiting, turbulence, or eddies - the removal of a given suspension is a function of the overflow rate, that is, the flow rate divided by the surface area. The overflow rate has the dimensions of velocity. In an ideal separator, any oil globule whose rise rate is greater than or equal to the overflow rate will be removed. This means that any particle whose rise rate is greater than or equal to the water depth divided by the retention time will reach the surface, even if it starts from the bottom of the chamber. When the rise rate is equal to the overflow rate, this relationship is expressed as follows:

$$V_t = \frac{d_i}{T_i} = \frac{d_i}{\frac{L_i B_i d_i}{Q_m}} = \frac{Q_m}{L_i B_i} = V_o \quad (16)$$

where:

d_i = depth of wastewater in an ideal separator [m].

T_i = retention time in an ideal separator [s].

L_i = length of an ideal separator [m].

B_i = width of an ideal separator [m].

Q_m = design flow to the separator [m^3/s].

V_o = overflow rate [m/s].

Equation 16 establishes that the surface area required for an ideal separator is equal to the flow of wastewater divided by the rise rate of the oil globules, regardless of any given or assigned depth. By taking into account the design factor (F), the minimum horizontal area (A_H), is obtained as follows:

$$A_H = \frac{Q_m}{V_t} \qquad (17)$$

where:

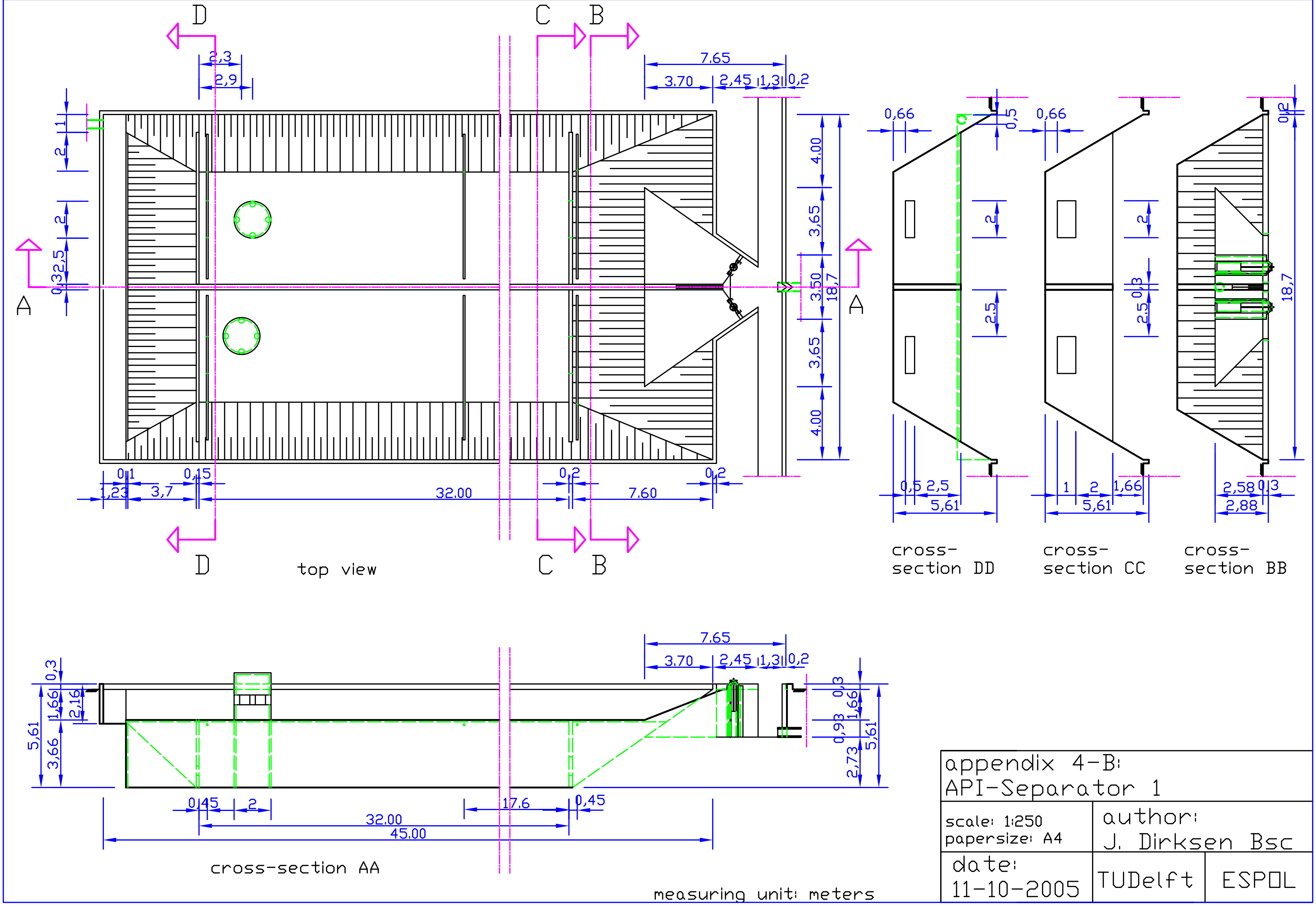
A_H = minimum horizontal area [m^2].

