DELFT UNIVERSITY OF TECHNOLOGY

MASTER THESIS

CIE5060-09

Outfall development to provide a good recreational water environment near a polluted river

Panama City, Rio Matasnillo

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Preface

Starting from the 26th of November 2018, this Master thesis was written to conclude the Master Hydraulic Engineering at the faculty of Civil Engineering and Geo-sciences at TU Delft. The subject of this thesis was granted to me by Boskalis after an earlier successful collaboration with a project in Jamaica. The opportunity to work on an international project at a leading company was a great experience.

I would like to thank my committee and supervisors for their help and support. My daily supervisor Mark Voorendt for all his guidance and help with the report. Chairman Oswaldo Morales Napoles and Marie-Claire ten Veldhuis for taking part in the committee. Their help and expertise were of great importance for the final result.

From Boskalis I would like to thank Davy Bijleveld ,for his daily supervision and introduction to the company, Daan Rijks for getting me involved with the project and guidance. Finally thanks go out to John Brocatus , for receiving and helping me during my stay in Panama City.

Simon Schilder
Volendam, January 30

Summary

The main objective of this project was to come up with a solution for the nuisance caused by odour and poor water quality from Matasnillo river, such that the water quality along the Cinta Costera area in Panama City is sufficient for recreational use and a beach can be made at the project location. To realise this the main objective was divided into the following three functional requirements:

- Water quality in the bay has to meet standards to be fit for recreational use. From water samples it became clear that the main pollutant in the water of Matasnillo river is of fecal origin. For fecal coliforms and E.Coli bacteria the tolerable limit is 250 CFU/100 ml. For the Total coliforms the limit is a maximum of 500 CFU/100 ml.
- During rainy season the river must have sufficient capacity to discharge the water and will not cause floodings. Probabilistic analysis of the rain data resulted in a discharge of 150 m^3/s with a return period of 20 years. This will be the upper limit for the design capacity.
- Remove the odour that originates from the river area.

At an early stage in the design phase, several concepts were formed which could be a possible solution. From the concepts, created in the field of "hard-engineering", the pipeline and hybrid concept looked most promising. Other ideas in the field of "soft-engineering" and "non-engineering" solutions, such as education and connection to sewage, could help reduce the overall problem but were considered inadequate to solve the problem all by itself. Therefore, the pipeline and hybrid concept were worked out in further detail. An upper and lower boundary were determined for the maximum discharge, since no data was available on the hydraulic properties of the river. The upper limit was set by performing a probabilistic analysis on a 21 year long data set of rainfall intensity measured at a weather station in Panama City and combining this with the rational method used in hydrology to compute the discharge. From this, the upper limit of 141 m^3/s (for a 20 year return period) was found. The lower limit was set in consultation with employees of Boskalis at location. They estimated the maximum discharge to be 60 m^3/s. Soon it became clear that the pipeline concept would have insufficient discharge capacity to meet even the lowest boundary condition of 60 m^3/s. From this point only the hybrid concept, which consists out of a culvert in case of high discharge and an outfall that redirects the polluted water to a safe distance offshore.

To prove that the water quality would be safe, a Delft3D model was made to model the water quality in the bay. Several outfall locations were tried until a location was found that met the tolerable limits. This resulted in an outfall location at 1.4 km South-East from the mouth of Matasnillo river, see Figure 1a.

For the discharge capacity of the culvert it was decided to work out 2 variants for the 60 and 150 m^3/s design capacity because the discharge was uncertain and this would provide an insight in difference in costs and constructability for the two designs. Based on a hydraulic head balance the dimensions of the culvert turned out to be (number of sections x height x width) 2x3x4 and 3x4x4 m for the 60 and 150 m^3/s design, respectively. For the outfall pipeline a similar approach was used. Here the dimensions should provide enough capacity such that all the water in the river can be discharged from high to low tide during spring and neap tide. This resulted in 2 pipelines with a diameter of 2 meters each.

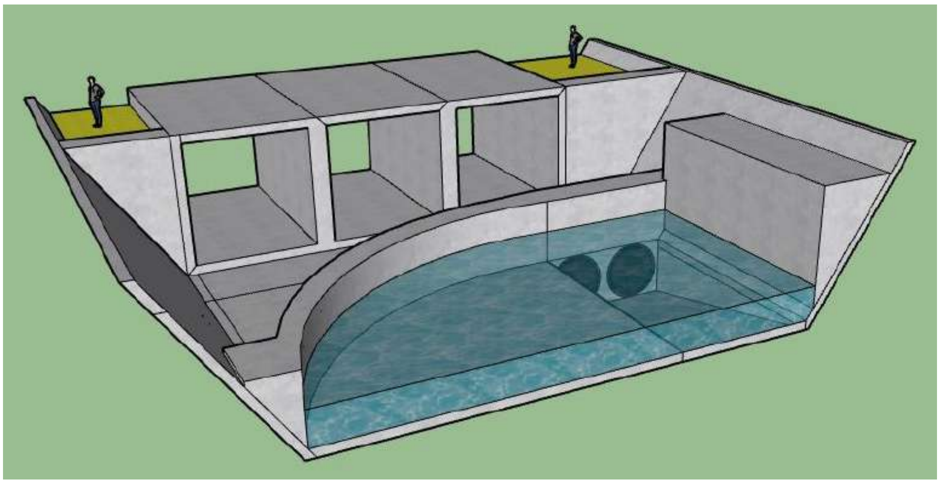
The odour will be solved by 1) choosing the inlet location at a distance of 125 meters from the beach, 2) remove polluted sludge that is settled in the river and around the river mouth, 3) provide a sufficient discharge capacity in the outfall pipeline during dry conditions such that nu polluted water builds up, 4) the Sanitation Program will continue their work to improve the water quality in the river.

At last, the construction method, a time planning and costs were worked out for both design variants of the hybrid concept. Because the 150 m^3/s has significantly more concrete culvert sections (600 vs 320 for the 60 m^3/s) the time duration for this variant is expected to be 100 days longer (474 and 368 days for the 150 and 60 m^3/s design, respectively). The costs of the total project were computed by looking at each construction phase (the costs for construction of the beach and groynes are not taken into account). In the end the total costs of the 150 m^3/s design variant turned out to be only 20% higher compared to the 60 m^3/s design variant. Therefore, in combination with the rainstorm data analysis, it was advised to go for the 150 m^3/s design variant

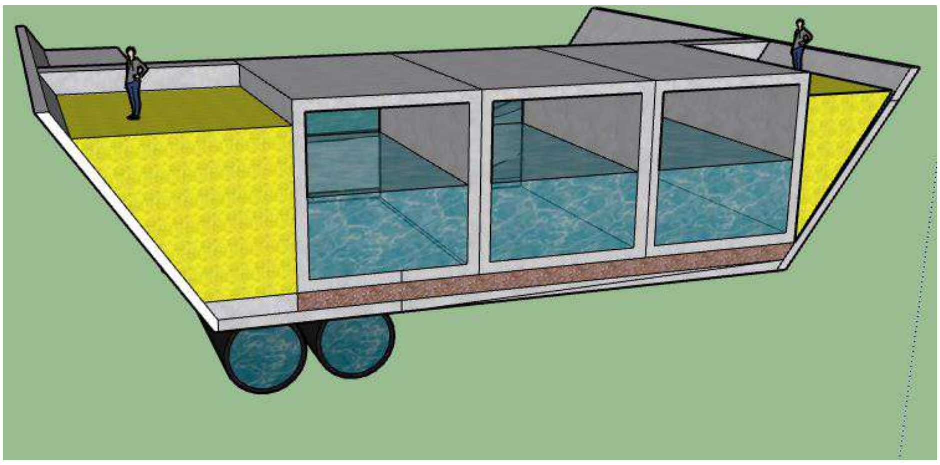
of the hybrid concept as final solution. This design can be seen in Figure 1b and 1c with the overall positioning of the design in Figure 1a.

To prevent garbage and rubble to clog the outfall pipelines it is advised to construct a garbage fence in front of the inlet structure. This can be an automated system that scoops out the collected garbage in front of the fence and dumps it in garbage bins. Such system already exist in the Netherlands at pumping stations that control the water level in the polders.

(a) Top view of the position for the hybrid concept at Bella Vista beach.



(b) River side view at culvert inlet for 150 m^3/s design capacity at low tide.



(c) Sea side view at culvert inlet for 150 m^3/s design capacity at high tide.

Figure 1: Culvert inlet for 150 m^3/s design capacity at river mouth

Contents

Chapter 1. Introduction

Over the past few decades the city of Panama City has developed itself in both economical activity as well as tourist attraction. Since the latest upgrade of the Panama Canal and improvements to the infrastructure, the city now has new opportunities for further growth in both areas.

To create a more attractive city, there have been several plans to develop the recreational space along Panama City's coastline in the past. Now the wish for improvements of the area have found new interest under the municipality and government. In reaction, Boskalis B.V. has proposed a plan to improve the waterfront in Panama City both in recreational and economical way.

This Msc thesis focuses on the design of an outfall for Matasnillo river. At the moment this river discharges in the bay along the Cinta Costera in city centre of Panama City. Matasnillo river runs directly through the city for 6 km, subjecting it to both industrial and residential pollution leading to poor water quality. In 2010 it was found that the river is critically contaminated (Endi, 2010) and that the water may not be used for consumption and recreation.

1.1 Location

Panama City is located at the Pacific ocean side of the country. In Figure 1.2 the layout of the bay and its location can be seen. The coastline area is called Cinta Costera and stretches over 2.5 km. Because of the large tidal range of 5 m some parts of the bay and largest part of the marina run dry during low water.

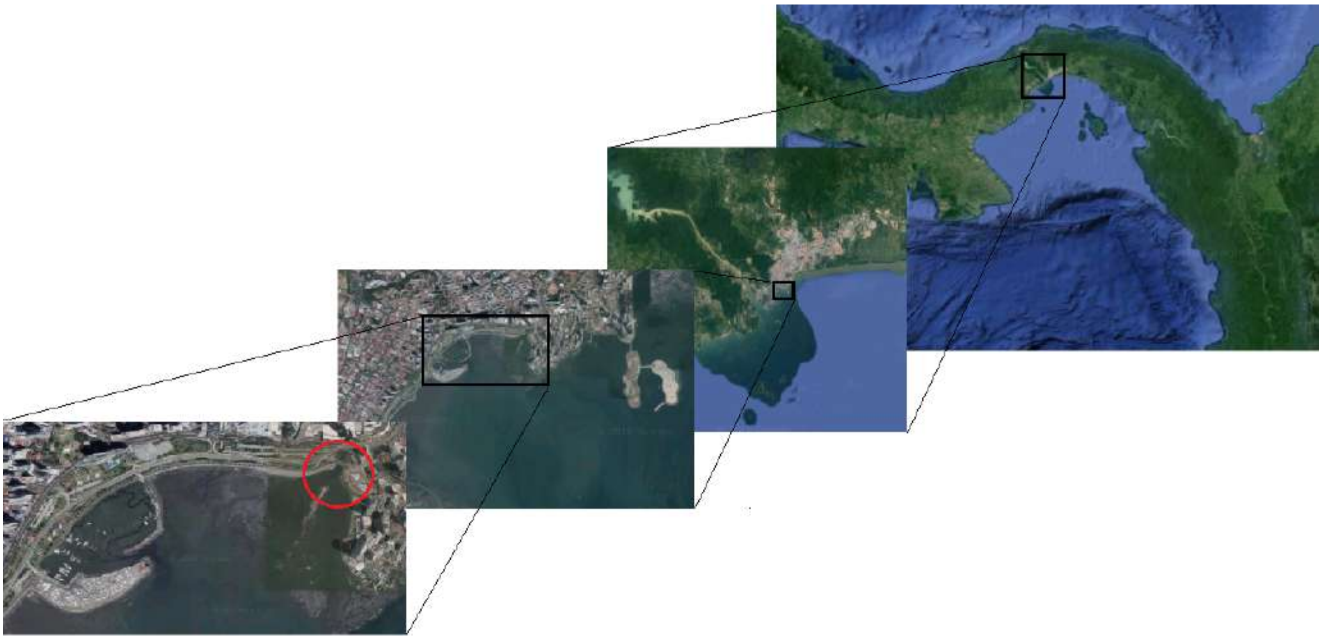


Figure 1.2: Project location at Panama City (Google Earth, 2018).

Figure 1.3a gives an overview of the entire waterfront of the Cinta Costera area. At the moment only a small stretch of land is located between the busy highway and the waterfront. Several recreational activities are located on the Cinta Costera area such as basketball fields, tennis fields and a sport complex, so it mainly focusses on sports. All these areas are connected with a small walkway which is only accessible from the city by a few pedestrian crossovers. Apart from the recreational sport activities, the area also has a marina which is home to Club De Yates y Pesca Panama.Currently, the marina is in a bad shape and functions more as a storage for boats on land than as an active marina. At the most North-Eastern section of Cinta Costera the Rio Matasnillo discharges into the sea (see Figure 1.3b).

(a) Current situation Cinta Costera **(b)** Rio Matasnillo

Figure 1.3: Current situation cinta costera, Panama City (Boskalis, 2018).

1.2 Problem analysis

To get a better look on the problems, the area is divided into several smaller sections and analyzed separately. The following parts are looked at in more detail:

- Rio Matasnillo water quality
- Cinta Costera green zone
- Accessibility & connectivity
- Lack of beach

1.2.1 Rio Matasnillo water quality

Rio Matasnillo is the only river in Panama that has its origin and mouth in the city. The river is only 6 km long and drains a river catchment area of 383 km^2 (ETESA, 2009). Because the river runs trough the city, most of the water is drained to the river quickly due to the hard revetments and can't be stored in softer subsoil. As a result, the discharge in the river rises significantly during heavy rainfall. However during a dry period the discharge can be very low as can be seen in Figure 1.4c.

Another notable occurrence is the difference between high and low water at the river mouth. Because of the local bathymetry and large tidal variation of 5 m in the bay, large parts of rock emerge from the water during low water. These differences can easily be seen in Figure 1.4a (during high water) and 1.4b (during low water). As a result the river has eroded a scour channel along the shoreline from East to West.

(a) Rio Matasnillo during high water **(b)** Rio Matasnillo during low water **(c)** Conditions Rio Matasnillo

Figure 1.4: Conditions for Rio Matasnillo

Pollution
The biggest problem of Matasnillo river is the water quality. In a report by the Ministry of Environment of Panama in 2016 the contamination was labelled to be severe. Another report by the Panama Water Resources Authority calculated that the number of coliforms were at 5 million units/100ml, in contrast to a tolerable of 500 units/100 ml. Furthermore the river exceeds the tolerable limit for biochemical oxygen demand (114 vs 35 mg/l tolerable), nitrogen (19 vs 10 mg/l) and phosphorus (7 vs 5 mg/l tolerable) and scores second most polluted river in Panama (Burón-Barahona, 2016).

The high level of pollution is a combination of industrial waste, fecal matter and domestic garbage which is dumped into the river. Over the last decade large improvements have been made under the Sanitation project, but the challenge remains to reduce the levels of pollution and smell at the river mouth further to tolerable limits (Burón-Barahona, 2016). Therefore the water is still unfit for consumption and recreation.

1.2.2 Cinta Costera green zone

The existing green zone along the Cinta Costera provides a nice get-away from the busy city life. Though in its current state it is only a small stretch between the waterfront and the city. The Avenida Balboa highway runs trough the green zone, making it both harder to reach and less relaxing because of the heavy traffic. If the green zone can be extended and easier to reach from the city it will be more accessible and increase its recreational value.

1.2.3 Accessibility & connectivity

Before the technical details of the area are investigated, first the social-economical value of the area is looked at. Along the Cinta Costera large hotels can be found and most of the rooms concentrate in the area between the marina and Matasnillo river (Boskalis/Okra, 2018), see Figure 1.5. In fact 55% of the lodging capacity concentrates just before mouth of Matasnillo river. Furthermore, on the peninsula of Punta Paitilla, a lot of business centers are located.

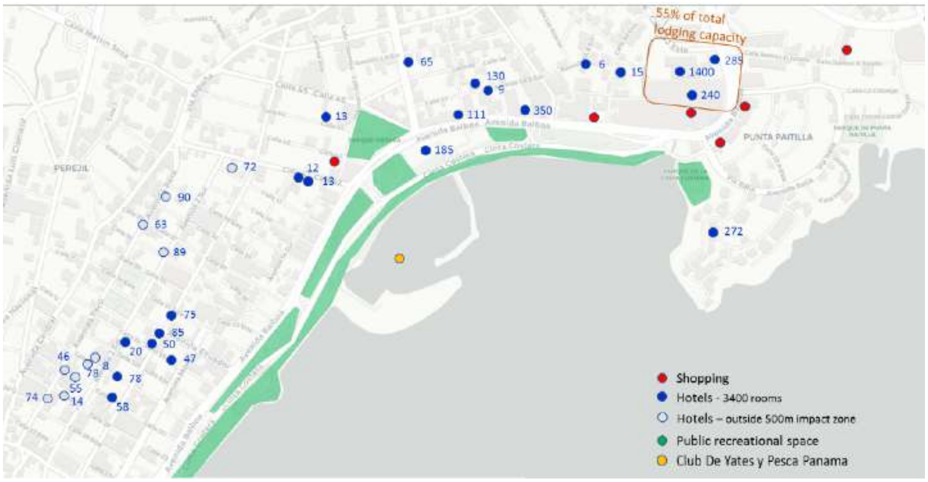


Figure 1.5: Overview of hotels and rooms at Cinta Costera (Boskalis/OKRA, 2018).

At the moment the busy highway and intersection of Matasnillo river with the 55% lodging area and business centers acts as a barrier for tourists and residents to access the Green zone at Cinta Costera, although the distance is only 200 meters. If the proposed recreational zone can be extended to reach the peninsula of punta Paitilla it would create more value and decrease the distance to reach the beach and Green zone by tourists and residents in the area. This will imply that Matasnillo river has to be diverted under the recreational zone.

Not only the distance and accessibility can be reasons for people to avoid the area but also the conditions caused by the water quality. From Section 1.2.1 it became clear that the water is highly polluted and causes an odour problem. It will be unpleasant for people to cross the river in these conditions. Possible solutions for this problem can be either to purify the water and thus remove the source of the problems, or to cover and create an outfall structure.

1.2.4 Lack of beach

In the past it was common to see beaches located along the waterfront of Panama City until 1940 (Saldana, 2019), see Figure 1.6a and the older generation of Panama City can still remember these beaches crowded with people. However, these beaches were reclaimed when the rapid expansion of the city began for the construction of many high rise buildings and a highway along the coast as can be seen in Figure 1.6b. At the moment the closest public beach access is at a distance of 20 and 60 km from Panama City and not easy to reach for citizens that are less mobile. Therefore the wish for a new, closer, beach is still strong to escape the busy city life.

(a) Historic beach before city expansion

(b) Current situation along Cinta Costera (Boskalis, 2018)

Beach development

To improve the potential of the city and meet the wishes of the citizens new plans for improving the Cinta Costera were considered by the government. In response Boskalis proposed an idea in which the creation of a city beach will serve as the main improvement to the area. The proposal consists of two beaches separated by the marina that lays in the middle of the bay. To the West of the marina Caldonia beach will be created, see Figure 1.7a and to the East Bella Vista beach, see Figure 1.7b. Both beaches will have a 50 meter wide commercial strip that provides space for restaurants, beach clubs, etc.

To maximize the reach and use of Bella Vista beach it is desirable that it reaches all the way to the peninsula of Punta Paitilla on the Eastern side of the bay. However, the location of Rio Matasnillo will make this difficult because it separates the Cinta Costera and Punta Paitilla. The beach can only be realized if a solution is provided that guarantees the outflow of Matasnillo during peak discharge conditions, such that no unwanted floodings occur.

(a) Caldonia beach

(b) Bella Vista beach

Figure 1.7: Artist impression of beach creation along Cinta Costera (Boskalis/Okra, 2019)

1.3 Stakeholder analysis

The stakeholders which are involved play an important factor as well. In the end the client will decide whether the project will be a go or no go, but the stakeholders can also exert their influence throughout the project. The following stakeholders have been considered:

- Panama City government
- Citizens
- Hotel owners and commercial facilities
- Tourists

Panama City municipality
Besides the President the municipality of Panama City will have an important vote in the final decision as well. Next to the President of Panama the mayor of Panama City will win the citizens if improvements to the area are made. At this level funding and planning will play a more important role. A good plan can convince and satisfy the city government to win the contract.

Citizens
The citizens of Panama City will have an important vote as well. After all it is their city and they will make use of it. Therefore it is important to know which improvements the general public wishes to see, such that the project will be embraced by the public rather than rejected.

Hotel owners & commercial facilities
Along the Cinta Costera a lot of hotels can be found. For them an improvement of the area can lead to a significant increase of value and income. When involved in the decision making these parties might also contribute to the investment costs. When completed the area will have more potential for commercial facilities as well. In time they can repay a part of the investment costs.

Tourists
While tourists are not directly involved in the decision making, they are an important stakeholder to take into account, because they are an important source of income. An attractive area will please the tourists and improve the reputation of the city. More tourists will lead to higher profits.

The stakeholders will be used to make a final decision between different designs. In the first place the client (this will be the municipality of Panama City) will have some requirements that a design must meet. This program of requirements will include both functional and structural requirements. After this stage the interests and wishes of the other stakeholders will be included in a multi-criteria analysis to come up with a final solution.

1.4 Problem description

At the moment the overall waterfront along Costa Cintera has potential to support recreation and possible commercial facilities. The lack of beach and accessibility discourages local residents and tourists to visit the area.

The biggest problem at hand is the presence of Matasnillo river. The smell from the water is known to cause nausea and thus is highly undesirable at a recreational area. Furthermore, the levels of pollution are that high that in case tourists or locals would engulf the water it could lead to health problems. The Sanitation Project showed that it is difficult to tackle the source of the problems and it can take many years before improvements become noticeable. Therefore another solution has to be found.

A second problem is the state of the marina. Under current conditions the marina has little use, due to the inaccessibility during low water, bad use of available space and dilapidated breakwaters. As a result the recreational opportunities and economical revenues are low.

1.5 Objective

With the problem description and background information the aim and objective of this project can now be formulated. In order for the project to be viable the biggest problem, caused by Matasnillo river and the bad condition of the marina, needs to be solved. Therefore the objective can be stated as follows:

Come up with a solution for the nuisance caused by odour and poor water quality from Matasnillo river, such that the water quality along the Cinta Costera area in Panama City is sufficient for recreational use.

This Msc thesis will focus on creating a solution for the problems caused by Matasnillo river such that a beach can be realized along the Cinta Costera. The challenge will be to design a concept that fits in an overall masterplan for the area.

1.6 Method

In order to realise the main objectives the problem can be subdivided in multiple parts and translated into smaller tasks, following the engineering design cycle shown in Figure 1.8. Each design step will be further explained.

- **Analysis**
  From the problem analysis the objective of the design is defined after an exploration of the problem. With this objective the program of requirements and boundary conditions can be formulated.
- **Synthesis**
  Come up with several concept solutions that can accomplish the objective. In this stage no restrictions should limit the creativity of possible designs, keeping in mind the boundary conditions and requirements.
- **Simulation**
  The concepts are reviewed based on the requirements and boundary conditions. When a concept does not meet the requirements it has to be adjusted or dismissed. A combination of different concepts is possible.
- **Evaluation**
  The remaining alternatives are reviewed with a multi-criteria analysis. They all meet the requirements, but in this stage the opinion and wishes of stakeholders and/or client play a role. The value of a design is compared to the costs.
- **Selection**
  From the evaluation the best alternative is selected. This design will be worked-out in further detail.

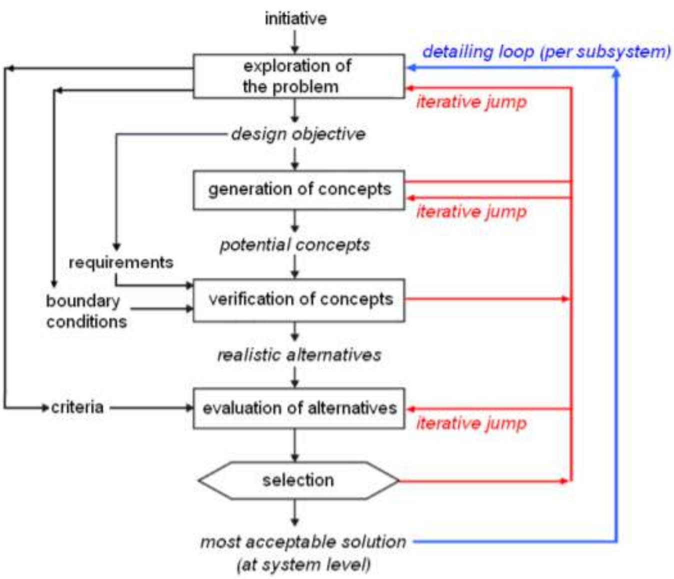


Figure 1.8: Engineering design cycle (Lecture notes Hydraulic structures, 2016).

1.7 Outline of Msc thesis

This Msc thesis follows the layout of the engineering design cycle,see 1.8.

- Chapter 1 gives an introduction and analysis to the problem. The problem analysis is followed by the objective of this project.
- Chapter 2 discusses the requirements and boundary conditions that are set for the project.
- In Chapter 3, several concept designs, that could solve the problems, have been worked out.
- In chapter 4 the concept designs are evaluated and the most promising designs have been worked out in more detail such that they fit the functional requirements.
- Chapter 5 shows the positioning and functional design of the hybrid concept.
- In Chapter 6 the structural design of the hybrid concept is made to determine the necessary dimensions and reinforcement to meet the structural requirements.
- In Chapter 7 a final verification is made on the two design variants of the hybrid concept. Based on the construction time, costs and flood risk a final decision between these two variants was made.
- Chapter 8 shows the final design and some recommendations for further improvements are given.

Chapter 2. Requirements & Boundary conditions

After the area was analyzed and the problem is clearly formulated, the requirements and boundary conditions that the design must meet could be determined. The required data was retrieved during a visit to Panama City from 7^{th} January 2019 till 7^{th} February 2019, under supervision of the Boskalis Panama department. In this chapter the requirements and boundary conditions, needed for concept design, are collected and explained.

2.1 Functional requirements

The functional requirements follow from the objective and define what function a concept should at least accomplish. In this case the functional requirements will be:

- Water quality in the bay has to meet standards to be fit for recreational use. For the most import substances the tolerable limits are given in Table 2.1

Parameter	Abbreviation	Unit	Tolerable limit
Dissolved oxygen	DO	mg/l	>8 (water >25° C)
Total dissolved solids	TDS	mg/l	<500
Turbidity	TU	NTU	<100
PH	PH		6.5-8.5
Oils and fats	O&F	mg/l	<10
Biological oxygen demand	BOD5	mg/l	<5
Chemical oxygen demand	COD	mg/l	<200
Phosphorous	PT	mg/l	<0.18
Ammoniacal Nitrogen	NH_4	mg/l	<2
Nitrates	NO_3	mg/l	<10
Total coliforms	CT	UFC/100 ml	<500
Fecal coliforms	CF	UFC/100 ml	<250
Fecal E. Coli	EF	UFC/100 ml	<250

Table 2.1: Tolerable limits for several important water quality paramaters (, 2019).

- During rainy season the river must have sufficient capacity to discharge the water and will not cause floodings. Using rain data in combination with the rational method the maximum discharge is expected to be 150 m^3/s for a 20 year return period. A detailed explanation of this method can be found in Section 2.3.3.
- Remove the odour that originates from the river area.

2.2 Structural requirements

The structural requirements make sure that a concept does not fail and can withstand the forces acting on it. The following structural requirements are taken into account:

- 20 year design life
- Constructability
- Stability
- Strength

2.3 Boundary conditions

The conceptual design phase should be a creative process in which the creativity is not restricted. However, each possible design should be able to meet the requirements or conditions for which the structure is intended. So before different concepts are worked out some information on the following boundary conditions were collected during the Panama visit:

- Beach profile
- River dimensions & infrastructure
- Hydraulic conditions (River discharge, water levels, flow speed)
- Water quality
- Bathymetry
- Soil conditions

2.3.1 Beach profile

The creation of a beach along the Cinta Costera will be the prime improvement to the area. For commercial purposes it was desired that a flat stretch of 50 meter of dry beach will be present at all times. Another requirement was to have a continuous beach from the marina in the East to Punta Paitilla in the West. The proposed beach profile that fulfills these requirements was made by the Morphology & Marine Environment department of Boskalis using a UNIBEST morphological model (Hendriks, 2019). The results of this model can be seen in 2.9 and a horizontal cross-section DP5 of the profile in 2.10. The yellow line represents the equilibrium situation according to the modeling results and the red line the beach profile with a minimum 50 meter dry width. After the 50 meter dry-zone a sloping section of 120 meter with a 1:20 slope towards the mean sea water level will be present, followed by a 1:30 slope to the low mean sea water level. The yellow line attached to the existing breakwater of the marina is an extension of that breakwater in order to prevent sediment transport and siltation of the marina.

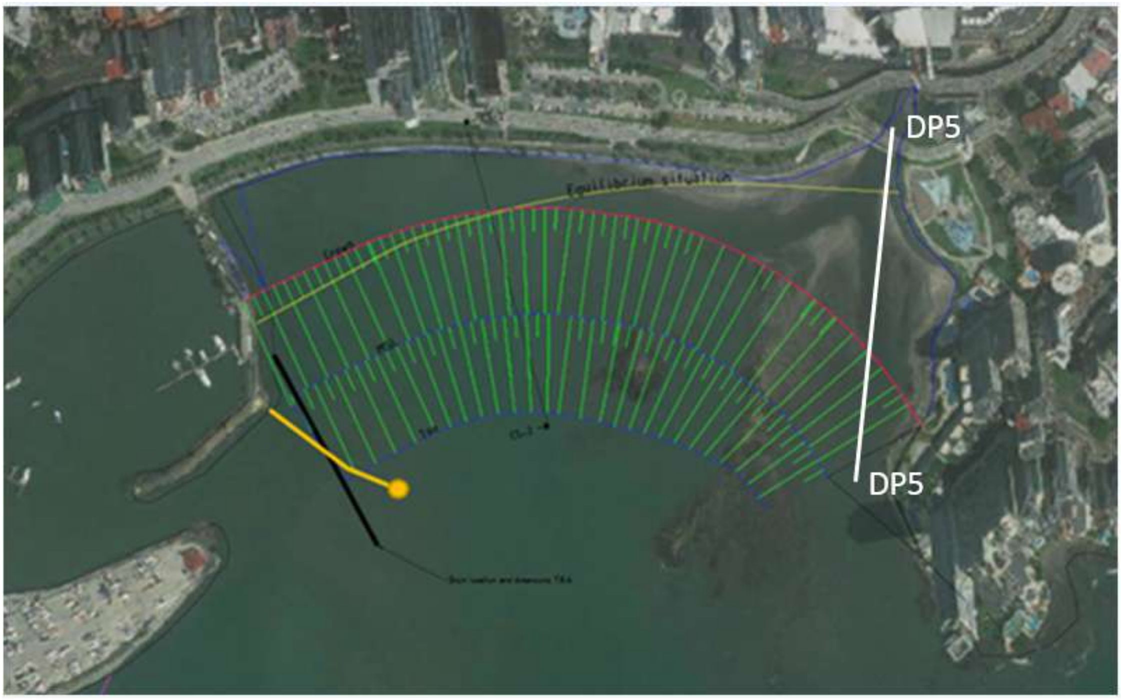


Figure 2.9: Intended beach profile (Boskalis, 2019).

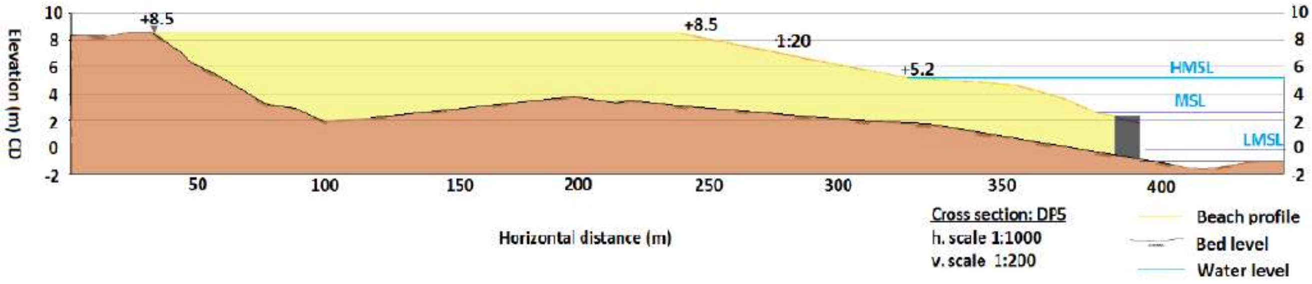


Figure 2.10: Intended beach profile, cross-section DP5 (Boskalis, 2019).

2.3.2 River dimensions & infrastructure

The existing infrastructure and river dimensions are important boundary conditions for the design. It was not possible to retrieve any existing information on the infrastructure near the mouth of Matasnillo river, therefore some measurements were done. During low water at spring tide on the 22^{nd} of January 2019, the dimensions of the river (width, height), distances to existing infrastructure and the GPS altitudes at two locations in the river were measured. The locations at which the measurements were done can be seen in Figure 2.11. The area was not easily accessible and the water level in the river was still 0.5 meter despite the low tide. Therefore some measurement errors are likely to have occurred and will be discussed for each topic. For more information about the measurement methods see Appendix B.

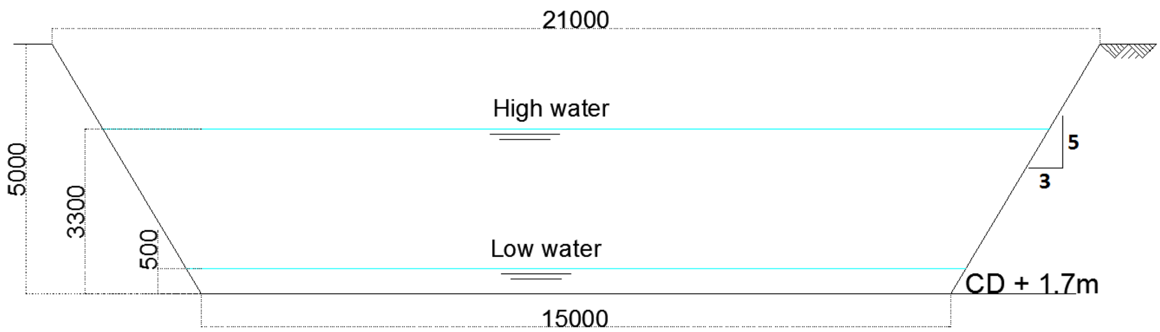
Figure 2.11: Location of measurements taken at Matasnillo river (Google Maps, 2019)

River dimensions

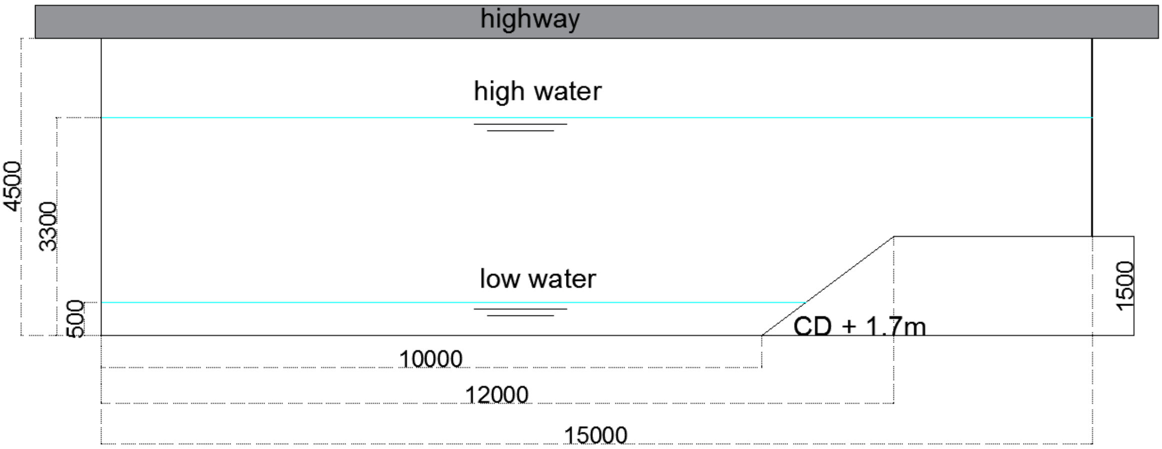
The river dimensions are measured at two locations near the river mouth. The most Northern line (cross-section A-A) represents a general cross-section of the river and is taken for reference of the lower reach of the river from the Hard Rock hotel to the river mouth. The dimensions of this section can be seen in 2.12a. The Southern line (cross-section B-B) represents a cross-section where the river is limited by existing infrastructure and is most narrow, see Figure 2.12b. The indicated water levels correspond to high and low tide, but can be different in case of rainfall and high discharge in the river.

Infrastructure

The existing infrastructure at the mouth of Matasnillo river restricts the available space both in horizontal and vertical direction. As can be seen in Figure 2.11 a number of highways and connections between buildings cross the river (red lines). Two locations are of most interest. At cross-section 1-1 a sky bridge is located that connects a parking garage at the East bank to a Business center at the West bank. At cross-section 2-2 a highway crosses the river and this location coincides with cross-section B-B and is incorporated in Figure 2.12b. These cross-sections indicate the limiting heights, measured from the bottom of the river, and widths



(a) Cross-section A-A, general river profile



(b) Cross-section B-B, narrowest section

Figure 2.12: Dimensions of two cross-sections in lower reach of Matasnillo river (in mm)

and are listed in table 2.2.

Cross-section	Limiting width (m)	Limiting height (m)
1-1 Skybridge	22	5,5
2-2 Highway	10-15	4,5

Table 2.2: Limiting heights and widths of existing infrastructures

Topography

The topography of the river is mapped using a high quality differential GPS measuring station. In Figure 2.11 the location of the GPS measuring points are indicated with A and B. Location A is at a confluence of the river and location B is at the river mouth. It was intended to have some more measuring points along the river but due to a visit of the pope in the week when the GPS survey was done half the city was closed off or hard to reach. The high rise buildings along the river also block the signal making it hard to get good readings. The results can be seen in Table 2.3.

Location	Latitude, Longitude	Altitude (m)
A	Lat: 8.992508 Long: -79.518074	8.23
B	Lat: 8.974925 Long: -79.518304	2.20

Table 2.3: Results of GPS reading at two locations along Matasnillo river

From the results in Table 2.3 it can be concluded that the decline of the river is 6.03 m between point A and B. To determine the slope the length between these point is needed. This is done using the *Measure distance* option in Google Maps. This resulted in a length of 2700 m. Now the slope of the river between point A and B can be calculated using the following equation:

$$i = \frac{\delta h}{\delta l} = \frac{6.03}{2700} = 0.0022\ (m/m) \tag{2.1}$$

Measurement errors
At some locations under the highway crossing it was hard to measure the lengths, widths etc. as was discussed in more detail in Appendix B. Because of these conditions it was hard to do accurate measurements. It was easier to measure the heights at the given locations in 2.12 than the width. Therefore it can be expected that a measurement error of 250 mm in width and 100 mm in height may have occurred.

The GPS measurements could be done very accurate, because the device had a good connection to several satellites. In these conditions the accuracy is in the order of centimeters. Because the measured altitude is used to determine the slope over a length of several kilometers, the inaccuracy of several centimeters is negligible.

2.3.3 River discharge

During the visit to Panama it was hoped to retrieve data about the hydraulic conditions in Matasnillo river. Unfortunately, this was not available. Therefore, other methods were used to get some information on river discharge, water levels and flow speed.

One of the most important criteria for the design is the governing discharge in the river. The design has to be able to discharge the water during peak conditions. Since the river has its origin inside the city most of the discharge is rain run-off. Therefore, during dry periods, the discharge is low. However, during Panamas rainy season, which lasts from May till November (WorldBankGroup, 2016), the discharge can be much higher due to heavy rainstorms. Tropical rainstorms often release large quantities of rain locally and during a short time period. Because of this high intensity and fast run-off times (due to the paved environment in Panama City) the discharge in the river can change a lot in a small time period in response to a rainstorm.

Rational method
A method often used in the field of hydrology to estimate river discharge in urban areas is the rational method. The rational method relates the potential discharge in the river to the average intensity of rainfall over a length of time, a runoff coefficient and the drainage area of the river. The formula for this method is given by (Savenije, 2010):

$$Q = C * i * A \tag{2.2}$$

In which:

$$\begin{aligned} Q &= \text{Design discharge } (\mathrm{m^3/s}) \\ C &= \text{Runoff coefficient } (-) \\ i &= \text{Rainfall intensity } (\mathrm{m/s}) \\ A &= \text{Drainage area } (\mathrm{m^2}) \end{aligned}$$

To calculate the governing discharge in Matasnillo river the three variables of Equation 2.2 need to be determined. Each variable will be discussed separately in the following sections.

Runoff coefficient

Not all the rain that falls in the drainage area ends up in the river. Due to storage, interception, infiltration a part of the water is trapped and can't reach the river as runoff. This effect is taken into account in Formula 2.2 by the runoff coefficient C. This coefficient is a dimensionless ratio that indicates the fraction of rainfall converted to runoff. It depends greatly on the type of land that is considered. In rural areas where the rain can penetrate into the ground the runoff is small and therefore the coefficient is low. Panama City is densely built with high rise buildings and roads. This will result in a high runoff coefficient. In Table 2.4 values of runoff coefficients for several types of land-use are given (Thompson, 2006). Within the boundaries of the drainage area several of these land-use types occur. Therefore, the runoff coefficient for this area is approximated by taking an average of the most occurring types, which are: Roofs, Asphalt, drives and walks and neighborhood areas. As a result the runoff coefficient would be estimated to be 0.8. However, this would lead to an overestimation of the runoff coefficient because many buildings in the area are also connected to the wastewater sewer. A part of the collected rainwater is thus discharged to the sewer instead of the river. It is difficult to put an exact percentage on the water that is discharge through the wastewater sewer, because at this point not much is known about the existing connections. Therefore, a conservative estimate of 20% is made. This leads to a final runoff coefficient for the area of 0.6.

Description	Runoff Coefficient
Business	
Downtown Areas	0.70-0.95
Neighborhood Areas	0.50-0.70
Residential	
Residential suburban	0.25-0.40
Parks, playgrounds	0.10-0.35
Drives and walks	0.75-0.85
Roofs	0.75-0.95
Streets	
Asphalt	0.70-0.95
Concrete	0.80-0.95
Brick	0.70-0.85

Table 2.4: Values of runoff coefficient for different land use (Thompson, 2006).

Rainfall intensity

The third variable of Equation 2.2 is the rainfall intensity i. During the rainy season rainstorms with different intensity and duration can occur. Therefore, in the design of hydrologic and hydraulic structures, most often Intensity-Duration-Frequency (IDF) curves are used. These curves describe the relationship between rainfall intensity, rainfall duration and return period and are developed using historical precipitation data fitted to a probability distribution (Aart Overeem, 2007). The rain data needed for this procedure was retrieved from the the ACP (Meteorology and Hydrology Branch, Panama Canal Authority, Republic of Panama) (Authority, 2019). The ACP monitors a network of 60 stations throughout the Panama Canal watershed. The station which was chosen for this research was located at Balboa Heights and is located 6 km to the West of the mouth of Matasnillo river. The retrieved data-set consists of measurements from a rain gauge from 1993 to 2018 (data was missing for the years 2002, 2007, 2008 and incomplete during 2017,2018). During this period a measurement was sent each 15 minutes.

For each year the annual maxima rainfall intensities were extracted from this data-set for time durations of 15, 30, 45, 60, 75 and 90 minutes. The results of this process can be seen in Appendix B.2. The data was then analyzed by using extreme value analysis for block maximum, in order to retrieve useful rainfall intensities for different return periods. The extreme value analysis was performed by using the generalized extreme value (GEV) distribution. The GEV distribution is a family of continuous probability distributions to combine the Gumbel, Frechet and Weibull (type I, II and III, respectively) extreme value distributions,to allow a continuous range of possible shapes . Each of these distribution has its own characteristic shape as can be seen in Figure 2.13. The cumulative distribution function of the GEV is given by Equation B.1.

$$F(x) = exp\left(-\left(1+\xi\left(\frac{x-\mu}{\sigma}\right)\right)^{\frac{-1}{\xi}}\right) \tag{2.3}$$

In which:

μ = Location paramater
σ = Scale parameter
ξ = Shape parameter

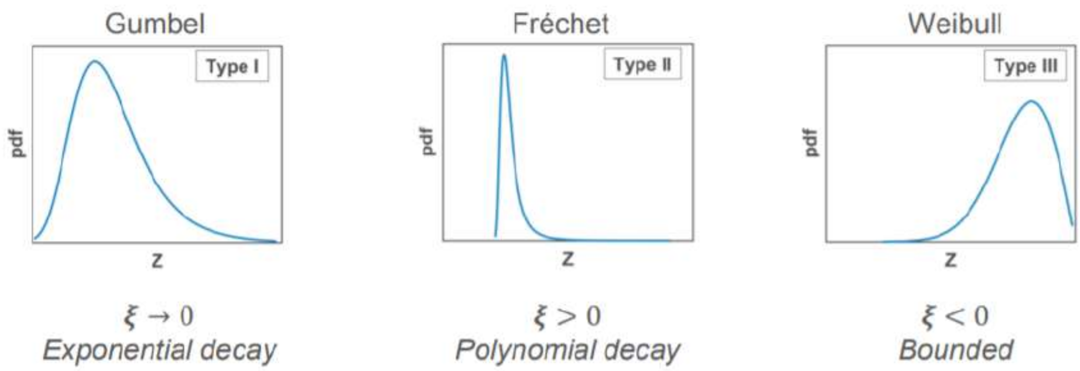


Figure 2.13: Shapes of the three different extreme value distributions

The GEV distribution is parameterized with a location, scale and shape parameters, μ, σ and ξ respectively. The shape parameter ξ is governing for the tail behavior. For $\xi = 0$, $\xi > 0$, $\xi < 0$, the GEV leads to the Gumbel, Frechet and Weibull distribution, respectively. The parameters ξ, σ, μ were determined for each time duration of 15, 30, 45, 60, 75 and 90 minutes using the corresponding maximum annual rainfall data with 21 years of data. This was done by using a function in Matlab called *gevfit*. The input for this function is the data set of maximum annual rainfall intensity for a certain time duration. As output the function returns the variables for ξ, σ, μ that best fit the data set. A more elaborated description of the *gevfit* function can be found in Appendix B.2. The results of the computations are presented in Table 2.5.

Duration (min)	ξ shape	σ scale	μ location
15	-0.0477	15.9336	107.9027
30	-0.4599	36.2832	70.6413
45	-0.2088	16.1452	69.2799
60	-0.0387	12.2268	59.3800
75	-0.0031	10.0856	52.0714
90	-0.0339	9.8503	45.5979

Table 2.5: Results for shape (ξ), scale (σ) and location (μ) parameters for different time durations.

The IDF curves could now be constructed for different return periods T. The rainfall intensity with a probability p of being exceeded is associated with the return period T. This value of p is defined by $p = 1/T$. To define the rainfall intensity for a given return period at each duration the inverse of equation B.1 is used, see Equation 2.4. The data contained 21 years of data, therefore it was decided to compute IDF curves for return periods up to 20 years. The results of the computations can be seen in Figure 2.14.

$$I(d) = \begin{cases} \mu - \frac{\sigma}{\xi}\left(1-(-\log(1-p))^{-\xi}\right) & \text{if } \xi \neq 0 \\ \mu - \sigma\log(-\log(1-p)) & \text{if } \xi = 0 \end{cases} \tag{2.4}$$

In which:

$$I(d) = \text{Rainfall intensity at time duration d}$$
$$p = \text{Exceedence probability } (1/\text{T})$$

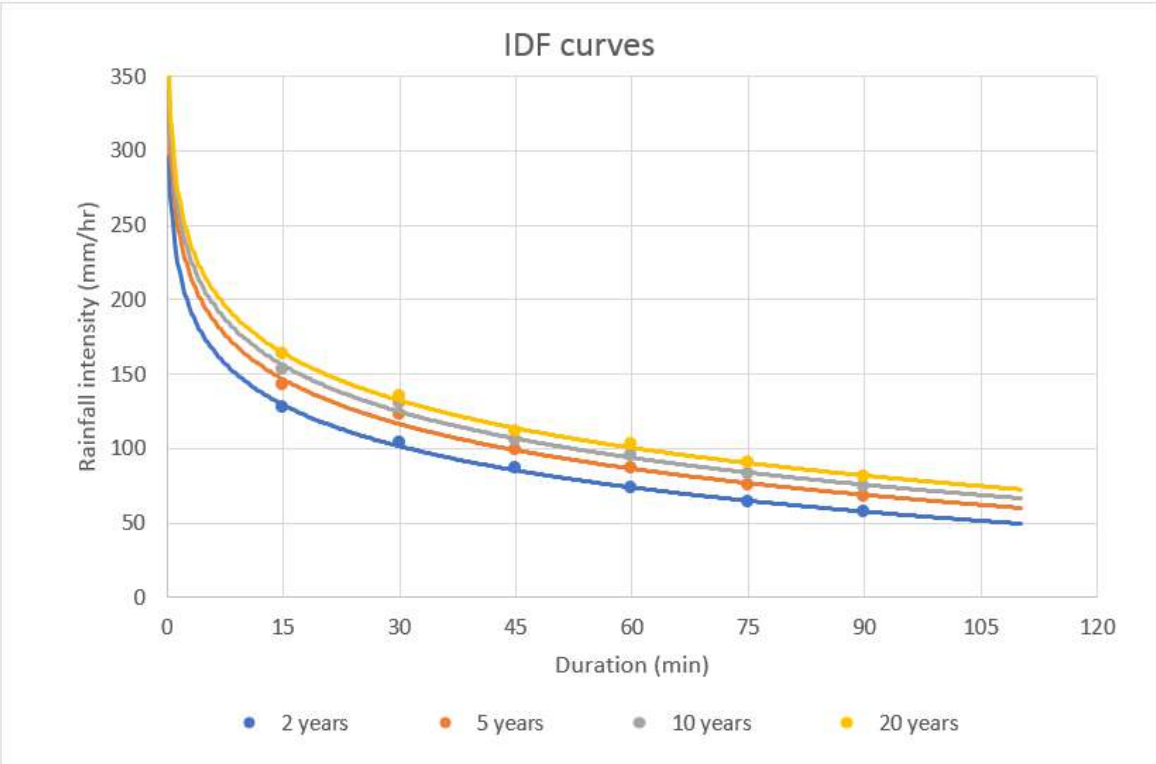


Figure 2.14: Rainfall Intensity-Duration-Frequency curves for return periods of 2, 5, 10 and 20 years.

Drainage area

Next the drainage area A of Matasnillo river is determined. This is done based on the height profile of the city at the area around Matasnillo river and by looking at the other rivers nearby. The outer contours of the drainage area can be found by locating the highest locations in the area, since water naturally flows from high to low. This is done using Google Earth, which shows the elevation at each location. In Figure 2.15 the drainage area is presented with the green area. This area has a total size of 8.4 km^2. Inside the green area the course of Matasnillo river can be seen. The river has three main branches which collect the rainwater directly from surface runoff and partly through stormwater sewer pipes.

Time of concentration

A final important concept in the rational method is the time of concentration t_c. This time unit is defined as the time required for water runoff to travel from the most hydraulically distant part in the drainage area to the outlet. So when the time of concentration is reached the entire drainage area contributes to the discharge at the outlet. If a rainstorm duration is smaller than t_c, then the drainage area is not fully contributing runoff to the outlet and the peak discharge will not be reached for that rainstorm intensity. To time of concentration for this drainage area was estimated by an assumption of the flow speed. The flow speed was estimated based on observations by engineers from a cooperating company. According to their findings the flow speed is on average 1.5 m/s during rainstorms. The most hydraulically distant part from the river mouth is at approximately 5.4 km. Therefore, the time of concentration is estimated to be 60 minutes.

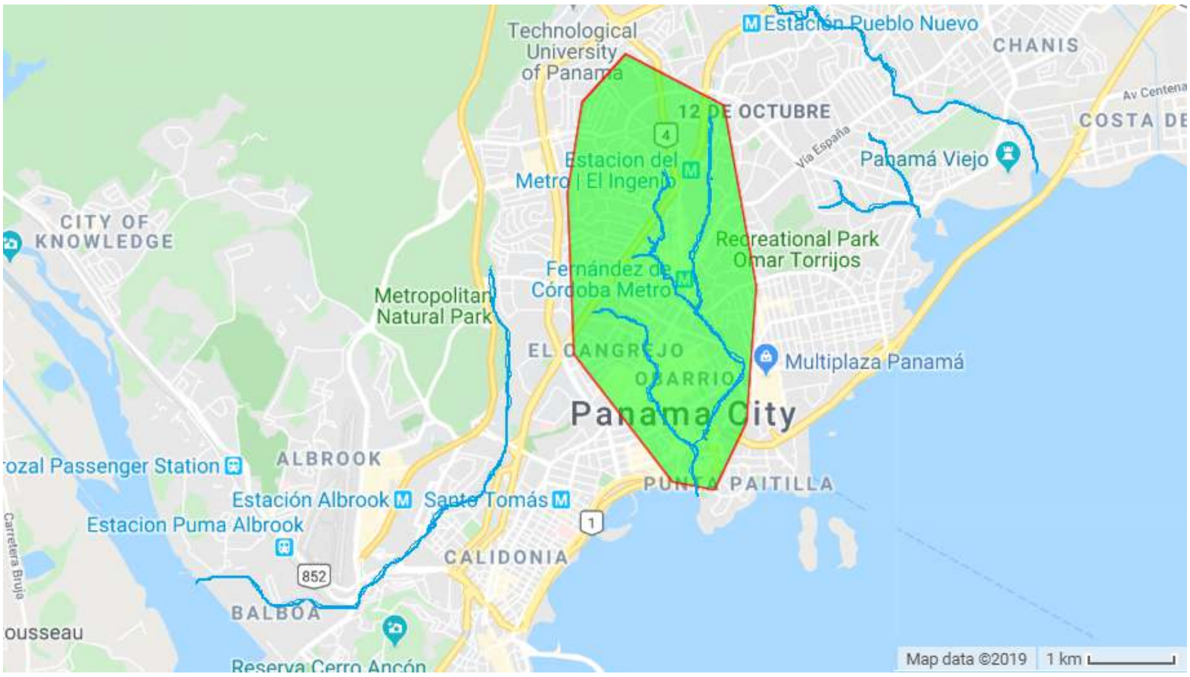


Figure 2.15: Drainage area for Matasnillo river (Google Maps, 2019).

Discharge calculations

The discharge can now be determined based on the different variables and by using the IDF curves for the different return periods. The time of concentration was estimated to be 60 minutes. For this duration the rainfall intensity was read from the IDF curves. Finally, by using Equation 2.2, in which the runoff coefficient C was estimated to be 0.6 and the drainage area A is 8.4 km^2, the discharge can be computed and is presented in Table 2.6. The discharge is presented with units of m^3/s, so a transformation from the units of rainfall intensity (mm/hr) was made.

Return Period (years)	2	5	10	20
Rainfall intensity (mm/hr)	74	87	95	103
Discharge (m^3/s)	102	121	130	144

Table 2.6: Maximum river discharge for return periods of 2, 5, 10 and 20 years.

To check whether the approximation of 60 minutes is correct, the resulting flow speed of the expected discharge is compared to the observed estimated flow speed. From Figure 2.12 the governing cross-sectional area was determined to be 80 m^2. This results in a flow speed of 1.8 m/s for a discharge of 144 m^3/s with a 20 year return period. Since the observed flow speed was in the order of 1.5 m/s and not properly measured, the approximation of 60 minutes seems to be reasonable. In case the results do not match a different time of concentration has to be chosen, resulting in a higher discharge for smaller durations and lower discharge for longer durations.

Discharge under dry conditions

Under dry conditions and low water level the discharge in Matasnillo river was estimated by observing the flow speed and the water level in the river. The water level inside the river at this time was 0.5 m. Multiplied with the width of the river (15 m) this results in an area of 7.5. The flow speed was measured by taking the time an object in the water took to travel 1 meter. This resulted in a flow speed of 0.2 m/s. From these parameters the discharge under dry conditions was calculated to be 1.5 m^3/s

2.3.4 Water levels

As was the case for the discharge no measurements exist on water levels inside the river over the year. Therefore most important data on this is retrieved using the water levels of the tide at sea.

Water levels - Bay conditions

From Section 1.2 it became clear that a large tidal variation occurs in the bay at the river mouth and marina. In Figure 2.16 the tide during spring tide is shown in Balboa (entrance Panama canal), which is 5 km to the west of the project location so the tidal range is expected to be the same. The tidal variation at this location ranges from CD +5.68 m to CD -0.75 m and can reach a maximum of 6.51 m. The mean water level is around CD +2.5 m.

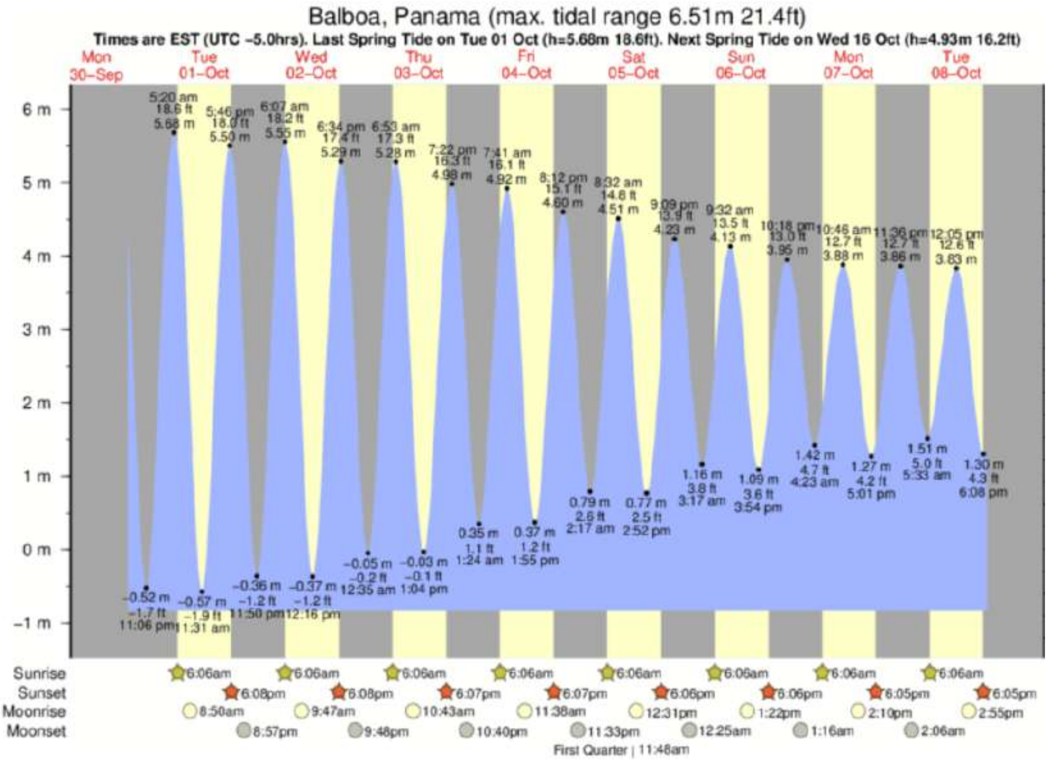


Figure 2.16: Tidal range during spring tide January 2019 (Tide-forecast, 2019).

Water levels - River conditions

During the dry season the discharge in the river is low. When measurements were done to determine the dimensions of river and infrastructure it was observed that near the mouth a water level of 0.5 m was inside the river at low tide. Because the discharge is low the water level inside the river follows the tidal range in the bay.

2.3.5 Bathymetry

As was briefly discussed in Section 2.3.3 the large tidal variation causes large differences in the outflow conditions at the mouth of the river. During low water rock emerges from the water and Matasnillo river discharges along a gulley from East to west. This can easily be seen Figure 2.17, where the bathymetry along the Cinta Costera is shown (for the full bathymetry of Panama City bay see Appendix B.4).The path in which the river water is discharged during low water poses a problem for the proposed recreational area, since the polluted water flows trough the gulley next to the Cinta Costera. This bathymetry was made after a survey done by Boskalis in July 2018. At the top-right corner Matasnillo river enters the bay and the red areas indicate land areas above MSL. This data corresponds with the site survey as can be seen in Appendix A where it can clearly be seen that an area similar in shape is emerged above water level.

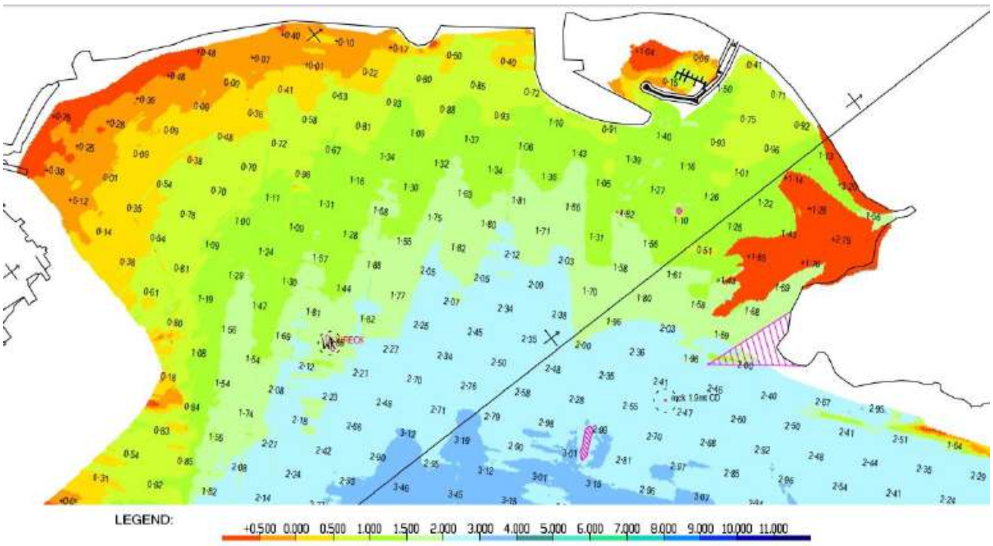


Figure 2.17: Bathymetry along Cinta Costera (Boskalis, 2018).