F = turbulence and short-circuiting factor [-] (see figure 4.23)

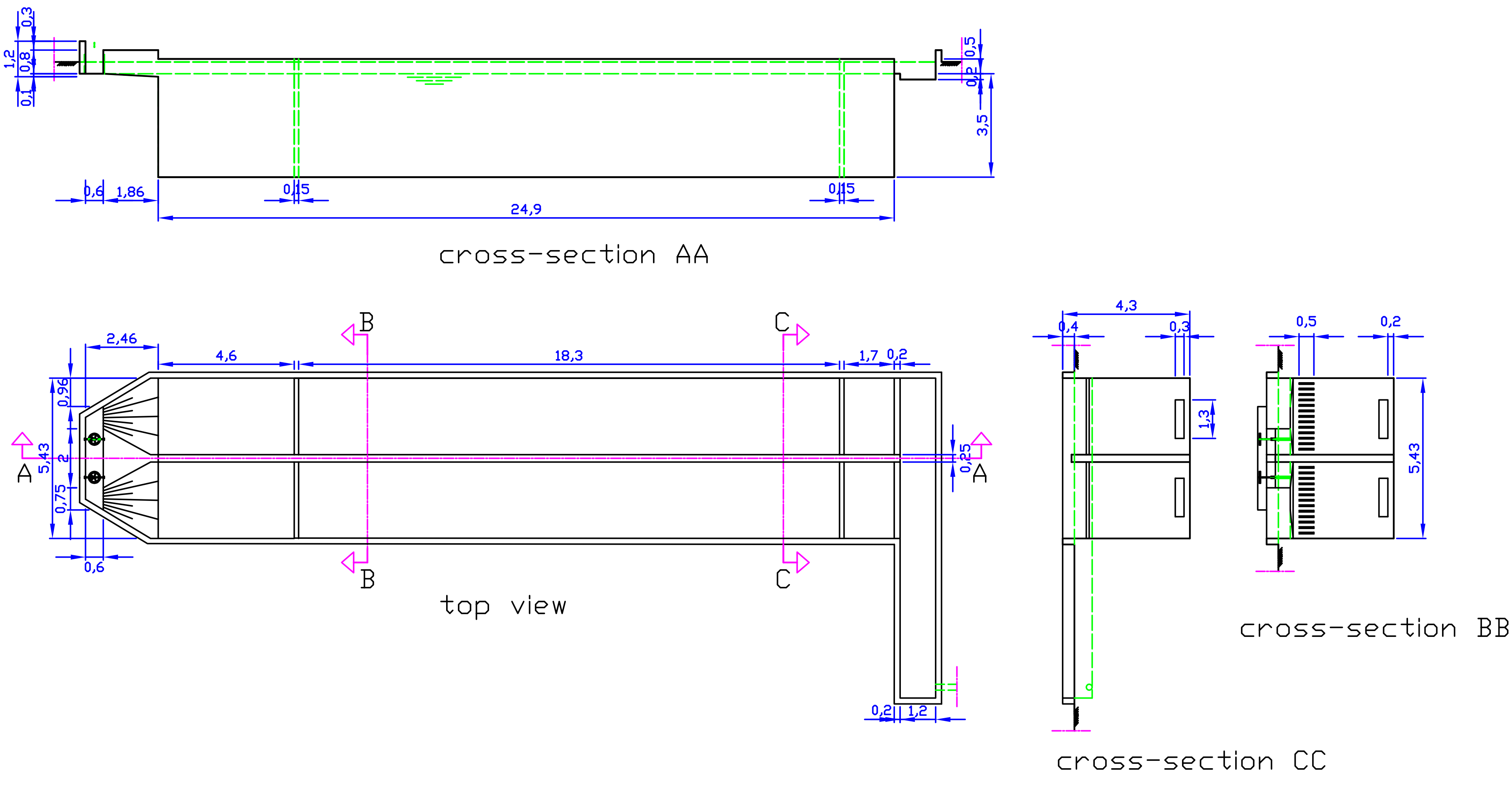
Q_m = wastewater flow [m^3/s].