2.3.6 Water quality

During the rainy season in August 2018 the Sanitation Program carried out several measurements to determine the water quality of the rivers and the bay of Panama City (Programa-saneamiento, 2018). In Figure 2.18 the location of the measurements are shown. During the research the water was tested on the parameters that are listed in Table 2.1. These parameters are tested to the tolerable limits established in Executive Decree 75 of June 4 2008 by the Panama Maritime Authority, which regulates the quality of bodies of water with or without direct contact with human beings. For more information on classification standards that were used by the sanitation program see Appendix B.3.

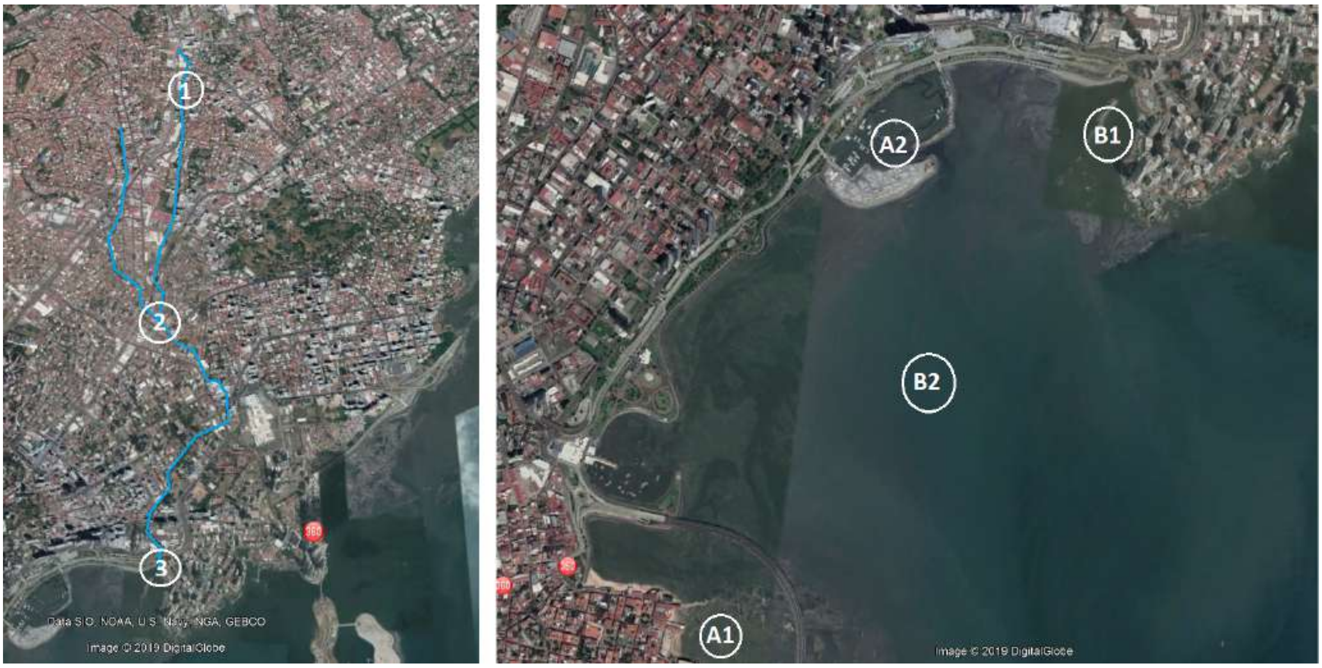


Figure 2.18: Measurement locations of water quality in Matasnillo river and bay in Panama City (Google Earth, 2019).

Matasnillo river

In Matasnillo river three measurements were done at the following locations (location with numbered label can be seen in figure 2.18):

- 1 Near the origin of the river
- 2 At the confluence of two streams of Matasnillo river
- 3 At the mouth of Matasnillo river

Location	Sample date	Atmospheric conditions (% cloudy)	PH	Turbidity (NTU)	T (°C)	Conductivity (µs/cm)	DO (mg/l)	DO (%SAT)	TDS (mg/l)	Redox potential (MV)	STATE
1	6-8-2018	60%	8.2	3.5	28.3	400	3.2	42.1	245.4	-79	BAD
2	6-8-2018	100%	8.2	3.3	28.7	532	0.4	5.4	318.5	-77	BAD
3	22-8-2018	100%	7.9	13.3	28.9	134	0.1	2.0	92.3	-29	BAD

Table 2.7: Results water quality (Sanitation program, 2018).

Location	O&F	HT	BOD5	COD	PT	NH_4	NO_3	NT	SS	S SED	CT	CF	EF	STATE
	mg/l	*mg/l*	*mg/l*	*mg/l*	*mg/l*	*mg/l*	*mg/l*	*mg/l*	*mg/l*	*mg/l*	*CFU/ml*	*CFU/ml*	*CFU/ml*	
1	<10	<0.05	<3	<5	0.18	<0.01	2.8	14.8	<5	2.2	4700	650	5200	BAD
2	<10	<0.05	<3	<5	2.09	11.7	6.8	17.1	31	0.4	111000	37000	13000	BAD
3	<10	<0.05	46.7	88	1.94	8	8.9	18.2	21.3	0.4	16000	8000	10000	BAD

Table 2.8: Results water quality (Sanitation program, 2018).

The results show that the river scores bad in dissolved oxygen (DO), biological oxygen demand (BOD5), fecal coliforms and enterococci (CF, EF). The values of the fecal coliforms and eterococci are that high (8000, 10000 UFC/ml respectively with a tolerable value of 250 UFC/100 ml) that is really unfit for recreational use.

During the survey done under the Sanitation program also a number of heavy metals were measured. However, none of those exceeded tolerable limits except for copper at location 2 upstream of Matasnillo river. The focus lays more on the mouth of Matasnillo river and therefore these results are not included here.

Bay along Cinta Costera
In the bay along Cinta Costera 4 measurements have been done, see Figure 2.18. Two at recreational areas indicated with label A and two at the mouth of Matasnillo and midpoint of the bay indicated with label B. The recreational samples only measured the enterococci and fecal coliforms.

- A1 Old city center of Panama City (Casco Viejo)
- A2 Marina club de yates y pesca de Panama
- B1 Near the river mouth of Rio Matasnillo
- B2 Midpoint between Casco Viejo and Punta Paitilla, bay entrance

Location	Sample date	Salinity (ppm)	PH	Turbidity (NTU)	T (°C)	DO (mg/l)	DO (%SAT)	Redox potential (MV)	STATE
B1	August-2018	30.450	7	14	29.3	5,6	85.7	180	Bad
B2	August-2018	30.700	7.1	15	28.5	6.4	95.8	190	Moderate

Table 2.9: Results water quality (Sanitation program, 2018).

Location	O&F	BOD5	COD	PT	NH_4	NO_3	CT	CF	EF	STATE
	mg/l	*mg/l*	*mg/l*	*mg/l*	*mg/l*	*mg/l*	*UFC/100ml*	*CFU/100ml*	*CFU/100ml*	
A1								300	25	GOOD
A2								70	10	GOOD
B1	<10		<5	<0.02	<0.01	2.5	140	<10	<10	GOOD
B2	<10		<5	0.04	-	-	14	<10	-	GOOD

Table 2.10: Results water quality (Sanitation program, 2018).

From the measurements it can be seen that the water quality in the bay of Panama along the Cinta Costera is actually good compared to the samples taken from Rio Matasnillo. The only parameter that doesn't reach the required standard according to the method of the Sanitation program is the dissolved oxygen. However this parameter is more of influence for aquatic life. Therefore the water quality in the bay is of appropriate quality to be used for recreation.

The large difference between the values of location 3 and B1, which are both near the mouth of Matasnillo river, can be explained by the time at which they are taken. At low tide there is no water at location B1 so no samples can be taken. During high tide clean water enters the bay and up to some point upstream of Matasnillo river. So this water is not yet polluted by the outflow of Matasnillo river.

2.3.7 Soil conditions

Information on soil conditions was retrieved from earlier works done ICA Panama for a land reclamation project in 1998 (ICA-Panama, 1998). When the first ideas to extend Punta Paitilla were worked out a series of drillings in the area were done to map the soil conditions. In the end the whole area was reclaimed and became Punta Pacifica and later, since 2013, work began on two artificial islands, see 2.19. The original drilling location (A-A) is only 1 km to the East of the mouth of Matasnillo, therefore the soil conditions there are expected to be of the same material. In appendix B.5 a drilling can be found along with a cross-section of the then intended islands. From this drilling it is clear that the rocky material that is found on location is a basaltic agglomerate.

Figure 2.19: Location and distance from Matasnillo river of soil conditions (Google Maps, 2019).

Chapter 3. Development of functional design concepts

In the conceptual design phase several solutions that could realize the objective to remove the nuisance caused by Matasnillo river are worked out. During the visit to Panama, a site survey was done to get an idea on the surroundings of the river and to get inspired by the area. The problem was approached by looking at it from different angles in order to offer variety in the range of proposed solutions.

3.1 Hard-engineering solutions

In the field of hard-engineering solutions structures out of concrete and steel are made. These structures often have a large impact on the local environment. At the project location an outfall structure will be an interesting alternative. Such an outfall collects the water and redirects it to another location. Several typical outfall structures exist as can be seen in Figure 3.20. In Figure 3.20a an outfall structure is shown which discharges directly at the waterfront. Figure 3.20b shows an outfall pipeline that is underneath a beach. Finally, Figure 3.20c shows an outfall pipeline extension into the sea. Depending on the requirements each of these examples has its own advantages and disadvantages. For the design of a concept at the project location a combination of several types is also possible.

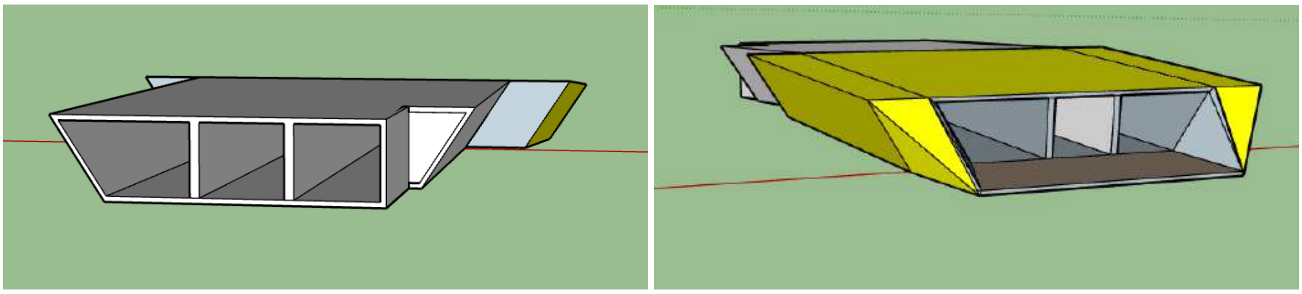
(a) Outfall emerged at waterfront **(b)** Outfall at Rehoboth Beach **(c)** Outfall extension into sea

Figure 3.20: Typical outfall structures (Smith & Cooper, Chris Flood, Marmara diving, 2017).

After the data collection and site survey at the project location some ideas came to mind that could be interesting to work out in further detail. From the site survey it also became clear that a lot of garbage is thrown in the river. This can not only pollute the beach but can also clog the system. So for every design a good working garbage fence should be installed. This will be considered in a later stage. First 3 concepts in the hard-engineering field are presented.

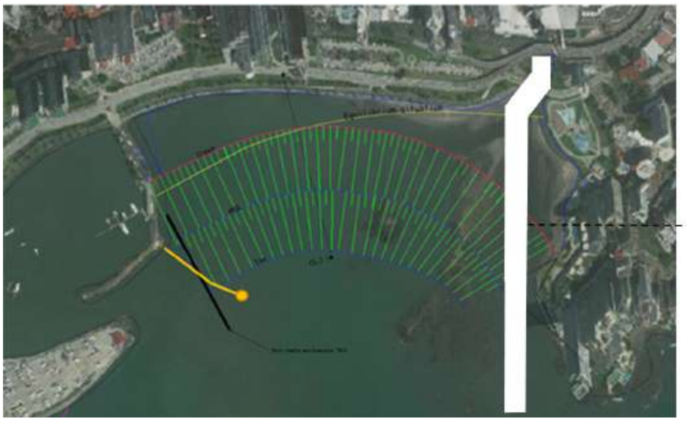
3.1.1 Culvert concept

The first concept is the most straightforward. In this system all the water from Matasnillo river is redirected through a large culvert. The culvert needs to be long enough in order to get the polluted water out of the coastal zone to improve the water quality nearshore. A visual representation of this concept is shown in Figure 3.21. The biggest advantage of this concept is the simplicity. There are no large obstructions that hinder the flow so it should be able to discharge all the water during heavy rainfall. However in this configuration the entire structure needs to be extended to a point where the polluted water can not reduce the water quality below an acceptable level at the beach, see Figure 3.21c. This will make construction more difficult because the culvert structure needs to be installed at deeper sections in the sea. Because the length of the culvert the material costs will also be high.



(a) Basic concept at riverside intake

(b) Basic concept at seaside outlet

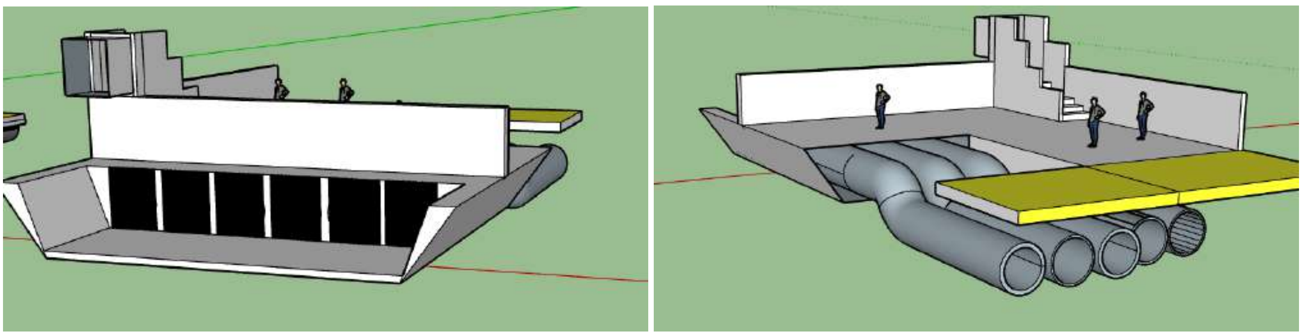


(c) Lay-out at location with beach profile

Figure 3.21: Culvert concept

3.1.2 Pipeline concept

Opposite to the culvert concept a system with pipelines could also be possible. In such a system all the water is collected in an outfall and from there transported through pipelines. A visual representation of this concept is shown in Figure 3.22. Though this is an attractive alternative due to simpler construction and lower costs compared to the culvert concept it will also lead to some difficulties. The effective area which can transport the water will be reduced. When the area is not large enough and the water can't flow through faster it will create floodings upstream of the outfall structure. The culvert part of the outfall will be placed far enough upstream of Matasnillo river such that the smell will not be a problem at the beach. From the culvert structure the pipelines can be extended to a location at which the polluted water will not return to the beach. For the location and configuration of this concept see Figure 3.23.



(a) Pipeline concept at river inlet

(b) Pipeline concept at seaside outlet

Figure 3.22: Pipeline concept

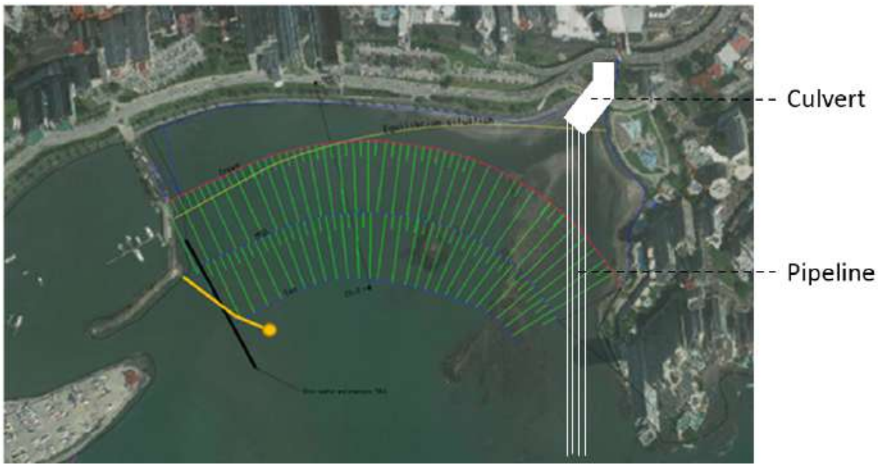
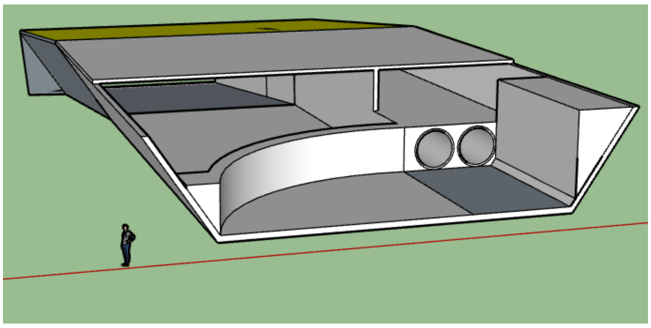


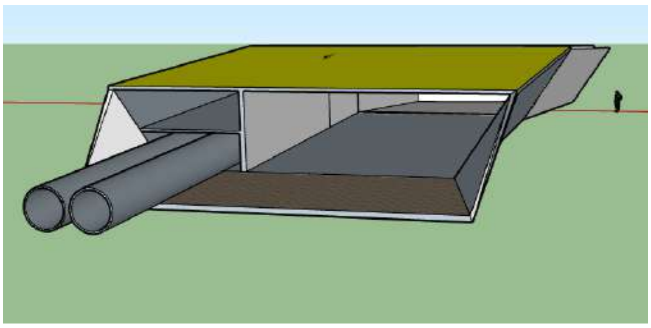
Figure 3.23: Concept 2

3.1.3 Hybrid concept

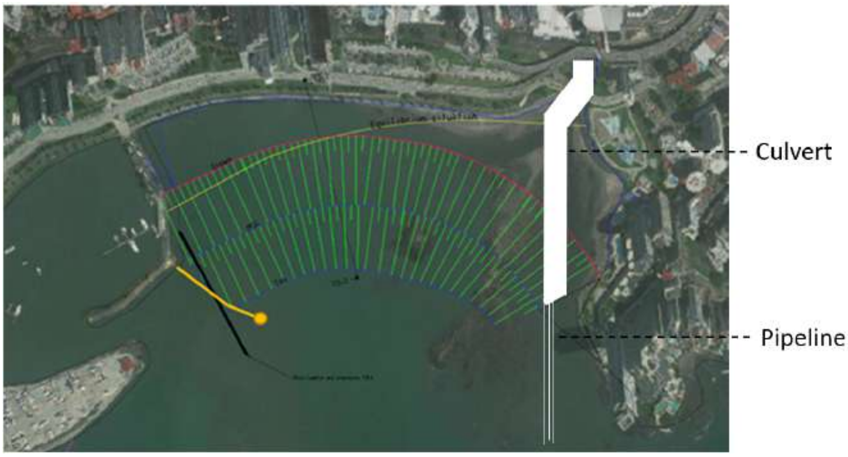
During the site visit it was observed that during the dry season and low water the water level is only 0,5 m and therefore the discharge is low. High discharge capacity is only needed during heavy rainfall in the rainy season. This observation led to a concept in which the previous two concepts are combined, see Figure 3.24. The idea is to collect the polluted river water and redirect it through two pipes with a 2 m diameter (Figure 3.24a) outside the coastal zone under normal conditions. Because of the low discharge there will be need for fewer pipes than in the pipeline concept where all the water needs to flow through the pipes. During heavy rainfall when large capacity is needed, the water can flow over the curved edge through a large culvert that discharges just outside the beach where a breakwater will be located, see Figure 3.24c. The tide can still enter Matasnillo river through the pipe and culvert system and can also be used to flush the pipes.



(a) Hybrid concept at riverside intake



(b) Hybrid concept at seaside outlet



(c) Lay-out at location with beach profile

Figure 3.24: Hybrid concept

3.1.4 Garbage fence

For the outfall concepts it is important that garbage which is thrown in the river does not go through and reaches the ocean. The reason for this is two-fold. First, the garbage that goes through the pipes and reaches the ocean will contribute to the pollution of the ocean. Or, when it is returned to the beach by the currents, it will pollute the recreational area nearby and at the beaches. Secondly, larger pieces of rubble can block the pipes or clog them from the inside which leads to reduction of the water flow capacity.

(a) Grid cleaner at a pumping station in the Netherlands. **(b)** Grid cleaner crane that collects garbage

Figure 3.25: Grid cleaner that collects garbage before it enters a pumping station in the Netherlands (Rlc-roosterreiniger, 2019).

An effective device which can prevent the above mentioned problems is a grid cleaner, see Figure 3.25. These structures consist of two main parts: the fence and the crane. The fence prevents the garbage from entering the pipes or pumps and the crane cleans the fence by grabbing the garbage from the fence. The crane releases the garbage inside a collector bin. This process can be automated to reduce human interaction, but will require some maintenance. In the Netherlands these type of grid cleaner are often used at pumping stations to prevent garbage from clogging the pumps.

For Matasnillo river a suitable location for such a grid cleaner could be at a bridge crossing near the Hardrock hotel, see Figure 3.26. This location is accessible by truck which can empty the collector bin when its full.

Figure 3.26: Possible location for grid cleaner installation near Hardrock hotel (Google Earth, 2019)

3.1.5 Sewage & water treatment

From section 2.3.6 Water quality, it became clear that the largest part of pollutants in the river water are of a fecal type. These enter the water by direct discharge of human sewage and waste from animals. According to the Joint Monitoring Program by UNICEF/WHO, 80% of the urban population in Panama city had access to improved sanitation (meaning that the human waste is separated from human contact) in 2015 (UNICEF/WHO, 2019). From an environmental perspective it would be best that this water is collected and treated before it is discharged into the bay. This can be done by connecting all the buildings (that are currently discharging their waste water in Matasnillo river) to the sewage system. The Sanitation Program is currently working on improving and extending the sewage system to improve sanitation of the inhabitants of Panama City and the water quality of the city's rivers and bay. This is done by installing new collectors, replacing old sewage lines, connecting more houses to the sewage system and by increasing capacity to prevent overflow during heavy rainfall. To be capable to treat the larger waste water capacity the Juan Diaz water treatment plant will be expanded from 250.000 to 475.000 m^3/day by 2020 (Ayesa, 2017).

When all the waste water in the Matasnillo river area is separated from the storm water and the water and odour quality meet the required standards, there will be no need for expensive outfall structures. In this case the river can be integrated in the beach design and follow a path along Punta Paitilla's West shoreline.

3.2 Soft-engineering solutions

Another interesting option is to approach the problem with a soft-engineering solution. Such solutions are often much more environmental friendly compared to the hard-engineering structures by incorporating ecological methods and are therefore often referred by as *Building with nature*.

3.2.1 Ecological floating bed

For contaminated rivers and lakes, such an eco-friendly technology is the integrated ecological floating-bed (IEFB). This technology tries to mimic conventional wetlands by attaching microorganisms, attain nutrients for biomass and entrapment of suspended solids (Li, 2009). In Figure 3.27 a visual representation of such an integrated ecological floating-bed is given. The floating bed needs a minimal depth of 1 m, for the roots and bivalve zone to be able to reach in the water. These kind of structures are often fragile and therefore not suited to withstand high flow speeds.

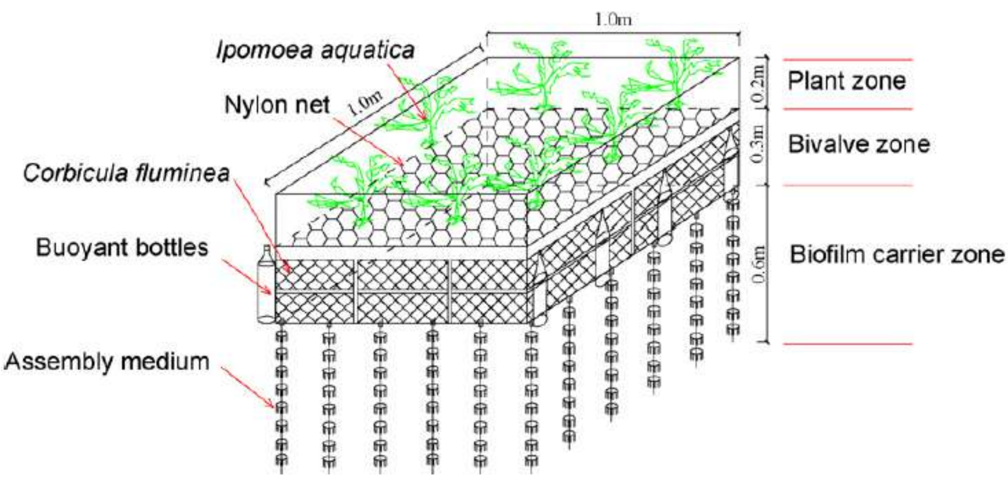


Figure 3.27: Integrated ecological floating-bed (Xian-Ning Li, 2009).

3.2.2 Water retention areas

During a rainstorm most of the water is quickly transported to the river because most of the surface in Panama City is paved. This leads to high peak discharges and thus a large capacity is needed. When this peak can be lowered the capacity can be lower and thus the culvert system can be designed smaller. There exist several innovative measures that can be used to lower the peak discharge such as:

- Water retention basins
- Green areas
- Permeable surfaces
- Rainwater tanks

Water retention basins

A water retention basin is basically a large area that can be flooded during a rainstorm. During the rainstorm the basin fills up and afterwards, the water is released to the nearby river or sewage. The basin can be transformed into a park or sport pitch which can be used under dry conditions. The advantage of these basins is that a lot of water can be stored but it needs a large area which can be difficult in urban areas. Two examples of existing water retention basins can be seen in Figure 3.28.

(a) Clear Creek basin in Atlanta (Fourth Ward Park, 2019). **(b)** Water Squares in Rotterdam (Anneke Bokern, 2015)

Figure 3.28: Two examples of water retention basins in Atlanta & Rotterdam

Green areas

Because Panama City is densely built and most of the area is paved, there is not much room from green areas throughout the city. Green areas reduce the run-off time and hold water for a longer time. Therefore creating parks with trees and grass with a subsoil that can hold the water can help reducing the peak discharge. Such a park can be found at Chulalongkorn University Centenary Park in Bangkok, see Figure 3.29a. However such parks require a lot of space which is not plenty available in Panama City. Therefore other measures can be taken, such as green roofs on top of buildings like at Chicago's City hall, see Figure 3.29b. These will hold some of the rain water at the green roof before it runs off, reducing the peak discharge in the river.

(a) Chulalongkorn urban park of Bangkok (WLA, 2019). **(b)** Green roof Chicago City hall (M.K. McGowan, 2017)

Figure 3.29: Two examples of water retention by green areas in Bangkok & Chicago.

Permeable surfaces

Another way to reduce the runoff time is by using permeable surfaces. These can be of different types and shapes, see Figure 3.30. In figure 3.30a a sidewalk is made with holes in which grass can grow. These holes and grass will hold some of the rainwater before it runs off immediately to the river. Another method is shown in Figure 3.30b. This is a type of permeable asphalt that can absorb a certain amount of water. Next to reduction in runoff time it is also safer to drive, because there will be no aquaplaning due to a layer of water on the surface.

(a) Permeable pavement with grass to increase water absorbance (TreelineHomes, 2016).

(b) Difference in runoff between permeable and non-permeable asphalt (CivilOgistix, 2017)

Figure 3.30: Two examples of permeable surfaces to reduce runoff time.

Rainwater tanks

The use of rainwater tanks can also help to reduce peak discharge. These tanks are installed on top or besides houses and buildings and collect the rainwater that falls on top of the roof, see Figure 3.31. This water can then be used for flushing toilets or showering. On industrial scale it can even be used for air conditioning or cooling of machinery. For existing buildings the plumping will have to be adjusted before the water from the tanks can be used.

(a) Rainwater tank for domestic use (Devonport, 2019).

(b) Rainwater tank for commercial use (Rhinotanks, 2019)

Figure 3.31: Rainwater tanks for domestic and commercial use.

3.3 Non-engineering solutions

Another approach can be to find a solution without constructing structures of both types discussed in the previous sections. This involves actions by governmental institutions and the inhabitants of Panama City to prevent pollution in the first place.

3.3.1 Education

One important measure is to keep the population educated about the effects of their behavior. Currently a lot of garbage such as plastic bags, soda cans and all other sorts of domestic waste can be found inside Matasnillo river (and other rivers in the city). All this waste eventually ends up in the bay, where it pollutes the shoreline and the water itself. Through education programs the government can make the people aware of this and try to improve their behavior. Possible ways in which the government can inform the population to prevent littering can be:

- Giving school lectures to inform children at young age.
- Newspapers with information brought to home.
- Commercials on national television.

- Put a deposit on household items.

3.3.2 Enforcement & regulation

When education proves to be insufficient the government can decide to take more direct measures in the form of enforcement or regulation. This will involve actions that are punishable by law and are applicable to civilians and companies.

The following actions can be made:

- Active monitoring at frequent dumping locations by camera or officials.
- Impose fines to citizens that get caught littering.
- Set maximum standards for waste water of companies that discharge in the river
- Impose fines to companies when they exceed the maximum standards.

Chapter 4. Verification of functional requirements

In this chapter the developed concepts are tested on the functional requirements. At first, a brief evaluation is made to select the concepts that look most promising. Then, the selected concepts are developed such that they meet the functional requirements. After this chapter the remaining concepts must satisfy the functional requirements. These were as follows:

- Water quality in the bay has to meet standards to be fit for recreational use. From the water samples it was clear that the main pollutant in the water of Matasnillo is of fecal origin. For fecal coliforms and E.Coli bacteria the tolerable limit is 250 CFU/100 ml. For the Total coliforms the limit is a maximum of 500 CFU/100 ml.
- During rainy season the river must have sufficient capacity to discharge the water and will not cause floodings. Probabilistic analysis of the rain data resulted in a discharge of 150 m^3/s with a return period of 20 years. This will be the upper limit for the design capacity.
- Remove the odour that originates from the river area.

4.1 Evaluation of concepts

Each of the developed concepts is briefly discussed in the following section. For each concept the advantages and disadvantages, based on the functional requirements, were looked at in order to get an idea of which concepts look most promising. An additional requirement that the beach is uninterrupted is also added to the evaluation. A first, rough indication of the costs has also been made. In the end scores are assigned to each concept and a selection is made.