V_t = vertical velocity of the design oil globule, in m/s.

Appendix 4.5: New designed API-Separator



appendix 4-B: API-Separator 1		
scale: 1:250 papersize: A4	author: J. Dirksen Bsc	
date: 11-10-2005	TUDelft	ESPOL



measuring unit: meters

appendix 4-C: API-Separator 2		
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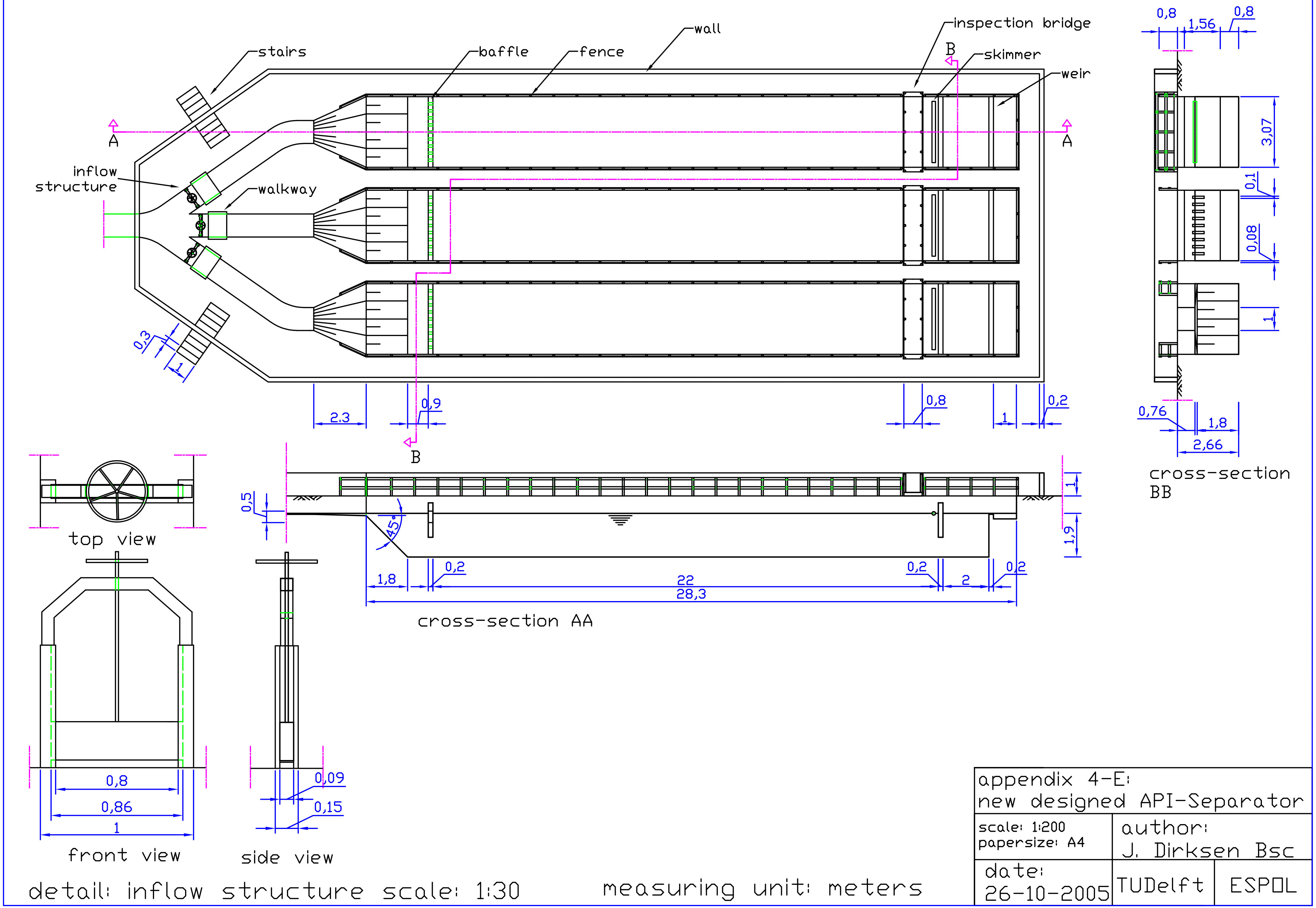
date: 11-10-2005	TUDelft	ESPOL



measuring unit: meters

appendix 4-E: new designed API-Separator	
scale: 1:200 papersize: A4	author: J. Dirksen Bsc
date: 26-10-2005	TUDelft | ESPOL