Culvert concept
With the culvert concept the river is more or less extended under the beach to a location offshore where the polluted water can be safely discharged. By itself it does not doe anything to improve the water quality. In theory the culvert cross-sectional area can be as large as that of the river. The discharge capacity in this concept is therefore guaranteed. The odour that originates from the polluted water will be trapped in the culvert and will probably not cause nuisance at the beach. The height of the culvert can cause problems, both for placement under the beach and along the path offshore. The main disadvantage of this concept will be the costs, since the concrete culvert sections will have to reach all the way to the outlet location.

Pipeline concept
The pipeline concept is similar to the culvert concept in terms that it does not improve the river water quality, but only redirects it to a safe discharge location offshore. The discharge capacity is limited to the width of the river, but the pipelines are easy to place under the beach due to their limited height. The odour will also be trapped in the pipelines. Compared to the culvert concept, the costs are expected to be lower since the pipeline material is cheaper and easier to install.

Hybrid concept
The hybrid concept combines the advantages of the culvert and pipeline concept. In this way it is expected that the discharge capacity is large enough during rainstorms while the polluted water is discharged offshore during dry conditions. However, the placement under the beach could be troublesome in case a large culvert is needed just like in the culvert concept. The costs are expected to be in the same range as the pipeline concept.

Sewage
Connection to the sewage is probably the best solution to solve the water quality and odour problems. However, it might take some time, money and effort to connect all the buildings in the area to the sewage system. Furthermore, the sewage will not have enough capacity to collect high discharges during rainstorms. This means that a combination with a culvert system is needed in order to meet the uninterrupted beach requirement.

Eco-floating bed
The ecological floating bed can improve the water quality and odour in the river in an environmental friendly way. Construction of these floating beds can also be done very cheap compared to the other concepts. However, these floating beds are mainly used in water bodies where there is hardly any current in the water. Therefore, it remains questionable as to what extend the water quality will improve in this river. When no further adjustments are made to the river it will also intersect with the beach.

Water retention measures
The water retention measures are a good way to reduce the peak water discharge during rainstorms. But on its own the water quality and odour in the river will not be solved and the river will still be needed to discharge the water. All these measures combined can also become very costly.

Education & regulation
Education & regulation can be a good way to improve the water quality of the river. However, it might take some time before the water quality meets the tolerable limits. Additionally, the river will remain in its current state when only this measure is taken and the beach will be interrupted by the outflow at the river mouth. The costs of this measure is low and can even produce money when fines are imposed to companies that discharge polluted water into the river.

4.1.1 Ranking of concepts

A ranking was made based on the evaluation in the previous section in order to decide which concepts are most viable. The concepts are assessed against each other and based on engineering judgment. The ranking was done by assigning scores for each concept to each functional requirement. For each functional requirement a +, / or − was given. The + represents a positive effect and can have a maximum of 2. The / represents a neutral effect, where conditions stay the same. And the − represents a negative effect with a minimum of 2. The total score shows the overall score for each concept. The results of this process can be seen in Table 4.11.

Concept	Water quality	Discharge capacity	Odour	Uninterrupted beachline	Costs	Total score
Culvert	/	++	+	+	--	2
Pipeline	/	+	+	++	-	3
Hybrid	/	++	+	+	-	3
Sewage	++	/	++	--	-	1
Eco-floating bed	+	/	+	--	+	1
Water retention measures	/	+	/	-	-	-1
Education & regulation	+	/	+	--	++	3

Table 4.11: Ranking of concepts on the functional requirements.

The ranking shows that the pipeline and culvert concepts look most promising along with education and regulation. The other concepts each have their own advantages but they do not meet all the requirements. They can be implemented in the overall design in an assistive function. Therefore, it was decided to only work out the pipeline and hybrid concept to see if they can meet the requirements.

4.2 Water quality

The most important requirement is that the water quality at the coastline along the Cinta Costera will be of proper quality to be used for recreational use. To prove that this is the case the modeling software Delft3D was used. Delft3D is a modeling program to investigate hydronamics, sediment transport, morphology and water quality for estuarine and coastal environments. In this case the water quality module, called D-Water Quality, is the most interesting. This module simulates the far- and mid-field water and sediment quality due to a variety of transport and water quality processes. To accommodate these, it includes several advection diffusion solvers and an extensive library of standardized process formulations with the user-selected substances

(Deltares, 2019).

4.2.1 Model set-up

Before the water quality can be modeled inside Delft3D, several steps need to be taken. Normally the following modules are worked out in the following order:

- **Grid module:** The grid module separates the area of interest in a grid of cell sizes. Depending on the required accuracy of the calculations these cells can vary in size. The smaller the cell size the larger the accuracy but this will require a longer runtime of the model to perform the calculations. When the grid is made the bathymetry file of the area can be inserted to create a depth file which Delft3D uses in further modules.
- **Flow module:** The flow module combines the spatial data of the environment (defined in the Grid module) with the forcing elements that create the currents and flow speeds in the area of interest. In the model this was realized by inserting the astronomical tide as boundary conditions of the model. These boundary conditions are located on the edges of the created grid. When the boundary conditions are defined a time frame over which the model will run simulations can be entered. In this case the time frame that will result in the worst case scenario for the water quality in the bay is used as input. This scenario occurs during neap tide (11-05-2019 to 14-05-2019) when the supply of fresh, clean water from the sea is lowest. Now that all information is selected the flow module can run a simulation.
- **Wave module:** When the behavior of waves is of interest the wave module can be used. In here, data of the mean wave height and wave period is coupled with the flow module to model the occurring wave heights and directions over the modeled area. In case of water quality the current in the bay is more important because this transports the water over the water height. Therefore it is chosen to not use the wave module in order to lower the run time of the overall model.
- **D-Water Quality module:** When the flow model is made it can be combined with the D-water Quality module to create a water quality model. First the substances of interest are selected from a library within the D-Water Quality module. Next the boundary conditions from the substances need to be formulated. These follow from Section 2.3.6 Water quality, where quantities of a number of substances were measured. Finally a discharge point with a certain discharge can be allocated on the grid.

The previous description is intended to give a brief explanation on how the model is made. For a more detailed look on how each module is made see Appendix C.

4.2.2 Validation of the flow model

The results of the flow model must first be verified before they can be used for the water quality model. This is to prove that the modeled behavior corresponds with reality. This verification is done by using tidal information from a measuring station at Flamengo island (see Figure 4.32), which is available trough a Delft3D tool called Delft Dashboard. At the same location, the results of the flow model are used to plot the tidal range. When the measured and modeled line are close together it can be assumed that the model is good enough to recreate the real occurring conditions. From the results, shown in figure 4.32, it can be seen that the lines coincide quite nicely. This shows that the modeled water levels are close to the actual measured water levels at Flamengo island. From this result it is concluded that the flow model is of sufficient quality and can be used in further water quality simulations.

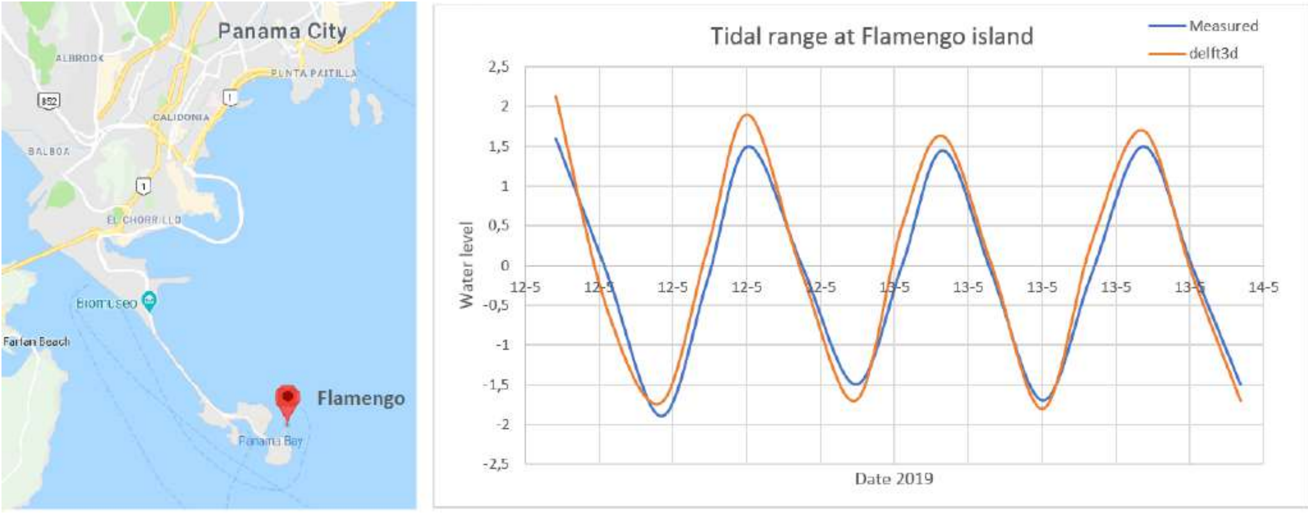


Figure 4.32: Modelled and measured tidal signal at Flamengo island

4.2.3 Water quality model set-up

The set-up of the water quality model is now discussed in more detail to clarify the used methods.

Selected substances

The first step is to select the substances from which the behavior will be modeled. In Section 2.3.6 Water quality, the measurement results of the Sanitation program were shown. From these it became clear that the conditions in the bay are good but inside the river are bad. The main contributors to the bad state of the river are fecal bacteria. Two other substances that scored below the tolerable limit are dissolved oxygen and biochemical oxygen demand. However these do not pose an immediate threat for human health when it comes in contact. Therefore it is decided to select the following substances from the substances library within the D-Water Quality module:

- Fecal coliforms
- Fecal E.Coli
- Total coliforms

Some additional parameters are entered in the model that are needed to imitate the environment. These parameters include the water temperature, salinity, mortality rate, etc. These are needed to simulate the natural processes that the fecal bacteria are subjected to. In the water quality survey the fecal coliforms were measured in CFU/100 ml (Coliform Forming unit). In the Delft3D model these substances must be entered in MPN/m^3 (Most Probable Number). From CFU to MPN is a 1 to 1 transformation. Therefore, to get the correct units in Delft3D, the substances are prepared by using the transformation from 100 ml to 1 m^3.Thus, $1MPN/m^3$ equals $1 * 10^4$ CFU/100 ml.

Outfall location & measuring locations

A number of measuring locations need to be allocated inside the Delft3D model before the outfall location can be determined. These measuring locations present the quantities of each substance from a certain outfall location at that point in the model. In Figure 4.33 these measurement locations are labeled from number 1 to 7. Number 1, 2 and 3 are located at the intended beach line of Bella Vista. Number 4 to 7 are located at the edge where swimmers from the beach are expected. These give the boundary at which the water quality has to be sufficient.

Next four outfall locations were chosen, see Figure 4.33. These were chosen randomly to see how the location affects the water quality at a observation point. The location of the outfall will also determine the length of the pipeline. Therefore, their exact GPS locations are recorded in Table 4.12 along with the length from river mouth to the outfall location.

Discharge in outfall

The final condition is given by the discharge from the river that exits at the outfall location. This discharge

Figure 4.33: Visual representation of outfall locations and observation points

Outfall location	GPS location	Distance from river mouth to outfall location (m)
1	Long: -79.515 Lat: 8.9645	1200
2	Long: -79.518 Lat: 8.9663	830
3	Long: -79.522 Lat: 8.9655	900
4	Long: -79.5158 Lat: 8.9675	1000
5	Long: -79.5131 Lat: 8.9661	1400

Table 4.12: Exact location of different outfalls with their distance to the mouth of Matasnillo river.

will vary with the tide. When the tide shifts from high to low there will be a higher discharge than during the shift from low to high due to the pressure of the water level in the river at high water tide. From Section 2.3.3 it became clear that under dry conditions the discharge in the river is more or less 1.5 m^3/s. Based on the remaining volume of water in the river at high water conditions it is estimated that the discharge under dry conditions from high tide to low tide will be 3 m^3/s. Conversely, from low tide to high tide, the discharge is estimated to be 1 m^3/s. In the model this behavior is simulated by giving a discharge of 3 m^3/s at time intervals from high to low water levels and a value of 1 m^3/s from low to high water levels.

4.2.4 Results of water quality model

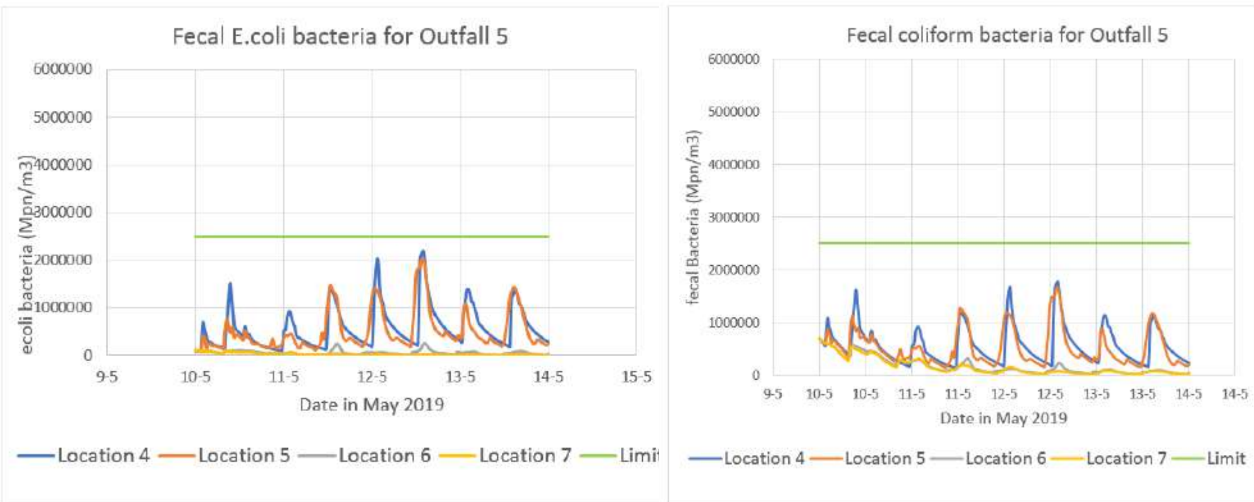
After completion, the water quality model was coupled with the flow model and the data were collected. For each outfall location all data for the three substances under consideration were analysed.In Table 4.13 a summary can be seen with results for outfall 1 to 4. For detailed graphs of every substance at each observation point see Appendix C.2.

Outfall location	Substance	Tolerable limit exceeded at observation point	Highest modeled value (MPN/m³)	Tolerable limit (MPN/ m³)	Number of times limit exceeded in 4 days runtime
1	Fecal coliforms	5	4.3*10⁶	2.5*10⁶	4
	Fecal E.coli	4 & 5	5.3*10⁶	2.5*10⁶	5
	Total coliforms	4 & 5	8.6*10⁶	5*10⁶	4
2	Fecal coliforms	4 & 5	7.3*10⁶	2.5*10⁶	8
	Fecal E.coli	4 & 5	9.2*10⁶	2.5*10⁶	9
	Total coliforms	4 & 5	15.7*10⁶	5*10⁶	8
3	Fecal coliforms	5	8*10⁶	2.5*10⁶	6
	Fecal E.coli	5 & 6	9.9*10⁶	2.5*10⁶	6
	Total coliforms	5	7.3*10⁶	5*10⁶	6
4	Fecal coliforms	4 & 5	3.7*10⁶	2.5*10⁶	6
	Fecal E.coli	4 & 5	4.6*10⁶	2.5*10⁶	7
	Total coliforms	4 & 5	7.2*10⁶	5*10⁶	6

Table 4.13: Results of 5 modeled outfall locations.

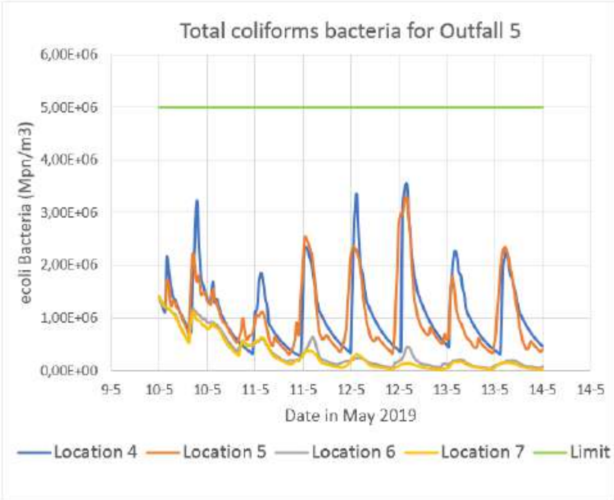
From the results ,it can be seen that for all the chosen outfall locations the tolerable limit is exceeded. With the chosen locations it is no surprise that the most vulnerable observations points are 4 and 5, which occur dominantly in the exceeded observation point column. However, some conclusions can still be drawn from the results. When the highest modeled value for each substance is compared with the tolerable limit, it can be seen that outfall location 1 and 4 come out better than location 2 and 3. For outfall location 1 the larger distance to the observation point is the main contributor to the better results. For outfall location 4 the placement around the head of the peninsula of Punta Paitilla results in better values, because a part of the polluted water cloud is redirected along the Eastern side of the peninsula with the current.

With the insight gained from the results it was decided to create a 5^{th} outfall location, see Figure 4.33. This location combines the positive effects from location 1 and 4. For each substance the results are plotted for every separate observation point, see Figure 4.34. In each figure the horizontal green line represents the tolerable limit for that fecal bacteria. This line may not be exceeded. It can be seen that the tolerable limit is not exceeded for all measurement locations. From this the conclusion is made that outfall location 5 is a location for which the water from Matasnillo can be discharged to meet the safety conditions at the beach.



(a) Results for outfall 5 of fecal E.coli bacteria

(b) Results for outfall 5 of fecal coliform bacteria



(c) Results for outfall 5 for total coliforms bacteria

Figure 4.34: Results for outfall 5 of Fecal E.coli and coliform bacteria

4.3 Discharge capacity

For each concept the discharge capacity needs to be guaranteed in order to prevent floodings in the area during heavy rainfall and will be treated separately. Because the discharge in the river is not exactly known it was decided, in consultation with Boskalis, to look at the capacity for different discharges ranging from 60 m^3/s to 150 m^3/s. The upper limit follows from the probabilistic analysis done in Section 2.3.3. The lower limit was provided by an employee of Boskalis living in Panama City as expected maximum discharge. The boundary conditions for the highest tide are used, because under these conditions the river will have the highest water level and therefore less capacity to discharge the water from a heavy rainstorm.

The flow speeds from the river to the sea through the outfall structure can be determined by Bernouilli's principle or the hydraulic head balance, see Equation 4.5. Important parameters for these calculations are the piezometric head at the entrance and exit level of the pipe (taken with the same reference level) and the head loss over the structure. For a visual representation of the situation see Figure 4.35.

$$H_1 + \frac{v_1^2}{2g} - \Delta H_{total} = H_2 + \frac{v_2^2}{2g} \tag{4.5}$$

In which:

$$\begin{aligned} H_{1-2} &= \text{Piezometric head at location 1 and 2 (m)} \\ v_{1-2} &= \text{Water velocity at location 1 and 2 (m/s)} \\ \Delta H &= \text{Hydraulic head losses (m)} \\ g &= \text{Gravitational accelaration (m/s}^2\text{)} \end{aligned}$$

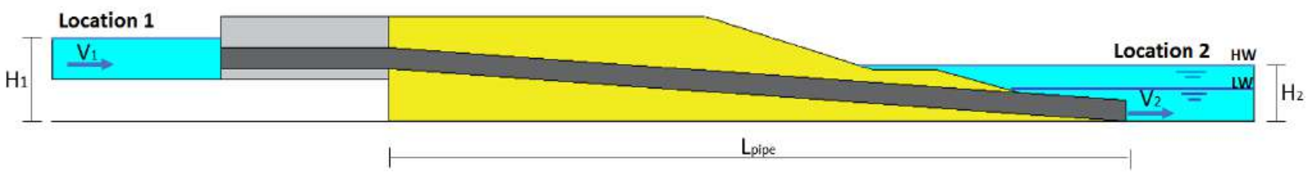


Figure 4.35: Visual representation of situation with outfall design

Due to the large tidal variation in the bay of Panama City, there will be outflow and inflow through the outfall and pipeline structure. At the moment this also happens in Matasnillo river and is desired, because the fresh sea water flushes the river from pollutants and reduces the odour. The structure will cause some head losses and when these exceed the piezometric head difference there will be no flow from the river to the sea and visa-versa in case of high water. Therefore, the head losses are an important factor and will be discussed in the following section.

4.3.1 Pipeline concept

To determine the discharge capacity of the pipeline concept, first, some assumptions considering the dimensions are made. In order to use the maximum available width in the river of 15 meters, 5 circular pipes with a diameter of 3 meters are chosen. The length of the pipes follows from section 4.2. Using the head balance the flow speed in the a pipe can be determined and ultimately the discharge by multiplying the flow speed with the area.

Head losses for pipeline
Over the entire structure head losses will occur due to entrance and exit flows and friction along the length of the pipe. The total head loss over the structure can be determined using Equation 4.6.

$$\Delta H_{total} = \Delta H_{entrance} + \Delta H_{pipe} + \Delta H_{exit} = K_{in}\frac{v_1^2}{2g} + f * \frac{L}{D}\frac{v_2^2}{2g} + K_{out}\frac{v_2^2}{2g} \tag{4.6}$$

In which:

ΔH = Hydraulic head loss (m)
K = Entrance & exit loss coefficient (−)
f = Darcy-Weisbach friction coefficient (−)
L = Length of the pipe (m)
v = Water velocity (m/s)
D = Inner diameter of the pipe (m)

The head losses over the entrance and exit are dependent on the loss coefficients and the water velocity at each location. For the inlet structure the entrance loss coefficient K_{in} depends on the shape of the structure. The most efficient shape is rounded. In addition the radius to diameter ratio determines the final K-value. Different values for a rounded inlet shape can be seen in Figure 4.36. The exit loss coefficient K_{out} for the outlet is equal to 1, because when the pipe discharges in a large body of water the velocity is reduced to zero and all kinetic energy is dissipated regardless of the exit geometry.

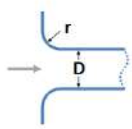
Type	Shape	r/D	K-value
Rounded		0.02	K = 0.28
		0.04	K = 0.24
		0.06	K = 0.15
		0.10	K = 0.09
		0.15+	K = 0.04

Figure 4.36: K-values for rounded entrance types and different r/D ratios

The Darcy-Weisbach friction coefficient can be found in an iterative way by using the White-Colebrook equation, see Equation 4.7.

$$\frac{1}{\sqrt{f}} = -2log(\frac{\epsilon}{3.7D} + \frac{2.51}{Re\sqrt{f}}) \tag{4.7}$$

In which:

$$
\begin{aligned}
f &= \text{Darcy friction factor } (-) \\
\epsilon &= \text{Effective roughness height (m)} \\
Re &= \text{Reynolds number}(Re = \frac{\rho * v * D}{\mu})\ (-)
\end{aligned}
$$

Results for pipeline concept

By combining all the equations and using the boundary conditions from high tide as input, the discharge capacity can now be checked. For the pipeline concept the maximum width of the river, which was 15 meters, is used by 5 pipes with each a diameter of 3 meters, resulting in a total area of 35 m^2. The most important input parameters and result can be seen in Table 4.14. The discharge capacity of the pipeline concept is only 56.5 m^3/s where it should be at least 60 m^3/s. Since there is not enough space to install extra pipes it can be concluded that this pipeline concept does not meet the requirements.

River water level H1 (m CD)	Sea water level H2 (m CD)	Pipe length (m)	Pipe area (m^2)	River flow speed V1 (m/s)	Pipe flow speed V2 (m/s)	River discharge (m^3/s)	Pipeline capacity (m^3/s)
5	3.3	1000	35	1.5	1.6	60	56.5

Table 4.14: Used input parameters and resutls for pipeline concept capacity

4.3.2 Hybrid concept

The hybrid concept consists of the pipeline and the culvert structure. Each part has a different function. The pipeline will discharge the polluted water under normal conditions, whereas the culvert will discharge the surplus of water during heavy rainfall. Because of the different functions the approach to determine the capacity is also different for both parts. These will be discussed separately.

Outfall pipeline section of hybrid concept

For the outfall pipeline part of the hybrid structure two situations are considered: discharge during neap and spring tide under dry conditions. For both tides the water reaches to a certain height in the river. With this height, together with the accompanying cross-sectional area and the slope, the total amount of water flowing in the river can be calculated. When the tide at sea shifts from high water to low water the water level inside the river must be able to follow this motion. In other words, the capacity of the pipeline needs to be large enough such that enough water can flow through for the water level in the river to drop. The water level on the river side can be described by Formula 4.8. An elaboration on how this formula was found can be seen in Appendix D.

$$
\begin{aligned}
h_i &= \sqrt{\frac{V_i}{3545}} + 1.7 \\
V_i &= V_{i-1} + V_{in\Delta t} - V_{out\Delta t} \\
V_{in\Delta t} &= Q_{river} * 900 \\
V_{out\Delta t} &= v_2 * A_{pipe} * 900
\end{aligned}
\tag{4.8}
$$

In which:

$$
\begin{aligned}
h_i &= \text{Water level at time interval i (m)} \\
Q_i &= \text{Total volume of water in river at time interval i (m}^3\text{)} \\
Q_{in\Delta t} &= \text{Incoming volume of water in river over time interval of 15 min (m}^3\text{)} \\
Q_{out\Delta t} &= \text{Outgoing volume of water in river over time interval of 15 min (m}^3\text{)}
\end{aligned}
$$

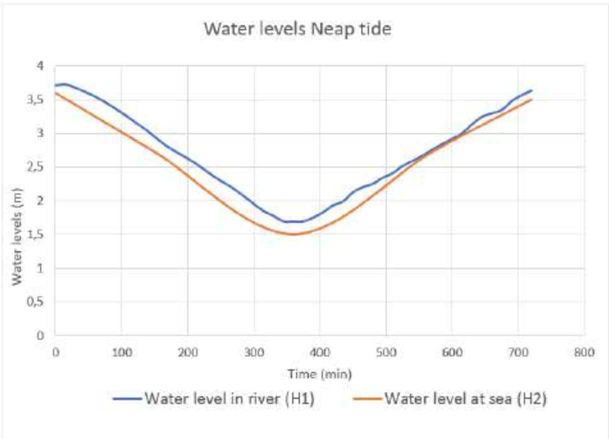
For the calculations of the flow speed in the pipe the same equations are used as for the pipeline concept, Equations 4.6, 4.7. However, in this approach for each time step of 15 minutes the volume of water that flows through the pipe is deducted from the total starting volume at high water level. The retaining wall, which collects the dirty water, is 2 meters high up to CD + 3.7. At high tide water will be able to flow over this wall until it reaches CD +3.7 m. From this level the water will flow through the pipes. The starting volume V_0 is thus calculated from the highest level up to CD +3.7 m. For each time step of the remaining volume the new water level in the river is determined. The starting values and used parameters in the calculations are presented in Table 4.15.

	High water level (m CD)	Low water level (m CD)	Wall height (m CD)	Pipe area (m^2)	Volume at high water level Q_0 (m^3)	Q_{river} (m^3/s)
Neap tide	3.6	1.5	3.7	6.3	12350	1.5
Spring tide	5,3	-0,7	3.7	6.3	14181	1.5

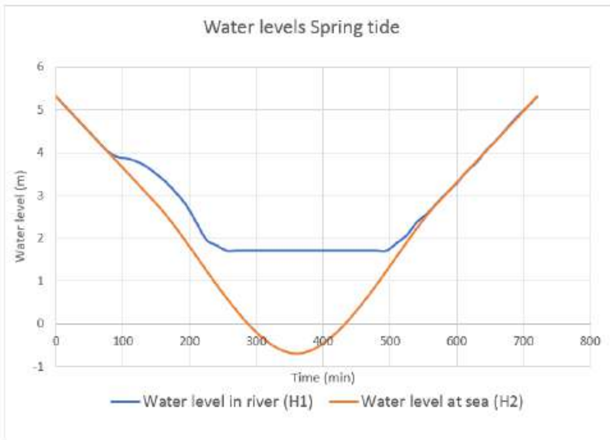
Table 4.15: Used input parameters for spring and neap tide calculations for pipeline capacity in hybrid concept.

Results outfall pipeline for culvert concept

The results of water levels for both neap tide and spring tide are shown in 4.37. In the figures the red line represents the water level at sea (location 2) that follows the tide. The blue line represents the water level in the river (location 1) and has a minimum of 1.7 meters because that is height of the bed level (CD + 1.7m). Due to losses there is some delay in which the water level in the river follows the water level at sea. However, for both neap tide and spring tide the water level in the river is able to reach its lowest level at bed level. This means that the capacity is sufficient and all the water can be discharged through the outfall pipelines before the tide rises again.



(a) Computed water levels for pipeline section of hybrid concept during neap tide



(b) Computed water levels for pipeline section of hybrid concept during spring tide

Figure 4.37: Computed water levels for pipeline section of hybrid concept during spring and neap tide

Culvert section of hybrid concept

The dimensions of the culvert are calculated for a range of different discharge capacities ranging from 60 m^3/s to 150 m^3/s. This is done to get insight in how much the dimensions of the culvert will change for different discharges, since the discharge is not exactly known. The flow speeds are determined using the principle of Equation 4.5. In turn, the required area for a given discharge capacity is calculated by dividing the discharge capacity by the corresponding flow speed. From this area the width and height of the culvert dimensions can be computed.

Head losses for culvert

The culvert structure will also experience head losses and this is calculated using the Strickler equation. This equation is generally used for open channel flow, which also holds for culvert structures that are not completely enclosed with water . However, the formula can still be used in case the culvert is completely submerged (Hoes, 2018). Using Strickler the head loss over the culvert is given by Equation 4.9.

$$\Delta H_{total} = \Delta H_{entrance} + \Delta H_{culvert} + \Delta H_{exit} = K_{in}\frac{v_1^2}{2g} + \frac{v_2^2 L}{k^2 R^{4/3}} + K_{out}\frac{v_2^2}{2g} \tag{4.9}$$

In which:

k = Strickler coefficient (70 for concrete) (−)

R = Hydraulic radius (m)

Results dimensions of culvert for hybrid concept

The results of the calculations for culvert dimensions are presented in Table 4.16. The top line indicated with Q_{river} shows the discharges for which a culvert is designed. The total area is given by multiplying the number of sections with the width and height. Finally the values in $Q_{culvert}$ show the maximum discharge capacity for the given dimensions and Q_{net} shows the net result between the required and obtained discharge.

Q_{river} (m^3/s)	60	70	80	100	120	140	150
Sections	2	2	2	2	3	3	3
Width (m)	4	4	4	5	4	4	4
Height (m)	3	3	3,5	4	3,5	4	4
Total (#*b*h)	2x4x3	2x4x3	2x4x3,5	2x5x4	3x4x3,5	3x4x4	3x4x4
Area (m^2)	24	24	28	40	42	48	48
$Q_{culvert}$ (m^3/s)	67	67	78,4	112	126	148,8	148,8
Q_{net} (m^3/s)	7	-3	-1,6	12	6	8,8	-1,2

Table 4.16: Dimensions of culvert for different discharge capacities.

From Table 4.16 it can be seen that the discharge capacity is insufficient for the given values of 70, 80 and 150 m^3/s. However the pipelines will also have a contribution of 10 m^3/s (follows from results of section 4.3.1). Therefore the total capacity of both the culvert section and the pipeline section of the hybrid concept will be sufficient for all the computed discharge capacities in Table 4.16.

4.4 Odour

Currently the problems with bad odour in the area occur mainly during low water. At this time the water in Matasnillo river is of the lowest quality, because it is not mixed with fresh seawater that flows in during high water. However, the smell does not only originate from the water in the river. During the visit to Panama City it was observed that a layer of sludge is located on the bottom and banks of the river and outside the river mouth area. This layer has built op over the years and contains pollutants and fecal residue. During low water this layer of sludge partly emerges and contributes to the odour in the area. It is difficult to provide a solution based on calculations for this problem since smell is hard to measure. But still a fourfold based solution is presented to prevent a bad odour when the beach is in operation and consists of the following:

- **Inlet location**: the inlet of the hybrid concept will be located under the highway crossing of Avenida Balboa, see Figure 5.38. This will provide a covered distance from the beach to the river of 125 meters. At this location it is not expected that the odour will penetrate through the exhaust gasses emitted by the traffic on this highway.

- **Sludge dredging**: The layer of sludge inside the river and in the area of the river mouth will be removed. Furthermore, the beach will cover a large part of the area that currently emerges at low water, which further reduces the odour of these areas.
- **Sufficient discharge capacity**: As was discussed in Section 4.3.2, the discharge capacity during dry conditions will be sufficient to discharge the water in Matasnillo river during high and low tide. In this way there will be no accumulation of dirty water (which will lead to an increase of odour) in front of the inlet structure.
- **Sanitation Program**: Over the last decade the Sanitation Program has done an excellent job in improving the water quality of the rivers in Panama City. Based on their efforts the water quality will further improve in the future. With a further improvement of the water quality the odour of the water will also decrease.

4.4.1 Conclusion concept designs

From the calculations of the discharge capacity it can be concluded that the pipeline concept does not fulfill the functional requirement since the maximum achievable capacity is 56.5 m^3/s where a minimum of 60 m^3/s was required. For the hybrid concept several dimensions were computed to guarantee the outflow in the river for different discharge capacities, see Figure 4.16. This was done in consultation with Boskalis, because at this point the discharge is not exactly known. From their point of view it will be interesting to work out the 60 and 150 m^3/s design of the hybrid concept and look at the differences in costs for future development. Therefore, as from this point, it was decided to only look at the hybrid concept and work out further details for the 60 and 150 m^3/s discharge capacities designs. These two design variants are worked out to see what differences will occur in constructablity and costs.

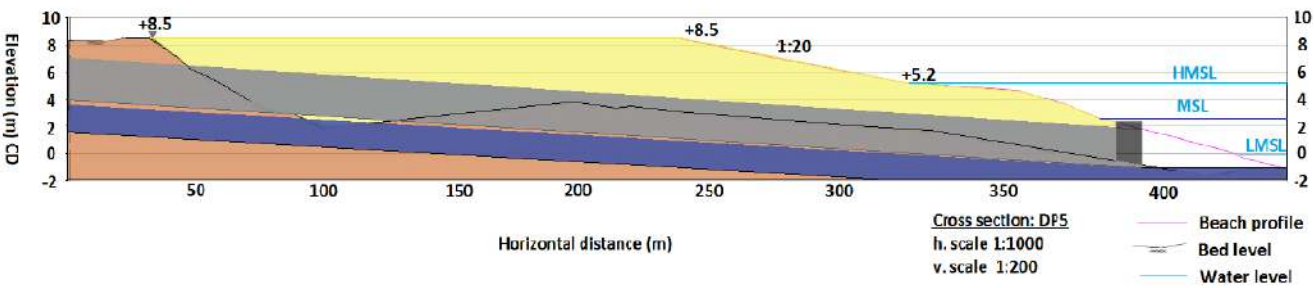
Chapter 5. Positioning & design of hybrid concepts

This chapter describes the positioning of the hybrid design variants. It also shows the improved designs compared to the conceptual design phase.

5.1 Positioning of culvert and outfall for hybrid concept

In Figure 5.38 the design layout can be seen for both the top view and side view. The position of the culvert is according to Figure 2.9 along path DP5. This is done to make use of the natural gully that exits there and thus minimize construction works to excavate a new gully in the rocky material. The position of the outfall needs to be deeper in the bed than the culvert. If the outfall would be installed at the same level it would appear above water at low tide. This would not only be ugly to look at but will also obstruct sailing.

(a) Top view of the position for outfall and culvert at Bella Vista beach.



(b) Side view of position for outfall and culvert in Bella Vista beach profile.

Figure 5.38: Position and layout for outfall and culvert

Although the position of the culvert and outfall will be similar for both the 60 and 150 m^3/s design, some differences will occur in volume of rock that has to be removed due to the different size. This is discussed in further detail for both the culvert and the outfall.

5.1.1 Culvert

The culvert will be placed along the existing gully. However, this gully is not deep enough for the culvert to lie in. If placed under current conditions the culvert would stick out of the beach until it reaches the groyne. Therefore some of the rock will have to be removed. In Figure 5.39 the area that is to be removed is indicated by the shaded red area. The figure represents the situation for the 60 m^3/s design culvert. In case of the 150 m^3/s design the culvert will be an additional meter deep.

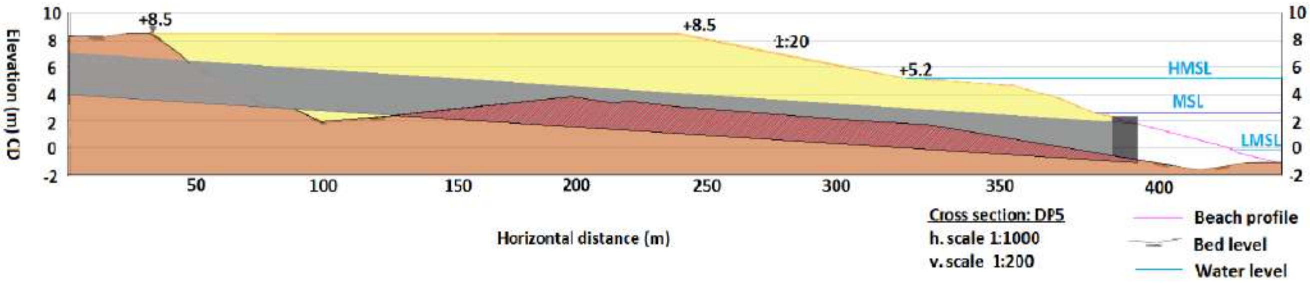


Figure 5.39: Side view for 60 m^3/s culvert, rock to be me removed is indicated by the shaded red area.

Using Figure 5.39 the area which has to be removed can be determined. When this is combined with the width of both the 60 and 150 m^3/s culvert design the total volume of rock that has to be removed can be calculated. The dimensions of both designs, including wall thicknes, are further explained in Section 6.3. The results are shown in Table 5.17.

Concept design	Width of culvert (m)	Depth of culvert (m)	Area to be removed (m^2)	Volume to be removed (m^3)
60 m^3/s	9.2	3	382	**3514**
150 m^3/s	13.8	4	615	**8487**

Table 5.17: Volume of rock to be removed for both the 60 and 150 m^3/s concept design culvert

5.1.2 Outfall

The positioning of the outfall can be considered in two sections. The straight section next to the culvert and the curved section from the groyne to the outlet of the outfall. Each section is discussed separately.

Section: straight outfall next to culvert
The section next to the culvert needs to be placed deeper than the culvert itself. As mentioned earlier this is to prevent the outfall from rising above the water at low tide. In Figure 5.40 the position of the outfall together with the area of rock that needs to be removed (indicated in the shaded red area) is presented. For two pipes with a inner diameter of 2 meters the width of the gully will be 5 meters. The volume of rock that needs to be removed is shown in Table 5.18.

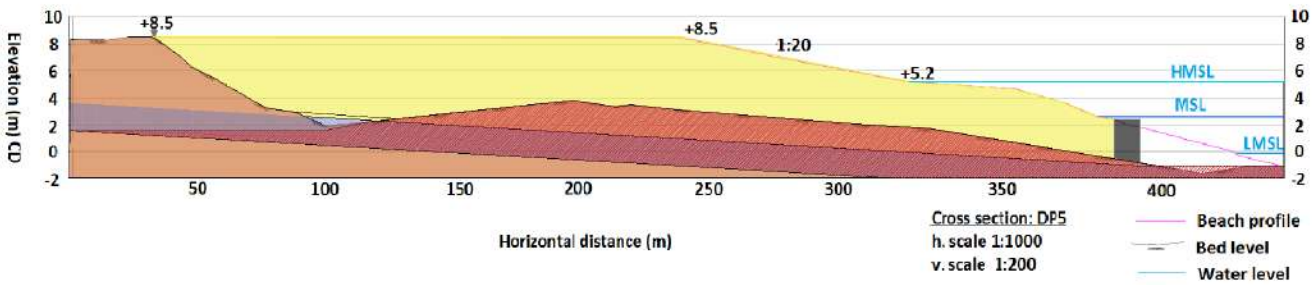


Figure 5.40: Side view for outfall, rock to be me removed is indicated by the shaded red area.

Section: curved outfall
The outlet location of the outfall was determined using the Delft3D model in section 4.2 Water quality. In order to reach this location the outfall will follow a curved path from the groyne to the outlet. For this section

Concept design	Width of outfall (m)	Depth of outfall (m)	Area to be removed (m^2)	Volume to be removed (m^3)
Outfall	5	2	1147	**5735**

Table 5.18: Volume of rock to be removed for the straight outfall next to the culvert

a different approach is needed. From the earlier performed drilling (which can be seen in Appendix B.5), and experience of Boskalis in the area, it is expected that the soil conditions change when the curved section starts at height of the head of Punta Paitilla. The rocky basalt is covered by a layer of silt and sludge. Therefore, a different approach is needed to create a trench in this section. This will be done by use of a backhoe dredging platform. On such a platform an excavator is situated, which can dig up the muddy soil easily. For harder materials the excavator can also be fitted with a drumcutter head. A more detailed description on this work method can be found in Section 6.1 Construction.

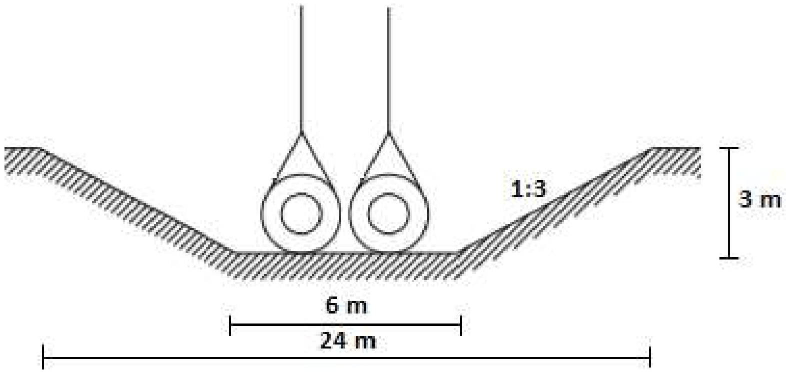


Figure 5.41: Cross-section of trench for the curved section of the outfall.

Concept design	Length trench (m)	Area trench (m^2/m)	Volume to be removed (m^3)
Outfall curved	700	63	**44100**

Table 5.19: Volume of silt to be removed for curved outfall trench

5.2 Functional design of hybrid concept

Now that the conditions which satisfy the boundary conditions are known, the functional design of the hybrid concept can be worked out in further detail. Each of the following components of the hybrid concept is worked out in further detail, supported with illustrations:

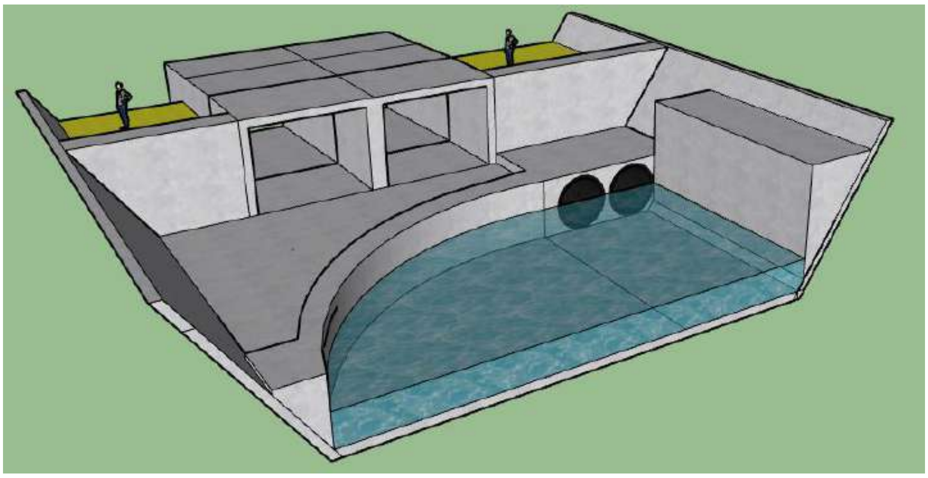
- Inlet structure
- Culvert outlet structure
- Culvert block section design
- Outfall pipeline

5.2.1 Inlet structure

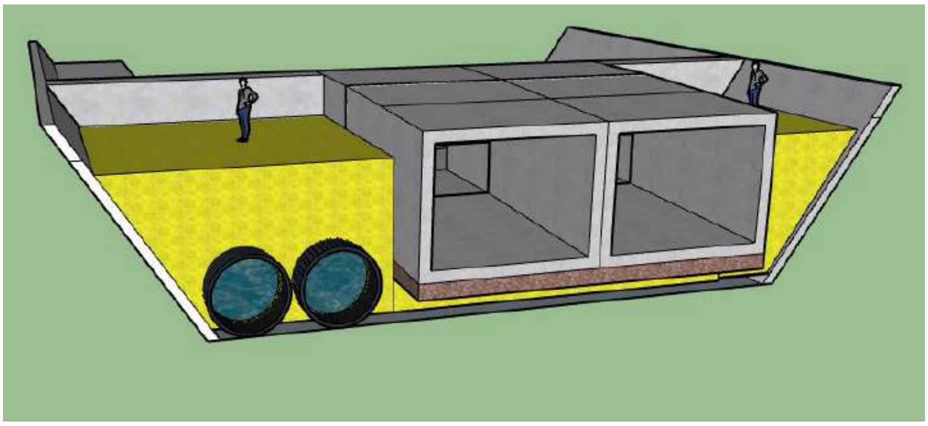
The culvert inlet structure is located at the river mouth and will have the important function to catch the polluted water under dry conditions and to discharge this water trough the outfall pipes. The overall design is similar to the concept design in Section 3.1.3. However, due to the implementation of the culvert dimensions, some adjustments had to be made for the 150 m^3/s design. These are discussed later.

60 m^3/s design

In Figure 5.42 a visual representation of the culvert for a 60 m^3/s design capacity is shown under dry conditions during low water (for a representation during high water conditions see Figure E.1 in Appendix E). In this situation the inlet can be built inside the current river embankments without making any mayor changes. The two outfall pipes with inner diameter of 2 meters are located on the Western bank of the river. This height is also used as wall height that will catch the polluted water in the river. The water will only exceed this wall if either the high water level at sea is higher than CD +3.7 m or if the water level on the river side exceeds 2 meters due to heavy rainfall. In this case the water will flow through the culvert. Under the culvert a gravel layer of 25 cm is placed.



(a) River side view at culvert inlet for 60 m^3/s design capacity.

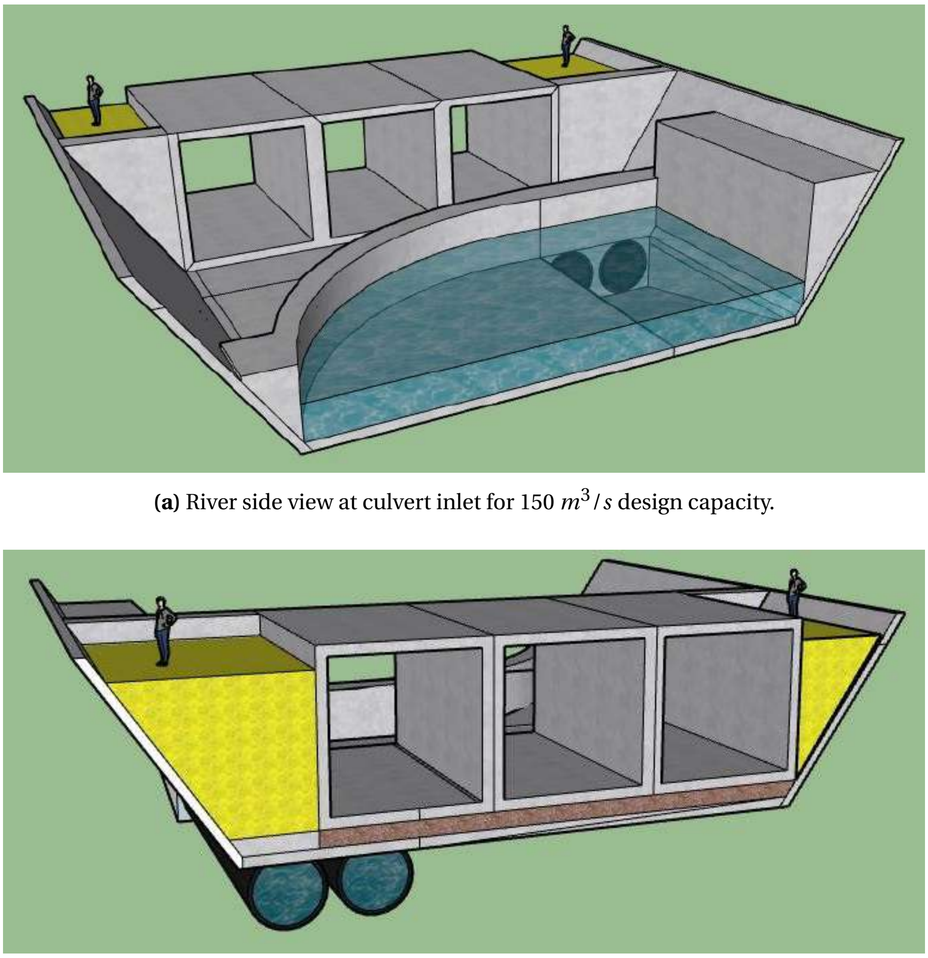


(b) Sea side view at culvert inlet for 60 m^3/s design capacity

Figure 5.42: Culvert inlet for 60 m^3/s design capacity at river mouth

150 m^3/s design

In Figure 5.43 a visual representation of the culvert for the 150 m^3/s design capacity is shown under dry conditions during low water (for a representation during high water conditions see Figure E.2 in Appendix E). For this design some changes will have to be made to the existing river banks, because in case the three culvert sections are placed next to each other there will not be enough room for the outfall pipes. As a solution the outfall pipes will be placed 2 meters lower than the current bed level, this can clearly be seen in Figure 5.43a. For water levels the same conditions apply as for the 60 m^3/s design capacity, because the water retaining wall is still positioned at the same level. The gravel layer will also be 25 cm.



(a) River side view at culvert inlet for 150 m^3/s design capacity.

(b) Sea side view at culvert inlet for 150 m^3/s design capacity.

Figure 5.43: Culvert inlet for 150 m^3/s design capacity at river mouth

Production of inlet design

The inlet structure will be a combination of prefab and in-situ cast concrete and will have to made during the dry season in a time period of 3 months. During this time period no high discharges are expected inside the river The sloping walls at the river banks and the barriers between the culvert units and beach can be made from prefab concrete slabs to speed up construction. The floor and the wall, which will catch the dirty water, will be cast on site. The total amount of concrete needed for both designs is presented in Table 5.20 and is based on engineering drawings which can be found in Appendix E.2.

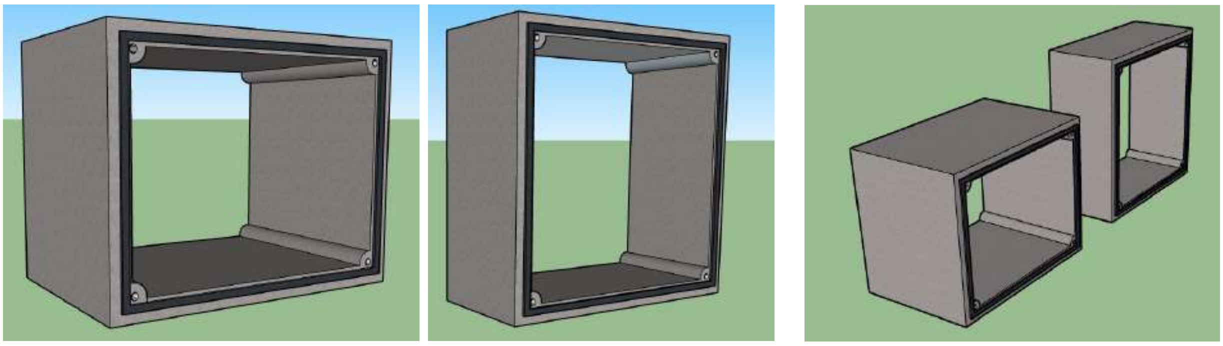
Design variant	Prefab walls (m^3)	Prefab floors (m^3)	In-situ cast (m^3)	Total (m^3)
60 m^3/s	35	32	140	207
150 m^3/s	35	40	120	195

Table 5.20: Volume of concrete needed for different parts of the hybrid culvert inlet design.

5.2.2 Culvert block sections design

In the design of the culvert block sections several points were taken into consideration. The dimensions determined in Section 4.3.2 serve as a starting point for the total design for each block section. In this stage the limiting factor of a cranes lifting capacity and workabilty is taken into account. Therefore, the maximum weight for each block section of both the 60 and 150 m^3/s design is limited to be 30 tonnes. In Section 6.3 the wall thickness will be calculated based on the loads and then the length of each block section can be determined based on the limiting weight. For now the design of the culvert block sections is discussed based on functionality. In Figure 5.44 the design of the culvert bock sections can be seen. Several components of the block sections are now explained in more detail.

- **Dimensions**: The inner dimensions for each block section are 3x4 m and 4x4 m for the 60 and 150 m^3/s design, respectively. With the limiting weight of 30 tonnes the length of the 150 m^3/s design block section will be lower than the 60 m^3/s design. The length and weight for each design will be determined in Section 6.3.
- **Rubber strips**: On both faces of each block section a rubber strip will be placed of the full circumference. When the block section are installed next to each other, this rubber strip will prevent leakage through the block sections.
- **Tension cable holes**: In each corner of the blocks a hole is located over the full length. A steel cable will be pulled through this hole during installment. When all the block sections are in place the 4 cables will be brought under tension. As a result all the sections will be neatly aligned with the rubber strips compressed to each other.

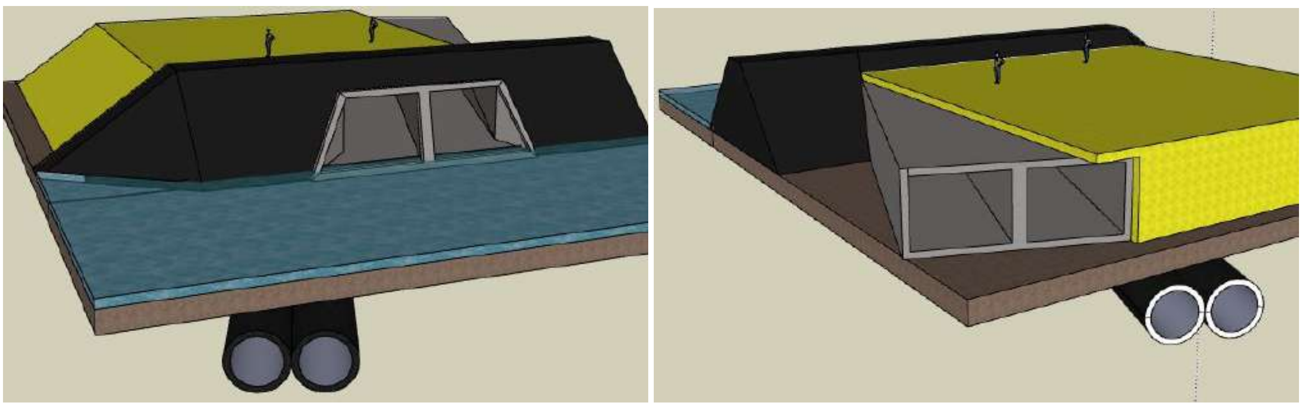


(a) Separate block section for 60 m^3/s design **(b)** Separate block section for 150 m^3/s design **(c)** Perspective both designs of culvert sections

Figure 5.44: Separate culvert sections for 60 and 150 m^3/s designs

5.2.3 Culvert outlet structure

The outlet of the culvert will be positioned on the Western edge of the beach at the head of Punta Paitilla, see Figure 5.38. At this location a groyne protects the beach from incoming wave attack. The culvert will penetrate through the groyne as is pictured in Figure 5.45. In this way the culvert is hidden from the public and can not be seen from the beach. When the outlet structure is in place the groyne can be built around it.

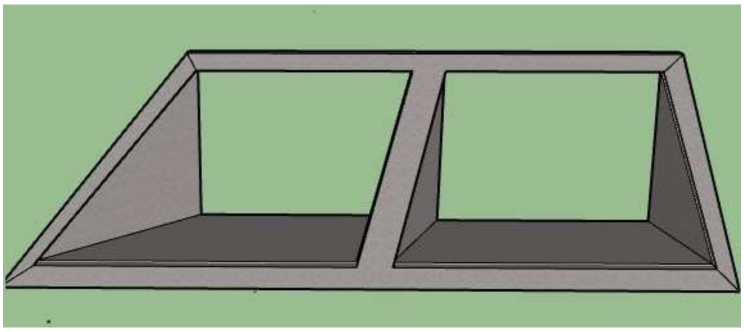


(a) Sea side view of culvert outlet and groyne **(b)** Beach side view of culvert outlet and groyne

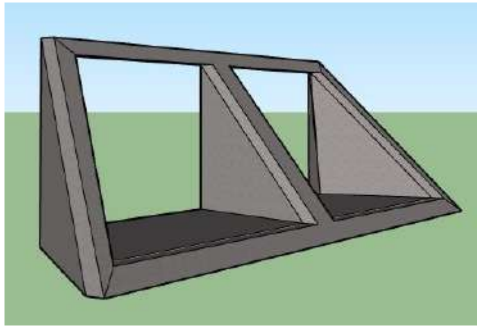
Figure 5.45: Culvert outlet design at groyne.

The shape of the culvert outlet section at the end of the culvert structure, which lies below the groyne, is different from the block shaped pieces. The shape of the outlet section depends on the angle of approach to the groyne and the slope of the groyne itself. The sloping sides of the walls will result in a wider base. At this point no design is made for the groyne so the exact dimensions of the outlet section can not be made. However, to get an impression of how it may look like, a visual representation of the outlet structure is given in Figure 5.46. Since the outlet will be positioned on top of the hard basalt layer there will be no danger of instability due to scour at high flow speeds.

For safety purposes a steel frame will be installed in the entrance of the outlet structure. This steel frame will consist out of bars ranging from top to bottom. This will prevent that people enter the culvert and get trapped inside when the water level rises. During spring tide the water level inside the culvert can get so high it will be completely filled with water near the outlet. This could lead to drowning when people are inside the structure at this time.



(a) Front view of outlet structure



(b) Side view of outlet structure

Figure 5.46: Visual representation of outlet structure.

5.2.4 Outfall pipeline

As was briefly discussed in Section 5.1 the total length of the outfall pipeline will be 1400 meters. Of which 400 meters are situated next to the culvert under the beach and the remaining 1000 meters will be installed in the bed level at sea. The outfall pipeline itself is built up from smaller sections of HDPE (high-density polyethylene) and concrete ballast units. Typically each section has a length of 6 meters, see Figure 5.47. The sections with an inner diameter of 2 meters and a wall thickness of 10 cm are connected using a method called electrofusion (Galicia, 2017). Both ends of each section are inserted into the electrofusion fitting which applies a certain voltage (Typically 40V). Then, a heater melts the inside of the fitting to the outside of the pipe walls to create a strong homogeneous and waterproof joint. The concrete ballast units are used during transport at sea to handle the outfall pipeline. When in position the section is sunk by pumping water into the pipeline. A detailed description of this process can be found in Appendix E.2. When in place the concrete ballast units also prevent uplift of the outfall pipeline.

(a) Weight and stiffener block for outfall pipeline

(b) Single outfall pipeline section

(c) Combined outfall pipeline sections

Figure 5.47: Different components of the outfall pipeline (Galicia, 2017).

Chapter 6. Structural design of hybrid concepts

In this chapter the structural design of the two design capacities of the hybrid concept are worked out in further detail.. The design must be able to withstand loads and forces during all stages of construction and during operation. The structural design criteria consist of:

- Constructability: the structure can be built on location using methods and materials such that the project is economically and structurally feasible.
- Stability: the structure is stable in its position during construction and operation.
- Strength: the structure is able to withstand forces acting on it to maintain its functionality over the designed lifetime. Also, during construction, the sections will not break when lifted into position.

The strength and stability are checked for both design capacities of the separate culvert sections (60 and 150 m^3/s). Since the dimensions of these two designs are slightly different, the forces acting on it will also be different. The larger dimensions of the 150 m^3/s design will also have an influence on constructability, leading to a longer placement time and higher costs. All these design criteria will be treated separately in the following sections.

6.1 Construction

The process of how to construct the entire project from start to finish was done before the structural design of the structure was made. The governing load cases for which the structure will be calculated follow from the construction steps. During the design phase some important construction limitations were already taken into consideration, such as the lift capacity of cranes. In the following section the entire construction process of the outfall/culvert structure is divided into several phases. The required work that has to be done during each phase will be explained in detail along with an illustration of the situation. This illustration consists of a cross section along A-A in Figure 6.48 for the culvert construction, cross-section B-B for the outfall pipeline construction, and a top view of the works done at that location.

Phase 0: Current situation
During phase 0 no actual work at the location is done yet and is as the current situation. At this time the culvert sections are casted and prepared at the concrete plant. When all the sections are completed, and the bed level at the site location is prepared, they can be transported.

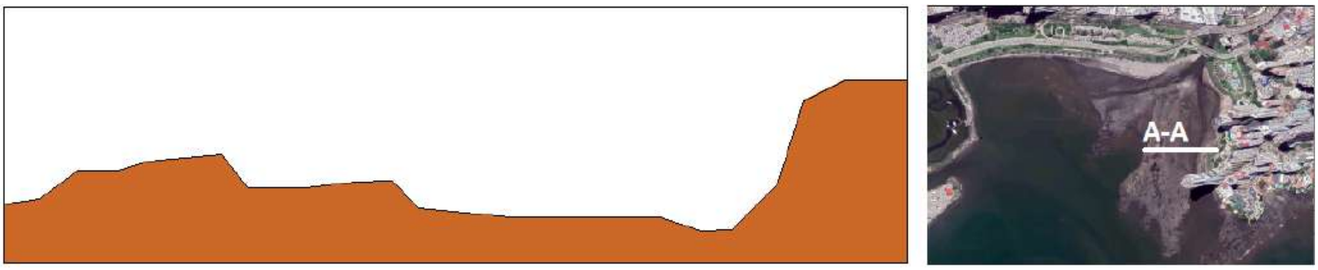


Figure 6.48: Phase 0: Current situation.

Phase 1: Drilling of bed level
The first phase involves drilling of the bed level to the appropriate depth for the culvert and outfall pipeline, see Figure 6.51. The required equipment depends on the characteristics of the rock. A basalt type of rock was found during earlier performed drillings in the area. It is expected that the rock can be cut using local accessible equipment such as a drum-cutter. The top layer of the basalt is softer, due to erosion, and easy to remove. For deeper layers the work can be more extensive and will take some more time. The drilled rock can be removed by excavators and dumped on the Eastern side next to the created trench. This layer of rock will provide a working path and will be explained in the next phase. Due to the high tidal variation it will only be possible to work during low tide.

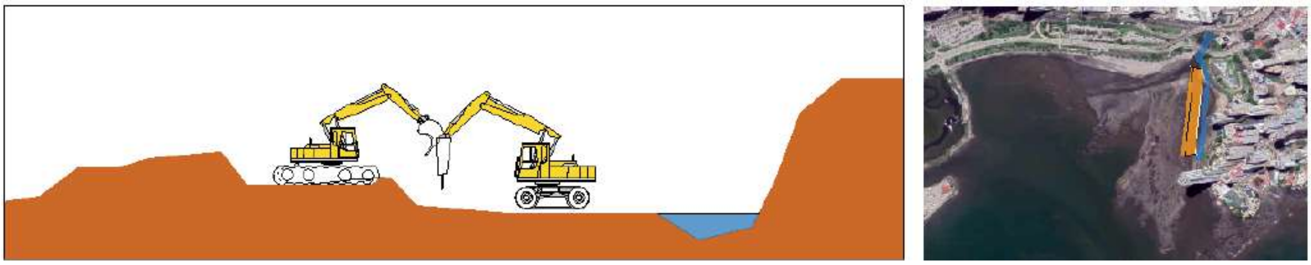


Figure 6.49: Phase 1: Drilling of bed level

Phase 2: Creation of work path

From the rubble retrieved from phase 1 a working path will be created on the Eastern side of the drilled trench (next to Punta Paitilla), see Figure 6.50. This working path will have two functions. The first will be to create an area from which the trucks and cranes can reach the worksite. The second function is to redirect the discharge of Matasnillo river to provide a dry worksite at the trench during low water. A Geo-textile will be placed in the work path to ensure that no water penetrates through the rock. This does not have to be a high-end quality Geo-textile since it will only be of use during construction of the culvert-outfall section.

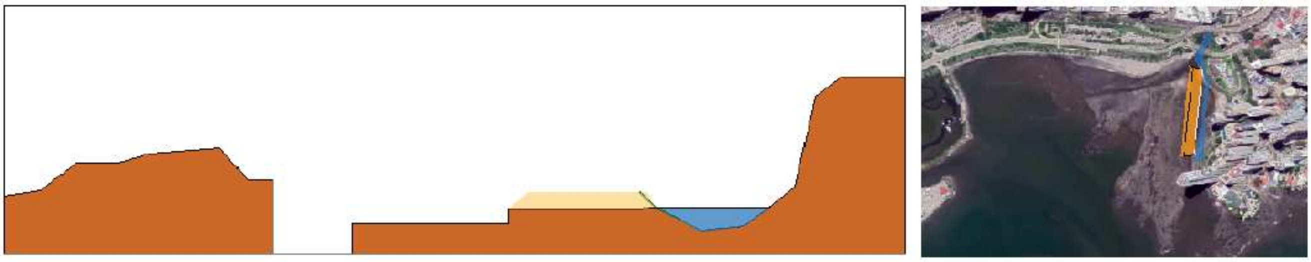


Figure 6.50: Phase 2: Creation of work path

Phase 3: Bed leveling & placement of gravel layer

When the sufficient depth has been reached the bed can be further leveled and prepared for placement of the culvert sections, see Figure 6.51. Before the sections are installed a gravel layer of 25 centimeters will be put in place along the culvert path. This will provide a base for the culvert to rest on and can serve as dewatering layer under the culvert.

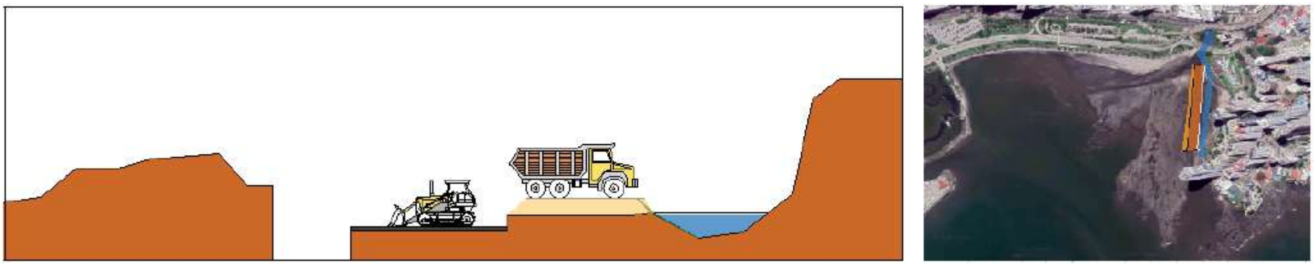


Figure 6.51: Phase 3: Bed leveling & placement of gravel layer

Phase 4: Placement of outfall & culvert sections

When the bed level is ready and prepared the outfall and culvert sections can be installed, see Figure 6.52. For proper handling and transport of the culvert sections a maximum weight of 30 tons was taken into consideration in the design phase. The concrete culvert sections can be delivered by deep loader truck over the work path. The work path itself is accessible by road from the marina. A crane located on the beach side of the temporary dams can lift the concrete sections from the trucks to their intended position on the gravel layer. Simultaneously, the first prepared HDPE outfall section can be brought in place.

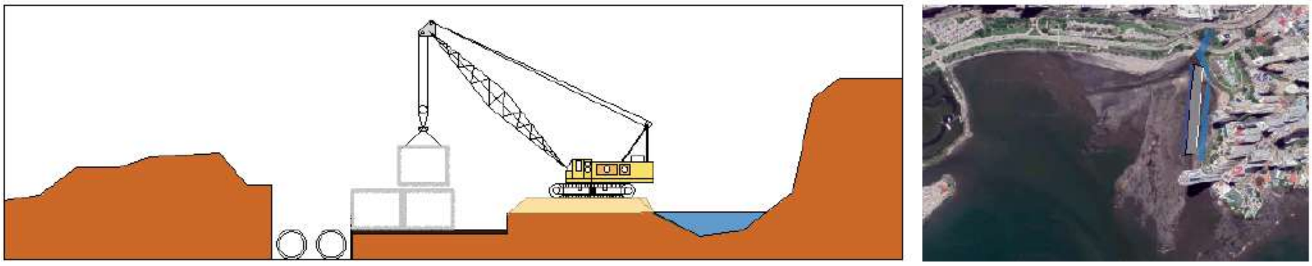


Figure 6.52: Phase 4: Placement of outfall & culvert sections

Phase 5: Removal work path
When the outfall pipeline and culvert are in place the work path can be removed. To safe costs the rubble is placed next to the culvert and over the outfall pipeline as in Figure 6.53. This will also provide a protective layer and some extra stability for the outfall pipelines against uplift during high water.

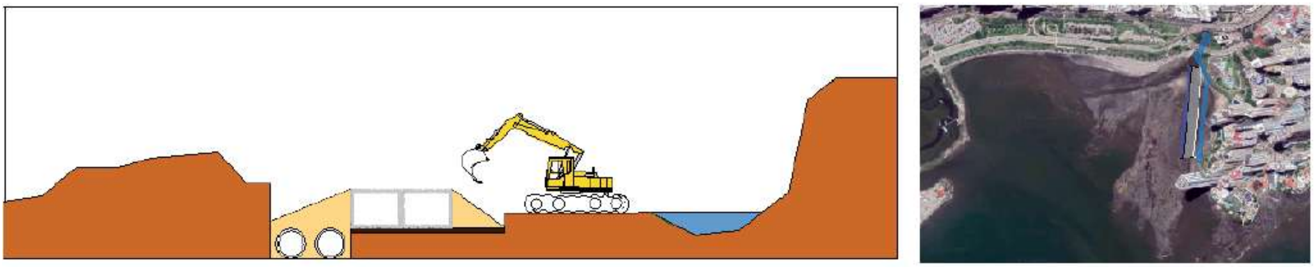


Figure 6.53: Phase 5: Removal work path

Phase 6: Construction of groynes & inlet structure
It is important that phase 6 is executed during the dry season, thus from December to March. During this time the discharge in Matasnillo river is lowest, because there will be hardly any rainfall. This makes it easy to collect water from the river and redirect it using pumps and pipelines. The inlet structure cannot be made from prefab sections and must be cast in-situ. Therefore, dry conditions are needed. To provide a dry working environment the mouth of Matasnillo river will be closed of using the same techniques as for the working path. Thus, using rubble from the drilled trench and Geo-textile. The side walls and floor slabs of the inlet structure can be delivered prefab to safe time. The pre-positioned outfall and outlet sections are connected with the inlet structure and cast in concrete.

Work on the groynes that will prevent deformation of the beach can also begin at this phase, since there will be no water flow from the river to the sea through the natural gully. The final configuration is shown in Figure 6.54.

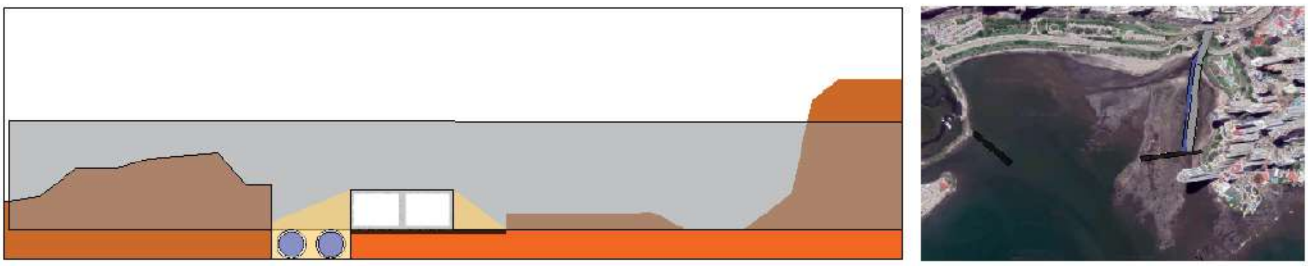


Figure 6.54: Phase 6: Construction of groynes & inlet structure

Phase 7: Beach creation
After all the outfall pipelines are connected and in place the final phase of the project can begin. At the inlet structure the pumps can be removed such that the water will flow through the outfall pipeline. The system can undergo a final check to look for possible leaks. When no failures are found the sand of the beach can be pumped from the dredging ship to the beach location. The sand is then positioned over the structure to create the desired beach profile, see Figure 6.55.

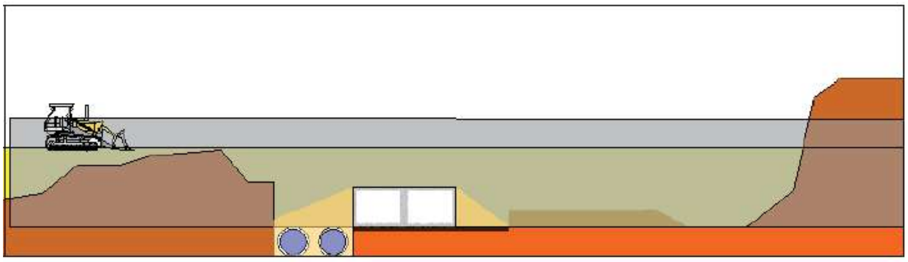


Figure 6.55: Phase 7: Beach creation

Phase 8: Trench dredging remaining outfall section
The second part of the outfall path follows a curved path from the groyne to the outlet location (see red area in Figure 6.56) and can't be reached by land. Along this path two soil conditions are expected. Close to shore the soil will be of the same basalt type as found during phase 2. The equipment used during that phase will cut this harder rock for as far as the low tide allows. For the section that the land-based equipment can't reach different material shall be used. Based on the expertise within Boskalis the decision was made that the best suitable equipment for the remaining section will be a backhoe dredging platform, see Figure 6.57. Such a dredging platform can operate in shallow conditions, with a maximum dredging depth of 18 meters (Treffers, 2018). The soil offshore consists of a mud layer which can be several meters thick. Under this mud layer the hard basalt rock is found. In the mud a trench needs to be dredged according to the dimensions indicated in Section 5.1.2.

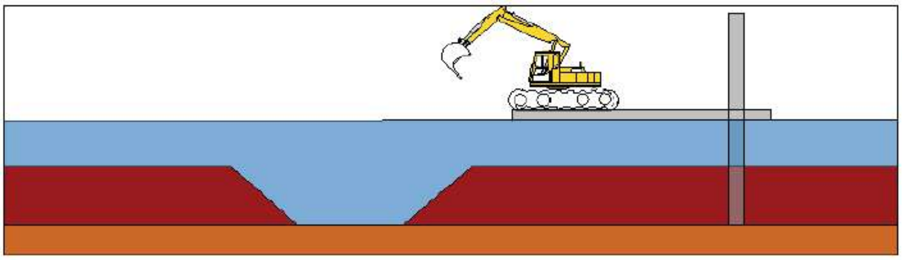


Figure 6.56: Phase 8: Trench dredging remaining outfall section

Figure 6.57: Boskalis backhoe dredging platform

Phase 9: Placing remaining outfall section and backfilling
After the trench has been dug the final outfall sections can be placed. The prepared sections float on water and are positioned by towboat, see Figure 6.59. At the location of the groyne the first section can be attached to the outfall section which is already in place on the beach during low tide. The total length of 1 km is difficult to install at once due to currents, waves and wind. Therefore, the outfall pipe is divided into 10 smaller sections of 100 meters and sunk separately. The connections are made with use of a special pontoon. A detailed description of the entire placement process of the outfall pipes is given in Appendix 6.1. When the outfall is in place the trench can be backfilled using the dredged material in the previous phase. A final layer of rubble is placed on top to protect it against anchors and other potential dangerous impacts, see Figure 6.58.

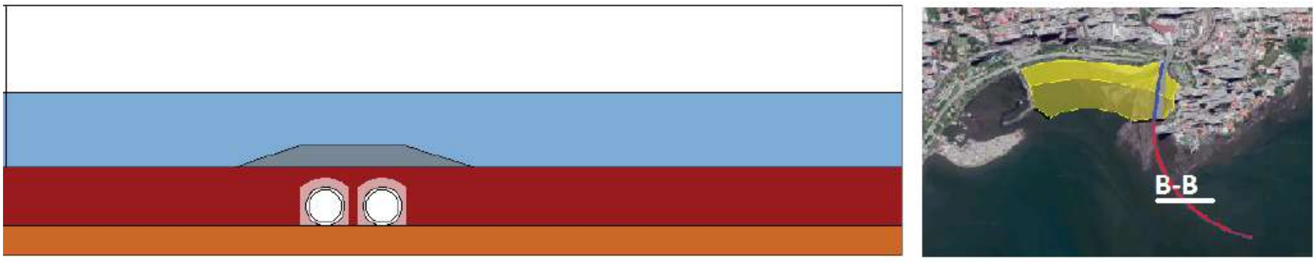


Figure 6.58: Phase 9: Placing remaining outfall section and backfilling

Figure 6.59: Positioning the outfall for the final section

6.2 Stability

Stability of the structure means that it may not move due to the loads acting on it, both during construction and operation. This implies that the following forms of stability need to be checked:

- Horizontal stability: horizontal forces acting on the structure based on a shallow foundation are transferred to the subsoil. The friction force of the subsoil should resist the resulting total acting horizontal force to prevent sliding. The friction force is determined by the forces acting in vertical direction on the structure. Horizontal stability is checked with the following equation:

$$\Sigma H < f * \Sigma V \tag{6.10}$$

- Rotational stability: rotational stability is achieved when the resulting action force intersects the core of the structure. Where the core is defined as the area extending 1/6 of the width on both sides of the middle of the structure. The rotational stability is checked with:

$$\frac{\Sigma M}{\Sigma V} \leq \frac{1}{6} b \tag{6.11}$$

- Vertical stability: For vertical stability the vertical effective soil stress, which is required to resist the acting loads ($\sigma_{k,max}$), may not exceed the maximum bearing capacity of the soil (p'_{max}). Or in other words $\sigma_{k,max} < p'_{max}$. The bearing capacity of the basalt rock at the project location is high and the acting load as a result of the weight of the culvert structure is relatively low. Therefore it is expected that the vertical stability criteria is fulfilled without further calculations.

6.2.1 Horizontal stability

As was briefly discussed in the previous section the horizontal stability criteria checks whether the vertical forces (ΣV) are large enough to prevent sliding generated by horizontal forces (ΣH) acting on the structure, according to Equation 6.10 and Figure 6.60. The friction coefficient f gives a relation of the interaction between the structure and the subsoil and is given by: $f = tan(\theta)$, where θ is the friction angle between the structure and subsoil . The friction factor for a concrete-gravel layer combination is 0.6 (Voorendt, 2019).

For the culvert structure the horizontal stability needs to be checked for construction steps 4 and 8. During construction step 3, when the culvert sections are being placed, the culvert section can be fully submerged. If the current is high enough the culvert sections can slide from their position. During construction step 6 the beach will be brought into the final position. It may occur that the sand is leveled up to the top of the structure on one side, while no sand is in place on the other. The stability will now be checked for both situations.

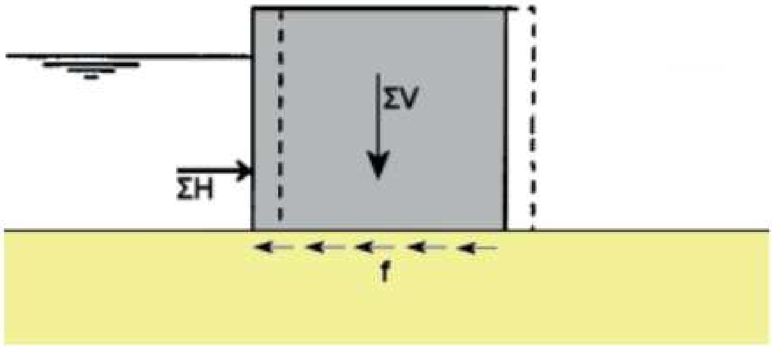


Figure 6.60: Principle of horizontal stability (Manual Hydraulic Structures, 2019)

Construction step 3: submerged placement
With the coming and going of the tide, the current will push against the walls of the culvert sections and could wash them away when strong enough, see Figure 6.61 . The horizontal force generated by the tide acting on the surface of the wall is given by:

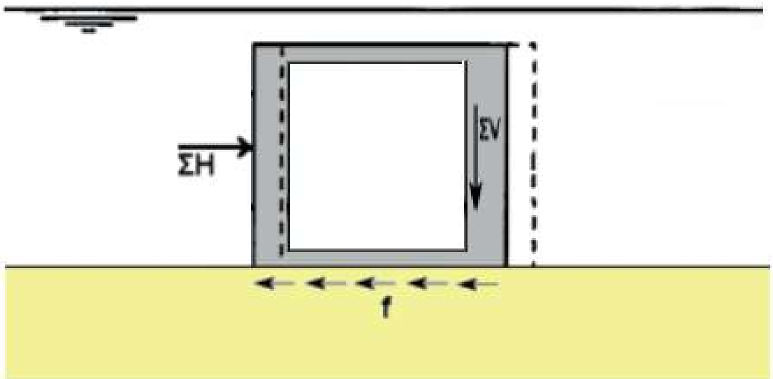


Figure 6.61: Principle of horizontal stability for submerged placement and tide (Manual Hydraulic Structures, 2019)

$$\Sigma H = \frac{1}{2} * \rho * v^2 * A \tag{6.12}$$

In which

ρ	=	Density of water (kg/m^3)
v	=	Flow speed of water (m/s)
A	=	Area of the wall (m)

The parameters used in the formula are all known, except the flow speed of the current. The density of the sea water is set to be 1025 kg/m^3 and the area of the wall subjected to the flow speed depends on the design variant an can be read in Table 6.21. For the flow speed an estimation has been made based on results of the flow model in Delft3D. In this model a maximum flow speed of 2 m/s occurred. For the calculations of the horizontal force acting on the structure a flow speed of 3 m/s is used to be safe.

When the culvert sections are submerged during high tide the vertical force of the sections is reduced due to the buoyancy. As a result the volumetric weight will be that of the volumetric weight of concrete minus the volumetric weight of water, or $\gamma_c - \gamma_w$. Based on the dimensions determined in Section 6.3.1 the vertical force can be calculated. The results for both forces are presented in Table 6.21, along with the stability check. It can be seen that both design variants are horizontal stable for the flow speed of 3 m/s.

Construction step 6: beach creation
Placement of the beach will occur under dry conditions. Therefore, the full weight of the culvert structure

Design variant	Wall surface area A (m^2)	Vertical force ΣV (kN)	Horizontal force ΣH (kN)	Friction coefficient f	ΣH/ ΣV*f
60 m^3/s	9	342	41.5	0.6	0.2
150 m^3/s	9.2	464.4	42.4	0.6	0.16

Table 6.21: Acting forces and check for horizontal stability for both design capacities for construction step 3: submerged placement

will act as vertical force. For the design variant of 60 m^3/s two sections are placed next to each other. The combined weight of these two sections will act as the total vertical force. Similarly, for the 150 m^3/s design variant, the combined weight of three sections will act as the total vertical force. The horizontal load acting on the structure is calculated for both design variants and is determined by the following equation:

$$\Sigma H = K_a * \frac{1}{2} * \gamma * d^2 * b \tag{6.13}$$

In which:

K_a = Active soil coefficient (0.33)
γ = Volumetric weight of soil (kN/m^3)
d = Depth of the soil layer acting on structure (m)
b = Width of the soil layer achting on structure (m)

The volumetric weight of sand was earlier set to be 20 kN/m^3. The depth and width of each section depends on the dimensions which were determined in Section 6.3.1. The active horizontal force is found using the vertical force and multiplying it with the active soil coefficient which is equal to 0.33. The resulting forces and check for horizontal stability for this construction step are presented in Table 6.22. For both design variants the structure is horizontally stable.

Design variant	Vertical force ΣV (kN)	Horizontal force ΣH (kN)	Friction coefficient f	ΣH/ ΣV*f
60 m^3/s	570	90	0.6	0.27
150 m^3/s	774	134	0.6	0.29

Table 6.22: Acting forces and check for horizontal stability for both design capacities for construction step 6: beach creation

6.2.2 Rotational stability

Overturning could occur when the acting moments on the structure are not counteracted by the vertical forces and the width of the base. The action line of the resulting force should intersect the core of the structure which is 1/6 of the width on each side of the centre of the structure as was written in Equation 6.11 and is shown in Figure 6.62. From the horizontal stability check it can be seen that the horizontal forces acting on the structure during Construction step 4: submerged placement are low compared to the vertical forces. From this it can be concluded that rotational stability will be achieved for construction step 3. Therefore, rotational stability is only checked for construction step 6 for individual sections.

The resulting forces and moments acting on the culvert sections for both design capacities are presented in Table 6.23. It can be seen that the structure is rotationally stable for the 60 m^3/s design variant but slightly unstable for the 150 m^3/s. This is the result of the higher moment acting on the structure due to the higher walls and less weight of the culvert section. However, the rotational stability check was performed for single culvert sections. In reality there will be three sections next to each other. In that configuration rotational stability will still be achieved.

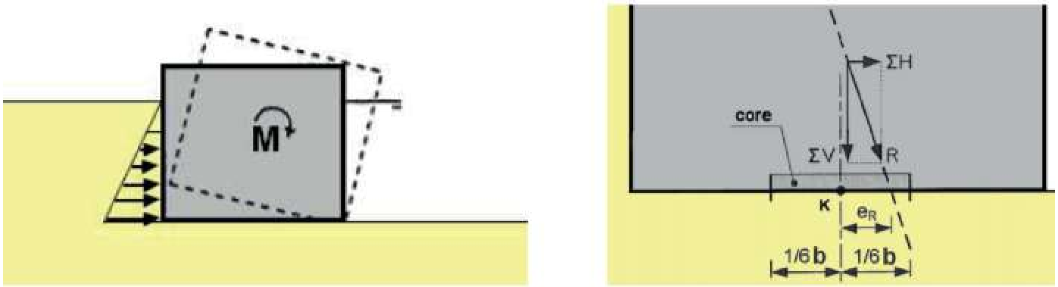


Figure 6.62: Principle of rotational stability (Manual Hydraulic Structures, 2019)

Design variant	Vertical force ΣV (kN)	Horizontal force ΣH (kN)	Moment ΣM (kNm)	Section width b (m)	(ΣM/ ΣV) / (1/6 b)
60 m^3/s	285	90	108	4.6	0.38
150 m^3/s	258	134	205.5	4.6	1.04

Table 6.23: Acting forces and check for rotational stability for both design capacities for construction step 6: beach creation

6.2.3 Vertical stability, piping & scour

The vertical stability does not need to be checked because the rocky subsoil at the project location will provide for support for the structure and additional loading of the beach. Therefore, no settlements are expected. The hard subsoil will also prevent two processes which can lead to instability over time, piping and scour. Piping occurs when groundwater flows under or besides the structure and erodes the material underneath the structure. In case of scour, erosion can occur when flow speeds exceed the maximum velocity for which the material in place remains stable. As a result scour holes can lead to instability of the structure. The rock basalt layer will not erode quickly and therefore, these instability processes will not occur during the design lifetime.

6.3 Strength: hybrid design culvert sections

The hybrid culvert sections are submitted to different forces during construction and operation. The governing situations for these forces to occur are during construction step 4 and 8 in Section 6.1. In construction step 4 the culvert sections are lifted into position and in step 8 the sections are in place and the structure is subjected to the weight from sand above it. For both steps the strength needs to be sufficient such that the structure will not fail. For both construction steps the dimensions of the walls and rebar in the concrete will be determined to provide the required strength.

6.3.1 Construction step 8: In operation - bending moment reinforcement

When the project is completed and the structure is in operation it will be subjected to the weight of the sand of the beach. The loads acting on the culvert section as a result of the weight are shown in Figure 6.63. Reinforcement steel inside the walls of the structure will be necessary to take the bending moments that are created by the loads. Elaborated calculations on the loads and reinforcement can be found in Appendix D.

Loads
The design load acting on each slab is determined for the Ultimate Limit State (ULS). Two different methods were applied to determine the design load for the horizontal and vertical slab of the structure and these will be discussed separately in the sections below. In Figure 6.63 the situation is illustrated together with the loads acting on the structure. The water level varies with the tide.

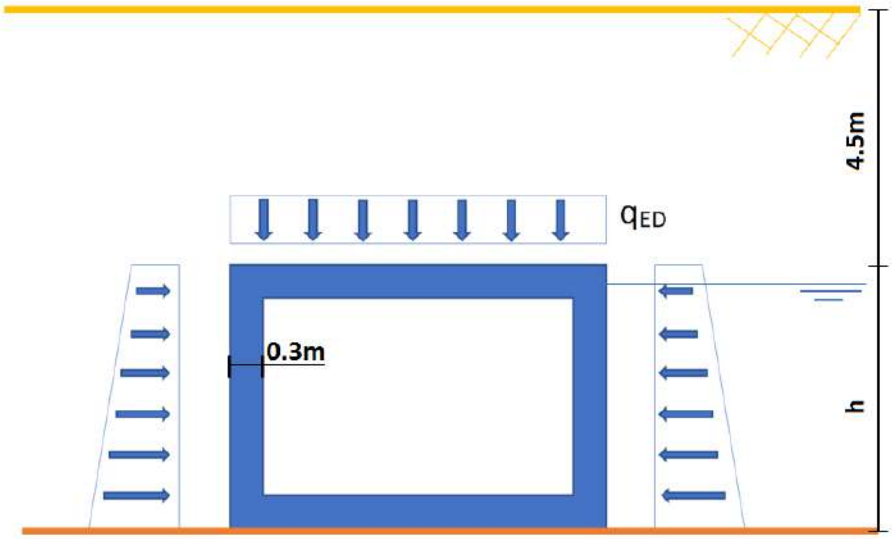


Figure 6.63: Forces acting on the structure

Horizontal slab
The design load q_{ed} acting on the horizontal slab consists of two parts: the dead weight load and the variable load, see Equations 6.14 and 6.15. These two formula contain the dead weight load q_{Gk} and a variable load q_{Qk}. The dead weight consists of the weight of the sand layer and the self weight of the concrete slab. The corresponding load factors γ_G and γ_Q are used to account for a safety factor in the load, since variations can occur in the weight of the concrete and soil. A variable load of 5 kN/m^2 is assumed to account for extra loading on top of the sand, for example an excavator driving over the structure. The parameters that are used in the calculations are listed in Table 6.24.

Wall thickness t (m)	Concrete weight γ_c (kN/m³)	Soil weight γ_s (kN/m³)	Sand layer d (m)	Variable load (kN/m²)	Permanent load factor γ_G	Variable load factor γ_Q
0.3	25	20	4.5	5	1.35	1.5

Table 6.24: Input parameters to compute the design load for horizontal slab.

$$q_{ed} = \gamma_G * q_{Gk} + \gamma_Q * q_{Qk} \tag{6.14}$$

$$q_{Gk} = \gamma_s * d + \gamma_c * t \tag{6.15}$$

In which:

q_{ed} = Design load on structure (kN/m^2)
γ_G = Permanent load factor (1.35)
q_{Gk} = Dead weight load (kN/m^2)
γ_Q = Variable load factor (1.5)
q_{Qk} = Variable load (kN/m^2)

The design loads acting on the horizontal slab were calculated to be 140 kN/m for both design variants (the step by step calculation can be found in Appendix D) and are presented in Figure 6.64. This load is identical for both design variants because the sand layer thickness on top of the structure is 4.5 meters in both cases.

Vertical slab
For the vertical slab a different approach is used. In this case the horizontal soil pressure acting on the vertical slab should be used. The starting value at the top of the slab is equal to active soil pressure above the horizontal slab plus the hydrostatic water pressure. The design load acting on the vertical slab was calculated according

to Formula 6.16. The load will increase linearly over the height and will therefore be largest at the base of the structure and different for both design variants. The results for the loads on the vertical slab are shown in Figure 6.64. Again the step by step calculations can be found in Appendix D.

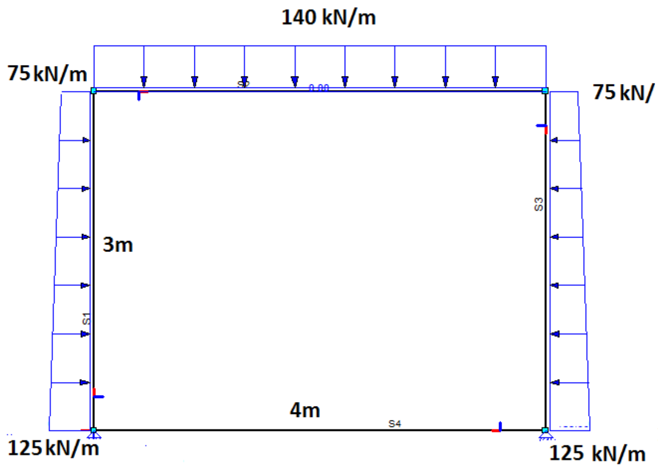
$$q_{ed} = \gamma_w * d + K_a * \gamma_s * d \tag{6.16}$$

In which:

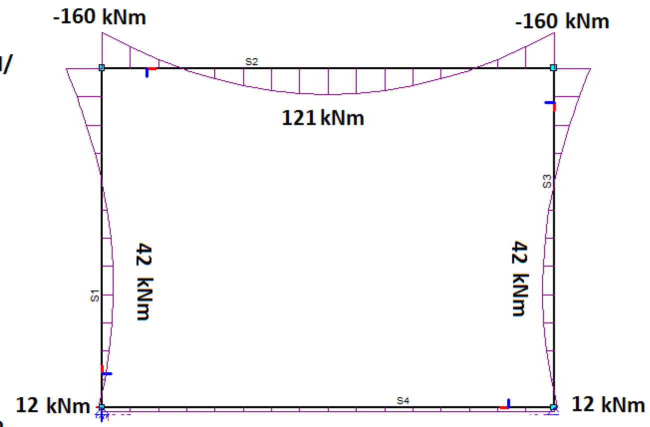
K_a	=	Active soil coefficient (0.33)
γ_d	=	Volumetric weight of soil (kN/m^3)
γ_w	=	Volumetric weight of water (kN/m^3)
d	=	Depth of the soil layer acting on structure (m)

Design moments

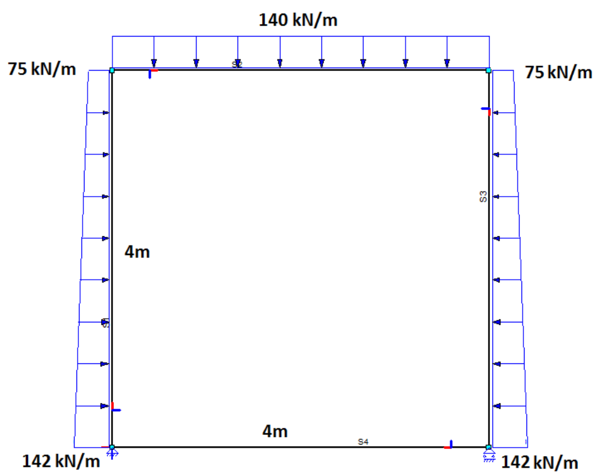
Next, the design moments are determined using the calculated design loads. This is done using modeling software called MatrixFrame. Within this software all kind of structures can be made and forces and loads can be applied to them. For this situation the culvert sections were modeled using a square frame with the dimensions of the 60 and 150 m^3/s design variants. At the bottom corner two fixed supports are placed to simulate that the structure is fixed on the ground. The results of the model can be seen in Figures 6.64b and 6.64d.



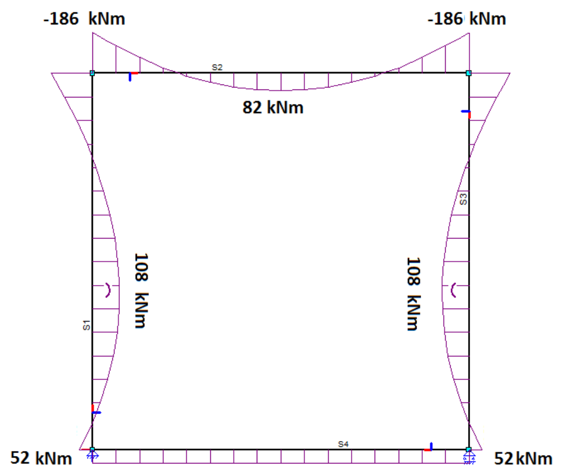
(a) Design load acting on structure for 60 m^3/s design variant



(b) Resulting design moments for 60 m^3/s design variant



(c) Design load acting on structure for 150 m^3/s design variant



(d) Resulting design moments for 150 m^3/s design variant

Figure 6.64: Forces acting on the structure for 60 and 150 m^3/s design variants

Required reinforcement steel

Now that the loads on the structure are known the required reinforcement steel can be determined. This is done using the model as pictured in Figure 6.65. Here a cross section of the slab is given with its internal forces. For this design, reinforcement steel of class B500B is used. The resistance moment M_{rd} created by the internal forces of the concrete compression force N_c and the steel force F_s needs to counteract the design moment The steel force Fs depends on the yield strength f_{yd} and the reinforcement steel area A_s.Therefore, a moment balance equation around the point of application of the concrete compression force is made and presented in Equation 6.17. This moment balance equation can be rewritten such that the required steel area can be calculated, see Equation 6.18. Finally, the diameter of the reinforcement steel ϕ can be determined by choosing a spacing distance s in the slab. The following values are used for the different parameters:

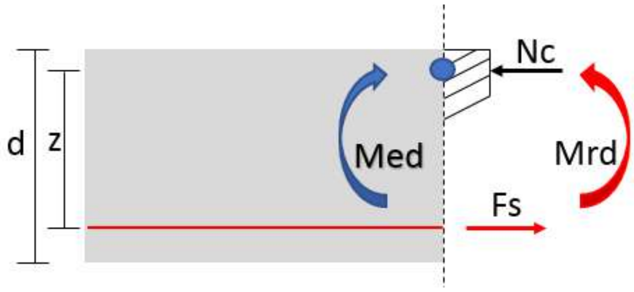


Figure 6.65: Forces acting on cross section in the slab

$$\Sigma M_{bluedot} = -M_{ed} + F_s * z = 0 \quad \rightarrow \quad \Sigma M_{bluedot} = -M_{ed} + A_s * f_{yd} * z = 0 \tag{6.17}$$

$$A_s = \frac{M_{ed}}{f_{yd} * z} \tag{6.18}$$

$$A_s = \frac{1}{4} * \phi^2 * \pi * \frac{1000}{s} \quad \rightarrow \quad \phi = \sqrt{\frac{A_s * 4}{\pi * \frac{1000}{s}}} \tag{6.19}$$

In which:

M_{ed}	=	Design bending moment (Nmm)
F_s	=	Force in steel rebar, $F_s = A_s * f_{yd}$ (N)
z	=	Inernal lever arm 0.9*d (270 mm)
A_s	=	Steel rebar area (mm^2)
f_{yd}	=	Reinfocement steel yield strength (435 N/mm^2)
s	=	Spacing of rebar over widht (200 mm)
ϕ	=	Diameter of steel rebar (mm)

The results of the calculations for the minimum required reinforcement steel are presented in Table 6.25 and 6.26 for the 60 and 150 m^3/s design, respectively. For both the horizontal and vertical slabs a wall thickness of 300 mm was assumed and the rebar spacing was chosen to be 200 mm. It can be seen that the minimal required diameter for the 60 and 150 m^3/s design variant is close to each other , 19 to 21 mm. To be on the safe side and to make construction of the culvert sections easier, all the slabs will be made with 25 mm diameter reinforcement steel for both design variants.

$60\ m^3/s$	Max. design moment M_{ED} (kNm)	Internal lever arm z (mm)	Spacing s (mm)	Required steel area A_s (mm^2)	Steel bar diameter ϕ (mm)
Horizontal slab	160	270	200	1362	19
Vertical slab	42	270	200	360	10

Table 6.25: Input parameters and resulting rebar dimensions for the 60 m^3/s design

$150\ m^3/s$	Max. design moment M_{ED} (kNm)	Internal lever arm z (mm)	Spacing s (mm)	Required steel area A_s (mm^2)	Steel bar diameter ϕ (mm)
Horizontal slab	186	270	200	1538	20
Vertical slab	108	270	200	919	16

Table 6.26: Input parameters and resulting rebar dimensions for the 150 m^3/s design

Final culvert section dimensions

Now that the strength calculations are done and the wall thickness of 0.3 meter is proven to be sufficient, the final dimensions of both the culvert designs can be determined. The inner width and height were calculated before to provide sufficient discharge capacity, so only the length per section is left. For handling and placement of the culvert sections the maximum limiting weight was set to be 30 tonnes. With this limitation the lengths of each culvert section could be determined for the 60 and 150 m^3/s design variants and turned out to be 2.5 and 2 meters, respectively.

The number of culvert sections that are needed in the overall culvert structure can be now also be calculated with the earlier defined lengths of each design variant. The length of the culvert will be 400 meters. For the 60 m^3/s design variant two rows of culvert sections are needed and for the 150 m^3/s design three rows. Using Equation 6.20 the number of culvert sections are calculated. The final culvert dimensions and required sections for each variant are listed in Table 6.27.

$$\#_{sections} = \frac{L_{culvert}}{l_{section}} * \#_{rows} \tag{6.20}$$

In which:

$$\begin{aligned}
\#_{sections} &= \text{Number of culvert sections required } (-) \\
L_{culvert} &= \text{Total length of culvert (m)} \\
l_{section} &= \text{Length per culvert section (m)} \\
\#_{rows} &= \text{Number of rows } (-)
\end{aligned}$$

Design variant	Inner width per section (m)	Inner height per section (m)	Length per section (m)	Wall thickness (m)	Total weight (tonnes)	Number of sections
$60\ m^3/s$	4	3	2.5	0.3	28.5	320
$150\ m^3/s$	4	4	2	0.3	25.8	600

Table 6.27: Dimensions for both culvert design variants

6.3.2 Construction step 4: Lifting culvert sections - shear force reinforcement

During construction the culvert sections will be subjected to different loading when lifted into their final position. The sections will be lifted by ropes in each top corner of the structure. This will mean that the half the weight of the structure is subjected on each side as is pictured in Figure 6.66. This will cause shear forces in the horizontal slab. The dimensions and reinforcement that were computed in the previous section are used here to check the shear force for both the design capacities. This means a wall thickness of 300 mm (with a effective width height of 270 mm) and reinforcement steel with a diameter of 28 mm with a spacing of 200 mm. The shear force is checked without shear reinforcement.

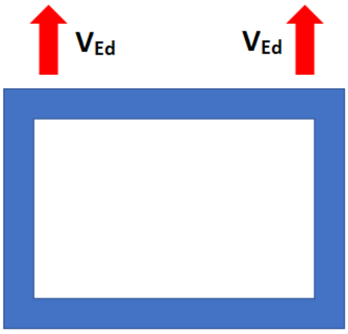


Figure 6.66: Forces acting on culvert section during lifting

Shear force without shear reinforcement

Without shear reinforcement the shear forces have to be taken by the web in the slab. For practical reasons there will be stirrups added in the slabs but these are not taken into consideration here. The acting shear force V_{Ed} depends on the weight which was determined in the previous sections. The shear resistance of the slab can be calculated with Equations 6.21 and 6.22. From these two equations the minimum value has to be taken into consideration for the shear force unity check, V_{Ed}/V_{Rd}.

$$V_{Rd,c} = [C_{Rd,c} * k * (100 * \rho_1 * f_{ck})^{1/3} + k_1 * \sigma_{cp}] * b_w * d \tag{6.21}$$

$$V_{Rd,min} = (v_{min} + k_1 * \sigma_{cp}) * b_w * d) \tag{6.22}$$

In which:

f_{ck} = characteristic compressive strength of concrete (N/mm^2)

k = $1 + \sqrt{\frac{200}{d}} \leq 2.0$ (−)

ρ_1 = reinforcement ratio in longitudinal reinforcement $\frac{A_s l}{b_w * d} \leq 0.02$ (−)

b_w = smallest width of cross-section in the tensile area (mm)

σ_{cp} = compressive stress in the concrete $\frac{N_{ed}}{A_c} \leq 0.2 * f_{cd}$ (N/mm^2)

N_{ed} = Axial force in the cross section (N)

A_c = Area of the concrete cross-section (mm^2)

k_1 = Coefficient: 0.15 (−)

$C_{Rd,c}$ = Coefficient: $0.18/\gamma_c = 0.12$ (−)

v_{min} = $0.035 * k^{3/2} * f_{ck}^{1/2}$ (N/mm^2)

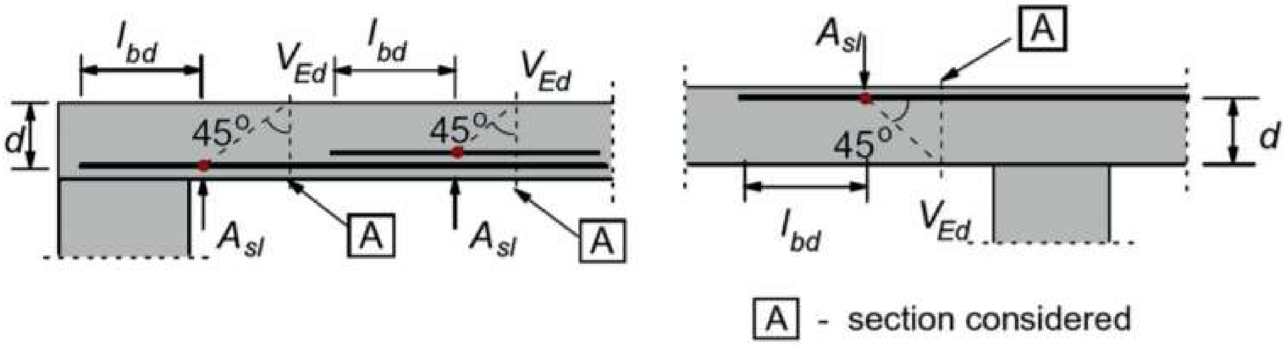


Figure 6.67: Principle of shear force without shear reinforcement(Manual Hydraulic Structures, 2019)

Results shear force check

For both design capacities the shear force was checked. Because of the height of the walls and the difference in weight also different forces occur in the slab. The results of the computations for all the needed parameters are listed in Table 6.28. It can be seen that the results for V_{R_min} are smaller than $V_{Rd,c}$, so these values are used for the unity check. The unity check turned out to be safe for both design capacities. This means that the slabs are safe without using extra shear reinforcement. An elaboration on these calculations can be found in Appendix D.

Design variant	f_{ck} (MPa)	$C_{rd,c}$	k	ρ_1	b_w (mm)	σ_{cp} (N/mm)	N_{Ed} (kN)	A_c (mm^2)	k_1	v_{min} (MPa)	$V_{Rd,c}$ (kN)	$V_{R,min}$ (kN)	V_{Ed} (kN)	UC
60 m^3/s	45	0.12	1.86	0.01	1000	0.86	232.5	2.7*10^5	0.15	0.544	249	181.7	143	0.79
150 m^3/s	45	0.12	1.86	0.01	1000	1.19	320	2.7*10^5	0.15	0.544	263	195	129	0.66

Table 6.28: Input parameters and results for shear force calculations with unity check.

Chapter 7. Verification of hybrid concepts & decision

In this chapter a final decision is made between the 60 and 150 m^3/s design variants based on the construction time, costs and flood risk. It was desired to incorporate the opinions of the stakeholders in this final evaluation but unfortunately this could not be done. Therefore, it was decided to make the final decision between the two design variants based on:

- Construction time
- Costs
- Flood risk

7.1 Construction time

To get an impression of the total duration of construction a time planning was made. For each design phase the duration was determined. This was done by looking at the required work that has to be done per design phase and estimate the capacity per day of the used equipment. For several design phases it was difficult to estimate an exact number for the capacity. For these phases, a final number of days needed was estimated. For each design phase the equipment, capacity and duration is given in Table 7.29.

Design phase	Equipment needed	Number of equipment	Capacity per day	Days needed	
				60 m^3/s	150 m^3/s
0: Casting & curing (30 days)	Formwork	10	3 sections	142	240
1: Drilling	Drum cutter	2	200 m^3	46	71
2: Create work path	Excavator	2	-	14	14
3: Gravel layer	Excavator	2	-	14	14
4: Placing culvert & outfall	Cranes	2	8 sections	40	75
5: Remove work path	Excavator	2	-	7	7
6: Groyne & inlet structure	Various	-	-	89	89
7: Beach creation	Various	-	-	91	91
8: Trench dredging	Backhoe dredger	1	2500 m^3	21	21
9 Placing outfall	Connection platform	1	0.5 sections	20	20

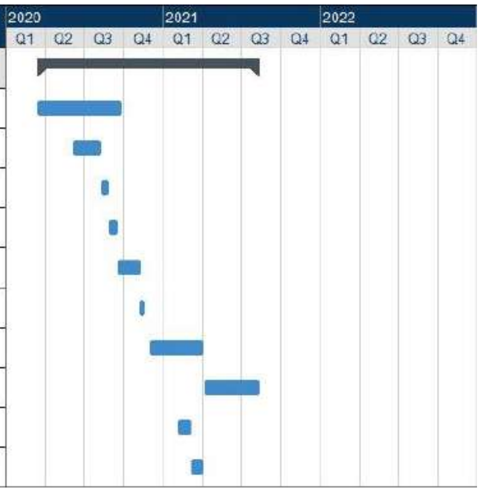
Table 7.29: Time planning per design phase

7.1.1 Gantt chart

To present the construction time per phase in an orderly manner a Gantt chart is used. In such a Gantt chart time periods can be assigned for each construction phase. The most import criteria ,while planning the time periods for each phase, is that phase 6: Groyne & inlet structure takes place during the dry season from December till March. All the other phases were planned in such a way that the necessary work is completed at the start of the dry season. To shorten the total construction time some phases are planned simultaneously when possible. For example, production of the culvert sections can take place at the same time as when the bed level is drilled and prepared. The Gantt charts for both design capacities are shown in Figure 7.68.

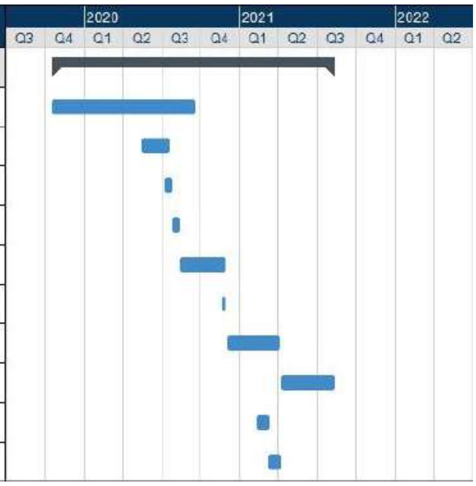
It can be seen that the total project duration is 368 and 474 days, for the 60 and 150 m^3/s design respectively. The main reason for this is the longer time needed for the casting of all the culvert sections in the 150 m^3/s design. The dry season deadline can still be met without causing delays by starting production of the culvert section earlier.

	Task Name	Duration	Start	End
1	**Total project**	**368 days**	**12-3-2020**	**9-8-2021**
2	Phase 0: Casting & curing	142 days	12-3-2020	25-9-2020
3	Phase 1: Drilling bed level	46 days	4-6-2020	6-8-2020
4	Phase 2: Creation work path	14 days	7-8-2020	26-8-2020
5	Phase 3: Gravel layer	14 days	26-8-2020	14-9-2020
6	Phase 4: Placing culvert & outfall	40 days	14-9-2020	6-11-2020
7	Phase 5: Removal work path	7 days	6-11-2020	16-11-2020
8	Phase 6: Groyne & inlet structure	89 days	1-12-2020	2-4-2021
9	Phase 7: Beach creation	91 days	5-4-2021	9-8-2021
10	Phase 8: Trench dredging	21 days	4-2-2021	4-3-2021
11	Phase 9: Outfall placing	21 days	5-3-2021	2-4-2021



(a) Gantt chart for time planning of construction for 60 m^3/s design variant

	Task Name	Duration	Start	End
1	**Total project**	**474 days**	**16-10-2019**	**9-8-2021**
2	Phase 0: Casting & curing	240 days	16-10-2019	15-9-2020
3	Phase 1: Drilling bed level	46 days	14-5-2020	16-7-2020
4	Phase 2: Creation work path	14 days	6-7-2020	23-7-2020
5	Phase 3: Gravel layer	14 days	24-7-2020	12-8-2020
6	Phase 4: Placing culvert & outfall	75 days	13-8-2020	25-11-2020
7	Phase 5: Removal work path	7 days	18-11-2020	26-11-2020
8	Phase 6: Groyne & inlet structure	89 days	1-12-2020	2-4-2021
9	Phase 7: Beach creation	91 days	5-4-2021	9-8-2021
10	Phase 8: Trench dredging	21 days	8-2-2021	8-3-2021
11	Phase 9: Outfall placing	21 days	8-3-2021	5-4-2021



(b) Gantt chart for time planning of construction for 150 m^3/s design variant (MindView)

Figure 7.68: Gantt chart for time planning of construction for 60 and 150 m^3/s design variants (MindView)

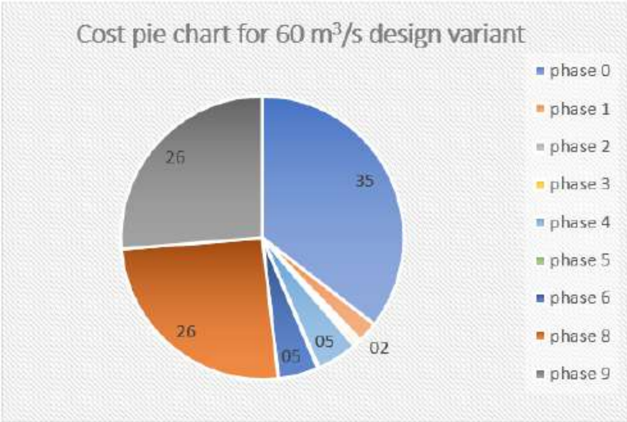
7.2 Costs

The costs for the project were determined for each construction phase and for each design capacity apart from the costs for the beach and groynes. So phase 7: beach creation is left out and for phase 6: Groyne & inlet construction only the costs for the inlet structure were taken into consideration. The results are presented, per phase, in terms of percentages of the total costs in Table 7.30. The costs for the 150 m^3/s design variant are expressed in percentages of the total costs of the 60 m^3/s design variant. In Figure 7.69 pie charts are presented that give percentages of the total costs for each construction phase.

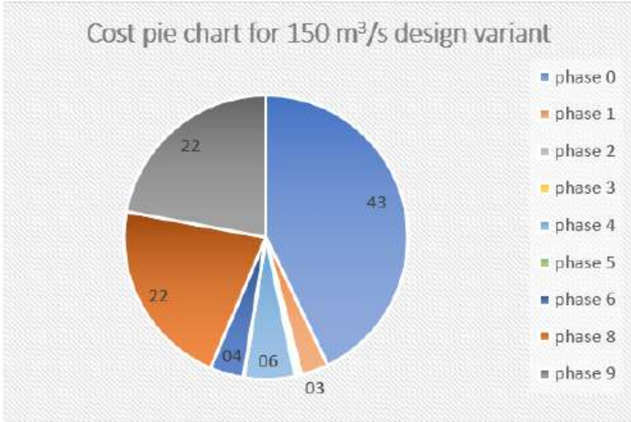
The 150 m^3/s design variant is more costly as was expected, but only by 19%. This can be explained by the costs for phase 8 and 9. These works involve the dredging and placement of the outfall pipelines and are the same for both design variants. This fraction accounts for 50% of the total costs. The 19% increase is mainly generated by the increase in culvert sections and longer placement time of these sections in the 150 m^3/s design variant.

Design variant	Phase 0	Phase 1	Phase 2	Phase 3	Phase 4	Phase 5	Phase 6	Phase 8	Phase 9	Total
60 m^3/s	35.5	2.4	0.4	0.5	4.7	0.1	4.6	25.6	26.3	100
150 m^3/s	50.9	3.7	0.4	0.5	6.8	0.1	4.5	25.6	26.3	118.8

Table 7.30: Percentages of total costs for each design capacity



(a) Percentages of total costs for 60 m^3/s design variant



(b) Percentages of total costs for 150 m^3/s design variant

Figure 7.69: Pie charts with percentages of total costs for each design capacity

7.3 Flood risk

Flooding occurs when the discharge in the river exceeds the design capacity of the culvert structure. This flood risk can be expressed in terms of a probability that at least one event, where the river discharge exceeds the design capacity of the culvert, occurs. This probability can be determined by using Formula 7.23.

$$R = \left(1 - \left(1 - \frac{1}{T}\right)^n\right) * 100\% \tag{7.23}$$

In which:

R = Probability of at least one event exceeding design capacity (%)
T = Return period (years)
n = Expected lifetime of structure (20 years)

The expected lifetime of the structure was defined in the structural requirements and was set to be 20 years. The remaining unknown parameter is the return period associated with the design capacity of the culvert design variants. In other words, the return periods for a 60 and 150 m^3/s river discharge need to be determined. In Section 2.3.3 the river discharged was approximated by multiplying the rainfall intensity with the drainage area and a runoff coefficient. In this case, the rainfall intensity for the 60 and 150 m^3/s discharge can be retrieved directly from the rational method equation. Rewritten this equation reads:

$$i = \frac{Q}{C * A} \tag{7.24}$$

In which:

Q = Design discharge (m^3/s)
C = Runoff coefficient (−)
i = Rainfall intensity (m/s)
A = Drainage area (m^2)

The calculations to find the required rainfall intensity to produce the 60 and 150 m^3/s river discharge can be made since the drainage area and runoff coefficient remain 8.4 km^2 and 0.6, respectively. The results can be found in Table 7.31.

In Section 2.3.3, the rainfall intensity was determined for four return periods using the inverse form of the Generalized Extreme Value distribution, see Equation 7.25.

$$I(d) = \mu - \frac{\sigma}{\xi}\left(1 - (-\log(1 - \frac{1}{T}))^{-\xi}\right) \quad \text{if } \xi \neq 0 \tag{7.25}$$

From the equation it can be seen that the return period T can be retrieved when the rainfall intensity $I(D)$ and the remaining parameters ξ, σ and μ are known. In Section 2.3.3 these parameters were defined for different time durations. It was shown that the 60 minutes rainstorm duration resulted in a flow speed that came closest to the observed flow speed during a rainstorm. Therefore, the parameters ξ, σ and μ for this computation were chosen to be equal to the 60 minutes duration and are -0.0387, 12.2268 and 59.3800, respectively.

Now, the return period associated with the previously determined rainfall intensities can be calculated using Formula 7.25. The resulting return periods turned out to be 1 and 30 years for the 60 and 150 m^3/s discharge, respectively. Finally, using Equation 7.23 the probability of exceedance was determined for both design variants. The results can be found in Table 7.31.

Design variant	Rainfall intensity (mm/hr)	Return period (years)	Probability of exceedance over lifetime (%)
60 m^3/s	42.9	1	100
150 m^3/s	107.1	30	50

Table 7.31: Results for computations of probability of exceedance for each design capacity.

Though it seems that the probability of exceedance for the lifetime of the 150 m^3/s design variant is unacceptably high, it has to be taken into account that this probability is computed under the least favourable conditions. Thus during high water at spring tide. The real probability of exceedance will be much lower since this high water level is only approached several hours each month.

7.4 Decision

In the previous sections it became clear that, while the construction time of the 150 m^3/s design variant is 100 days longer, the costs of this variant only increase with 19% compared to the 60 m^3/s design variant. This is a relatively small increase in investment because the discharge capacity increases with a factor 2.5. The flood risk for both designs showed that the probability of exceedance for the 60 m^3/s design variant is 100%, while the 150 m^3/s design variant is 50 % (it should be noted that these values are calculated for the least favourable conditions during high water levels). Both have a design lifetime of 20 years. Based on the risk analysis and the relatively small increase in costs of 19% the decision was made to advise the 150 m^3/s design variant as final design.

Chapter 8. Final design & recommendations

In this final chapter the final design, based on the found solutions on the problem requirements and boundary conditions, is shown. Furthermore, some recommendations are given to further improve the design.

8.1 Final design

The main objective of this project was to come up with a solution for the nuisance caused by odour and poor water quality from Matasnillo river, such that the water quality along the Cinta Costera area in Panama City is sufficient for recreational use and a beach can be made at the location. To realise this the main objective was divided into the following three functional requirements:

- Water quality in the bay has to meet standards to be fit for recreational use. From the water samples it was clear that the main pollutant in the water of Matasnillo is of fecal origin. For fecal coliforms and E.Coli bacteria the tolerable limit is 250 CFU/100 ml. For the Total coliforms the limit is a maximum of 500 CFU/100 ml.
- During rainy season the river must have sufficient capacity to discharge the water and will not cause floodings. Probabilistic analysis of the rain data resulted in a discharge of 150 m^3/s with a return period of 20 years. This will be the upper limit for the design capacity.
- Remove the odour that originates from the river area.

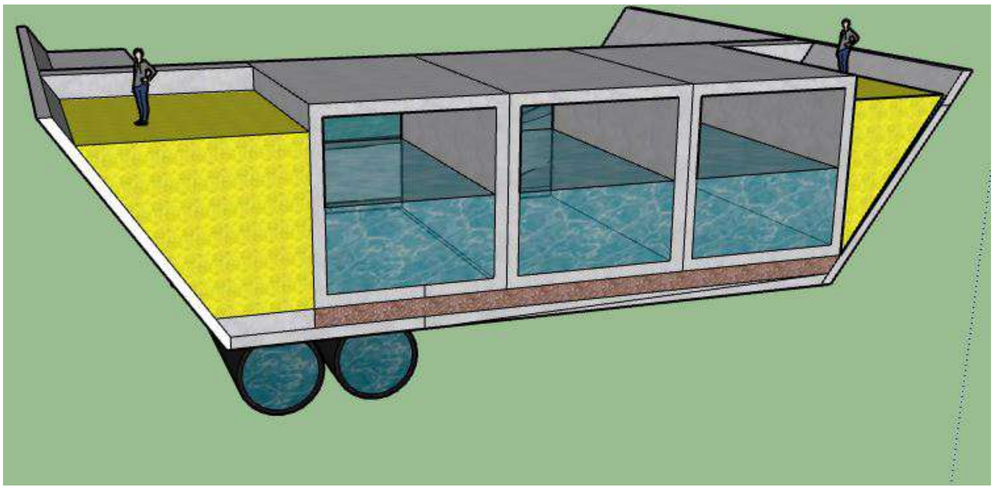
From the early conceptual design phase, the pipeline and hybrid concept looked most promising to meet the functional requirements. The "soft-engineering" and "non-engineering" solutions can be used in a supportive function but will not be able to meet all the requirements by them self. When the pipeline concept was tested for the discharge capacity it turned out that the capacity was around 57 m^3/s. The lower limit was set to be 60 m^3/s and therefore the pipeline concept was dismissed. The hybrid concept was able to meet the functional requirements and was worked out in further detail for a discharge capacity of 60 and 150 m^3/s. This was done because of uncertainty in the maximum discharge during rainstorms. In the end it was decided to choose the 150 m^3/s design variant, because this design is able to discharge 2.5 times the volume of water compared to the 60 m^3/s design variant while the costs are only 19 % higher. The hybrid concept of 150 m^3/s meets the requirements for the following conditions:

Water quality
The water quality at the beach is under the tolerable limits in case the outlet of the outfall pipeline is located at 1.4 km South-East from the mouth of Matasnillo river. In this configuration the tolerable limits for E.Coli bacteria, fecal coliform bacteria and total coliforms are not exceeded. In Figure 8.70 the configuration of the complete structure (culvert and outfall pipeline) is shown.

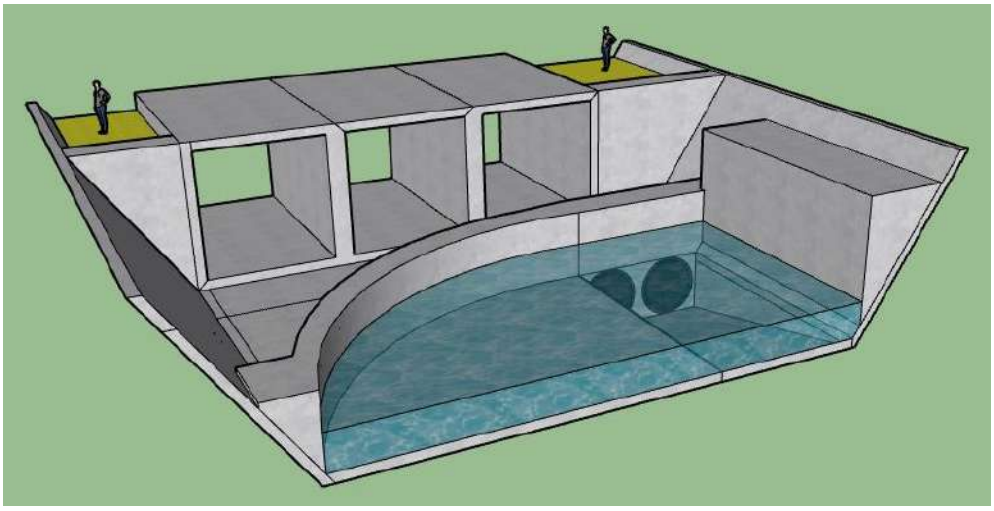
Figure 8.70: Top view of the position for the hybrid concept at Bella Vista beach.

Discharge capacity

The culvert area needed to provide sufficient discharge capacity was determined based on a hydraulic head balance. The culvert needed a total area of 48 m^2 in order to guarantee the discharge capacity of 150 m^3/s under spring tide conditions. This area was designed into 3 sections with an inner width of 4 meters and an inner height of 4 meters (3x4x4). The culvert inlet structure is shown in Figure 8.71a in a situation with high water. For the outfall pipelines, two pipes with an inner diameter of 2 meters were designed in order to provide the capacity needed to discharge the polluted water inside the river at high tide without accumulation of water at the inlet structure. The inlet structure is designed in such a way that the polluted water from the river is collected under dry conditions and discharged through the outfall pipelines. The inlet structure design can be seen in Figure 8.71b.



(a) Beach side cross-section of culvert inlet structure for the 150 m^3/s design capacity at a high water event.



(b) River side view at culvert inlet for 150 m^3/s design capacity at low tide.

Figure 8.71: Culvert inlet structure for 150 m^3/s design capacity at river mouth

Odour

The odour at the river area will be solved by 4 measures:

- Choosing the inlet location at a distance of 125 meters from the beach.
- Remove polluted sludge that has settled in the river and around the river mouth.
- Provide a discharge capacity in the outfall pipeline that is capable to discharge the water in the river under dry conditions such that nu polluted water builds up in front of the inlet structure.
- The Sanitation Program will continue their work to improve the water quality in the river.

A final part of the overall design will be the construction of a garbage fence (as was presented in Figure 3.25), which will be located further upstream of the inlet structure. This garbage fence will prevent rubble and garbage from entering the outfall pipelines, which can cause clogging of the system.

8.2 Recommendations

Throughout the course of the project several points of attention came to light which can be improved to get more reliable results and/or boundary conditions. This will lead to a better design. The most important improvements are given as recommendations for further research and are listed below. For each recommendation an explanation is given.

- Hydraulic survey
- Bathymetry survey and land mapping
- Improvement of the Delft3D model
- Lower discharge capacity during dry conditions for outfall pipeline
- Perform drillings to determine the actual soil conditions along the outfall pipeline path

Hydraulic survey
The most important criterion that was unknown during the project was the maximum discharge capacity. Approximations of the discharge were made based on the rational method in combination with rain data and estimates made by employees of Boskalis on location. However the rational method also relies on the drainage area and runoff coefficient. The drainage area was determined by looking at height maps of the area. This drainage area has a large effect on the final computed discharge. To get a better insight in the maximum discharge a hydraulic survey can be executed. During the rainy season the discharge can be measured in Matasnillo river. The discharge that occurs at a rainstorm of a certain intensity can then be compared with the historical data set. From this better predictions can be made for different rainfall intensities.

Bathymetry survey and land mapping
The bathymetry data that was used inside the Delft3D model dated from July 1018. It is unlikely that the bathymetry has changed significantly over the course of 1 year. However, it was unclear to what reference level the water depths were given. It was tried to get the correct depths in the model based on observations during the site visit and known water levels.To be sure that the numbers are correct, a new bathymetric survey can be performed along with a land height mapping. In such a way the bathymetry of the seabed and the land heights will have the same reference level. Then, no mistakes in height differences will occur.

Lower discharge capacity during dry conditions for outfall pipeline
The two outfall pipelines with a diameter of 2 meters each were designed under the conditions that the water level in the river must be able to reach the water level at sea. Especially during neap tide, when the water level differences are smaller, this required a larger area of the pipes. When a small accumulation of polluted water is permitted in front of the inlet structure, the area of the outfall pipeline can be reduced. This will also have a large effect in terms of costs since the construction phases for dredging and placing the outfall pipeline account for around 50% of the total costs. In case a single, 2 meter diameter, pipeline can be used the costs for these two construction phases would be reduced with a factor 2. The total costs will then be reduced by 25%. However, accumulation of polluted water could lead to an increase of odour. So this measure should be discussed with the stakeholders.

Improvement of the Delft3D model
The Delft3d model that was created to model the water quality could be further improved. A very basic model was set-up due to inexperience with the program and a limited amount of time. Inside the flow model of the overall model an error occurred at the boundary between water and land at Flamengo island. This resulted in an inconstancy of water levels and, in turn, to high flow speeds at this boundary that propagated further. Since this location seemed to have no effect on the flow conditions at the location of interest it was decided that the model could still be used. However, a better and more detailed model could further improve the generated results of the water quality.

Perform drillings to determine the actual soil conditions along the outfall pipeline path

For the soil conditions at the project location it was assumed that the soil is similar compared to the soil at Punta Pacifica and the two artificial islands. From these locations drillings existed from earlier projects. For the placement of the outfall pipeline it is important to know how deep the sludge layer offshore of Punta Paitilla is, because it will take more time (and therefore more money) to create a trench in harder material. Soil drillings should be performed along of the intended path of the outfall pipeline to get a good insight in the soil conditions.

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Appendix A

Site servey

This Appendix gives a representation of the site survey done in Panama City at the project location, to give a good impression of the area. Photos were taken at the location during high and low tide.

During the first week of my trip to Panama I visited the site location several times to get an impression. The most striking to me was the difference between high and low water. In the series of photos below the difference between high and low water can clearly be seen. The low water pictures were taken on 8th of January when low tide was at CD +0.4 m. The high water pictures were taken on the 18th of January when the high tide was at CD +4 m. At spring tide the water can reach up to a minimum of CD - 1 m and a maximum of CD +5.5 m. A second point of attention is the difference in water color, as can clearly be seen in Figure A.5 and A.7.

The first series of pictures show the situation at mouth of matasnillo river A.1, A.2, A.3, A.4. Figure A.5 shows the river mouth under the pedestrian crossing and Figure A.6 the river mouth on top of this pedestrian crossing. Finally Figure A.7 shows the conditions at Hard rock hotel several hundred meters upstream.

(a) River mouth low tide **(b)** River mouth high tide

Figure A.1: Water conditions during low and high water at river mouth

(a) River mouth low tide **(b)** River mouth high tide

Figure A.2: Water conditions during low and high water at river mouth

(a) River mouth low tide

(b) River mouth high tide

Figure A.3: Water conditions during low and high water at river mouth

(a) River mouth low tide

(b) River mouth high tide

Figure A.4: Water conditions during low and high water at river mouth

(a) River mouth low tide

(b) River mouth high tide

Figure A.5: Water conditions during low and high water at river mouth under pedestrian crossing

(a) River mouth low tide

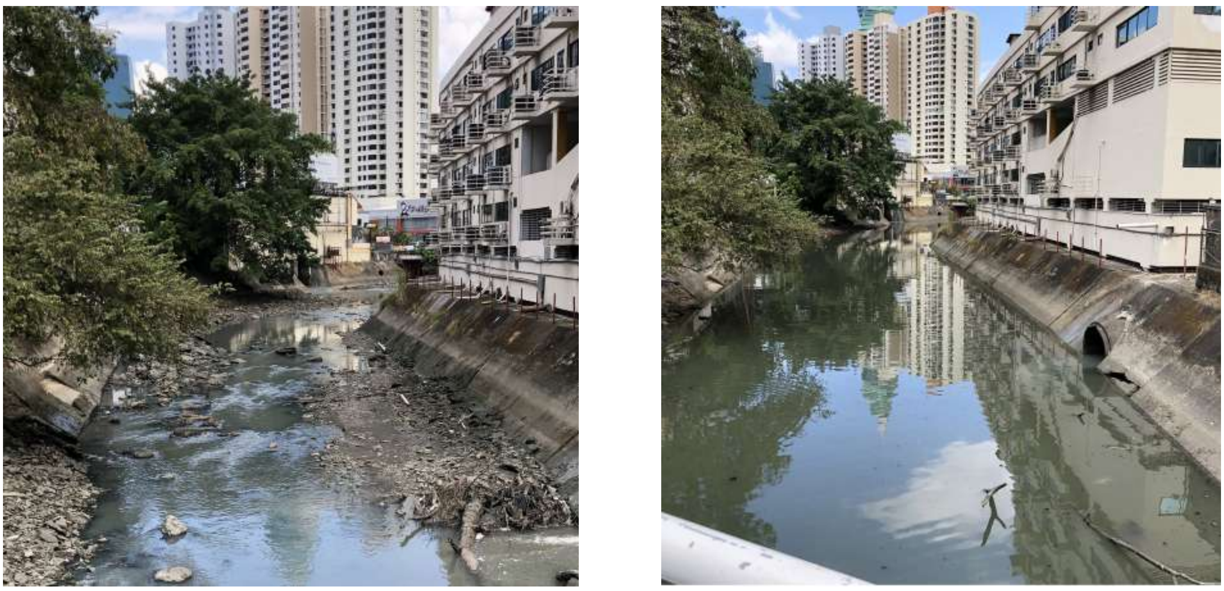
(b) River mouth high tide

Figure A.6: Water conditions during low and high water at river mouth from pedestrian crossing

(a) Hardrock low tide

(b) Hardrock high tide

Figure A.7: Water conditions during low and high water at Hard rock hotel



(a) Hardrock low tide

(b) Hardrock high tide

Figure A.8: Water conditions during low and high water at Hard rock hotel

Skybridge
Another interesting sight were two connections from the large shopping mall Multicentro to a nearby building on the other side of the river. From the outside it doesn't look like much, but on the inside it is nice and cool, see Figure A.9. This crossing could be used for a connection from the high density tourist area (Hard Rock hotel etc.) to the beach and reduce the traveling effort.

Figure A.9: Outside and inside of Skybridge connection over river

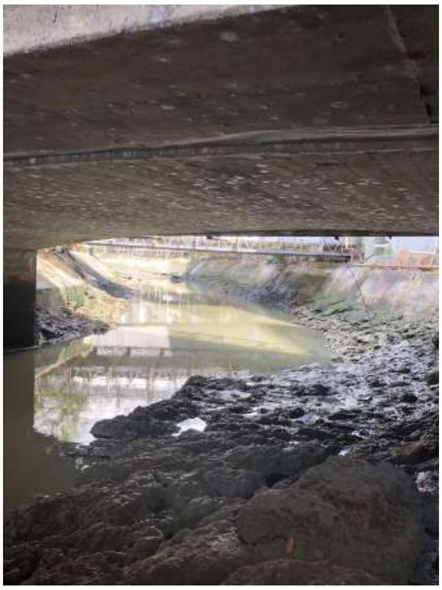
Appendix B

Measurements & Data

To define the boundary conditions a number of measurements were done and data was gathered. Some more detailed information on how the measurements were performed and how the methods work will be provided in this Appendix. Relative data gathered from external sources are also further explained on how they were retrieved by that source.

B.1 River dimensions & infrastructure

To measure the dimensions, widths, heights and lengths of the river and existing infrastructure a standard 50 meter measuring tape was used as in Figure B.1b, together with a colleague (Manuel) of the Boskalis Panama office. On one side one would hold and read the measuring tape while the other would take the end of the tape to other side. However conditions at the location were far from ideal and hard to reach as can be seen in Figure B.1a. The site was slippery, dirty and still some water was flowing trough the river. Therefore it was impossible to reach some areas. To still get the required measurement, the tape was connected to a rock and thrown to the location that couldn't be reached by foot. In this way the end of the tape would not be at the ideal point so therefore the measurement which was read was adjusted with a certain value that was thought to be the distance that was off from the wanted point.



(a) Conditions under highway crossing

(b) Measurement tape used

Figure B.1: measurements

GPS

The longitudinal profile of the river was measured using a GPS device at two locations in the river, see Figure 2.11. The GPS measuring method includes two receivers, that are connected to several satellites, see Figure B.2. From the difference in received signal from the satellites an accurate location and altitude can be determined. The more satellites that are connected to the receivers the more accurate the reading is. The first receiver is a fixed base station from which the position is known. During the measurements this fixed base station was located on the East-Island of Panama City. This is an artificial island that Boskalis has realized by the end of 2018 after the first West-island. The second receiver is the portable receiver that is taken to the desired location. The base station and portable receiver were calibrated in such a way that the reading given by the portable receiver gave the altitude relative to CD 0 m.

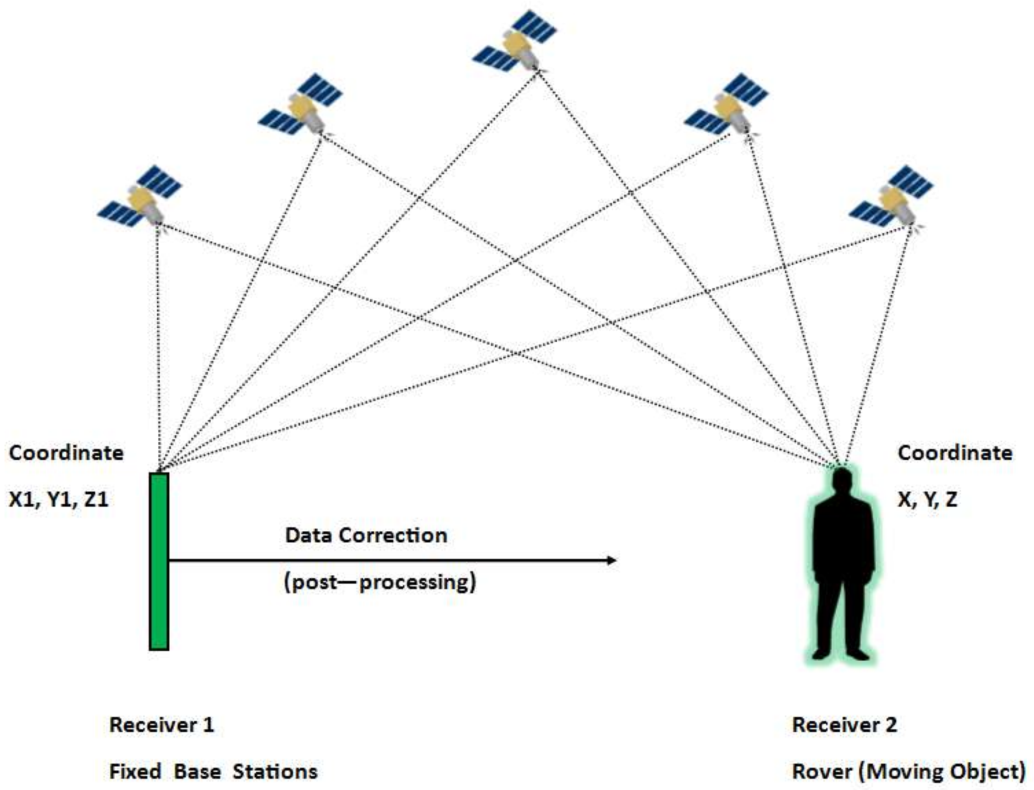


Figure B.2: GPS measuring method (Pearlywhisper, 2019).

B.2 Hydraulic conditions

The hydraulic conditions in the river were determined based on rainfall data from measuring station Bilboa Heights in Panama City. In the analysis the maximum rainfall intensity was retrieved from the data set per 15 minutes. The results for the summed up rainstorms can be seen in Figure B.3a. In Figure B.3b the same data is transformed into hourly averages.

Generalized Extreme value distribution

The GEV distribution is a family of continuous probability distributions to combine the Gumbel, Frechet and Weibull (type I, II and III, respectively) extreme value distributions,to allow a continuous range of possible shapes . Each of these distribution has its own characteristic shape. The cumulative distribution function of the GEV is given by Equation B.1.

$$F(x) = exp\left(-\left(1+\xi\left(\frac{x-\mu}{\sigma}\right)\right)^{\frac{-1}{\xi}}\right) \tag{B.1}$$

In which:

$$\begin{aligned} \mu &= \text{Location paramater} \\ \sigma &= \text{Scale parameter} \\ \xi &= \text{Shape parameter} \end{aligned}$$

The GEV distribution is parameterized with a location, scale and shape parameters, μ, σ and ξ respectively. The shape parameter ξ is governing for the tail behavior. For $\xi = 0$, $\xi > 0$, $\xi < 0$, the GEV leads to the Gumbel, Frechet and Weibull distribution, respectively as can be seen in Figure as can be seen in Figure B.4.

The parameters μ, σ and ξ that were used in the analysis were retrieved for each time duration from the maximum annual rainfall intensity datasets in Figure B.3b using a Matlab function called *gevfit*. The input for this

Maximum annual rainfall intensity for given duration (mm)						
	15	30	45	60	75	90
1993	33	40.5	56	66	73.6	81
1994	30.5	50.8	63.5	63.5	68.6	71.1
1995	30.5	50.8	61	71.2	76.3	81
1996	27.9	38	48.2	60.9	66	66
1997	30.5	53	78.8	86.4	101.6	112
1998	22.9	43	53.4	66.1	73.7	76.6
1999	43.2	71	86.2	106.7	114.2	116.7
2000	30.5	50.8	61	66	68.5	71
2001	22.9	38	43	48.3	56	61
2002						
2003	25.4	40.6	48	61	68.6	71.1
2004	33	56	63.5	67	67	67
2005	22.9	38.1	43.1	48.3	68.6	83.5
2006	30.5	40.7	55.9	61	64	64
2007						
2008						
2009	27	38	65	77	82	93
2010	29	57	70	91	105	111
2011	30	51	65	75	76	84
2012						
2013	26	34	37	45	45	45
2014	29	51	59	62	62	62
2015	22	37	47	55	55	56
2016	34	41	56	65	68	71
2017	24	28	33	44	59	63

(a) Maximum accumulative annual rainfall intensity for given duration (mm)

Maximum annual rainfall intensity for given duration (mm/hr)						
	15 min	30 min	45 min	60 min	75 min	90 min
1993	132	81	74.7	66	58.9	54.0
1994	122	101.6	84.7	63.5	54.9	47.4
1995	122	101.6	81.3	71.2	61.0	54.0
1996	111.6	76	64.3	60.9	52.8	44.0
1997	122	106	105.1	86.4	81.3	74.7
1998	91.6	86	71.2	66.1	59.0	51.1
1999	172.8	142	114.9	106.7	91.4	77.8
2000	122	101.6	81.3	66	54.8	47.3
2001	91.6	76	57.3	48.3	44.8	40.7
2002	0	0	0.0	0	0.0	0.0
2003	101.6	81.2	64.0	61	54.9	47.4
2004	132	112	84.7	67	53.6	44.7
2005	91.6	76.2	57.5	48.3	54.9	55.7
2006	122	81.4	74.5	61	51.2	42.7
2007	0	0	0.0	0	0.0	0.0
2008	0	0	0.0	0	0.0	0.0
2009	108	76	86.7	77	65.6	62.0
2010	116	114	93.3	91	84.0	74.0
2011	120	102	86.7	75	60.8	56.0
2012	0	0	0.0	0	0.0	0.0
2013	104	68	49.3	45	36.0	30.0
2014	116	102	78.7	62	49.6	41.3
2015	88	74	62.7	55	44.0	37.3
2016	136	82	74.7	65	54.4	47.3
2017	96	56	44.0	44	47.2	42.0

(b) Maximum annual rainfall intensity for given duration (mm/hr).

Figure B.3: Maximum annual rainfall intensity for given duration (mm/hr)

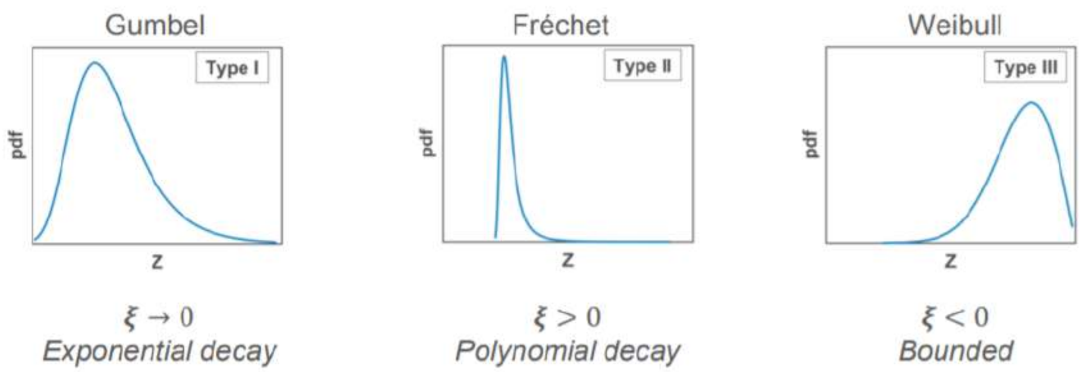


Figure B.4

function is the data set of maximum annual rainfall intensity for a certain time duration. As output the function returns the variables for ξ, σ, μ that best fit the data set. The Mathworks website that provides information on all kinds of Matlab related functions gives the following description regarding *gevfit*:

parmhat = gevfit(X) returns maximum likelihood estimates of the parameters for the generalized extreme value (GEV) distribution given the data in X. parmhat(1) is the shape parameter, k, parmhat(2) is the scale parameter, sigma, and parmhat(3) is the location parameter, mu.

$[parmhat, parmci]$ = gevfit(X) returns 95% confidence intervals for the parameter estimates.

When k < 0, the GEV is the type III extreme value distribution. When k > 0, the GEV distribution is the type II, or Frechet, extreme value distribution. If w has a Weibull distribution as computed by the wblfit function, then -w has a type III extreme value distribution and 1/w has a type II extreme value distribution. In the limit as k approaches 0, the GEV is the mirror image of the type I extreme value distribution as computed by the evfit function.

In Figure B.5 the used code in Matlab is shown. For each time duration the maximum annual rainfall intensity is listed. In this example the results of the parameters for the 60 minutes duration are shown under variables.

```
max15 = [132, 122, 122, 111.6, 122, 91.6, 172.8, 122, 91.6, 122, 108, 116, 120, 104, 116, 88, 136, 96];
max30 = [81, 101.6, 101,6, 76, 106, 86, 142, 101,6, 76, 81,2, 112, 76.2, 81.4, 76, 114, 102, 68, 102, 74, 82, 56];
max45 = [74.7, 84.7, 81.3, 64.3, 105.1, 71.2, 114.9, 81.3, 57.3, 64, 84.7, 57.5, 74.5, 86.7, 93.3, 86.7, 49.3, 78.7, 62.7, 74.7, 44];
max60 = [66, 63.5, 71.2, 60.9, 86.4, 66.1, 106.7, 66, 48.3, 61, 67, 48.3, 61, 77, 91, 75, 45, 62, 55, 65, 44];
max75 = [58.9, 54.9, 61, 52.8, 81.3, 59, 91.4, 54.8, 44.8, 54.9, 53.6, 54.9, 51.2, 65.6, 84, 60.8, 36, 49.6, 44, 54.4, 47.2];
max90 = [54, 47.4, 54, 44, 74.7, 51.1, 77.8, 47.3, 40.7, 47.4, 44.7, 55.7, 42.7, 62, 74, 56, 30, 41, 37.3, 47.3, 42];


variables = gevfit(max90)
```

```
variables =
      xi,     sigma,      mu,
   -0.0339     9.8503    45.5979
```

Figure B.5: Matlab code used to retrieve the parameters μ, σ and ξ

Discharge calculations

The discharge in the river was also determined using the exact rainfall intensity data. Using this information the discharge for each rainstorm can be calculated using an extended form of Formula 2.2. At each time step the discharge can be calculated using the rainfall intensity data and the contributing isochrone area at that time step according to Equation B.2. So at time time step 1 the discharge is determined by the rainfall intensity of the first 15 minutes of a rainstorm over isochrone area A1. At time step 2 the discharge is determined by the rainfall intensity of the second 15 minutes over isochrone area A1 plus the contributing part of the rainfall intensity over isochrone area A2. This process repeats for each time step. The isochrone areas can be seen in Figure B.6.

$$
\begin{aligned}
Q1 &= C*(A1*i_1)\\
Q2 &= C*(A1*i_2 + A2*i_1)\\
Q3 &= C*(A1*i_3 + A2*i_2 + A3*i_3)\\
Q4 &= C*(A1*i_4 + A2*i_3 + A3*i_2 + A4*i_1)
\end{aligned}
\tag{B.2}
$$

In which:

$$
\begin{aligned}
Q_i &= \text{Discharge at time step i } (\mathrm{m^3/s})\\
A_i &= \text{Area of isochrone } (\mathrm{m^2})\\
i_i &= \text{Rainfall intensity at time step i } (\mathrm{m/15min})
\end{aligned}
$$

Year	1993	1994	1995	1996	1997	1998	1999	2000	2001	2002	2003	2004	2005
Precipitation (mm/rainstorm)	81	77	118	92	112	87	106	71	61		113	67	86
River Discharge (m^3/s)	93	83	88	67	120	94	151	95	70		86	95	65

Year	2006	2007	2008	2009	2010	2011	2012	2013	2014	2015	2016	2017	2018
Precipitation (mm/rainstorm)	66			97	123	106		45	62	56	73	64	84
River Discharge (m^3/s)	64			89	127	89		60	88	77	90	57	

Missing data year   Incomplete data year

Table B.1: River discharge compared to the maximum rainfall in given year for 20 years of data.

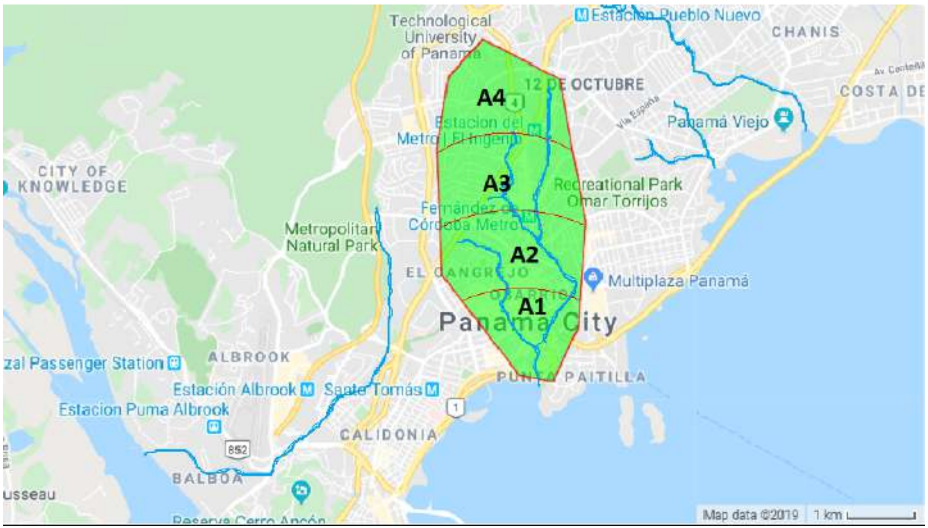
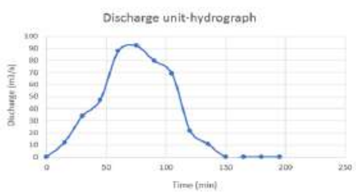


Figure B.6: River discharge compared to the maximum rainfall in given year for 20 years of data.

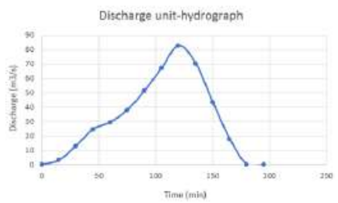
The results of the discharge calculations can be seen in Table B.1. The highest discharge was calculated for 1999 and is 150 m^3/s. This corresponds with the statistical analysis where the discharge with a 20 year return period was calculated to be 141 m^3/s. In addition, for each year a unit hydrograph was made based on the discharge calculations to see how the discharge reacts at the rainfall intensity. These can be seen in the figures below for each year with data. Compared to the rain data of other rainstorms it was found that rainstorms with a short duration and high intensity led to a higher discharge compared to rainstorms which have a longer duration.

Date	time	mm/15min
21-5-1993	0:30:00	10,16
21-5-1993	0:45:00	15,24
21-5-1993	1:00:00	7,62
21-5-1993	1:15:00	33,02
21-5-1993	1:30:00	7,62
21-5-1993	1:45:00	7,62
	Total	81 mm



(**a**) Rain data & hydrograph for largest rainstorm in 1993.

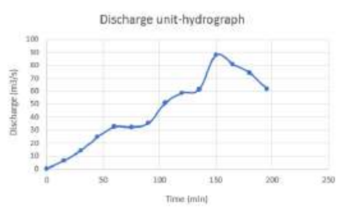
Date	time	mm/15min
1-11-1994	9:00:00	2,54
1-11-1994	9:30:00	7,62
1-11-1994	9:45:00	7,62
1-11-1994	10:00:00	2,54
1-11-1994	10:15:00	10,16
1-11-1994	10:30:00	17,78
1-11-1994	10:45:00	17,78
1-11-1994	11:00:00	12,7
	Total	77mm



(**b**) Rain data & hydrograph for largest rainstorm in 1994.

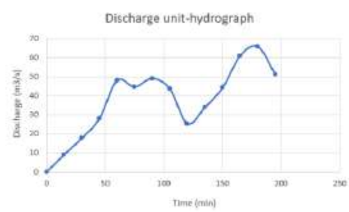
Figure B.7: Rain data & discharge hydrograph

Date	time	mm/15min
15-5-1995	13:15:00	5,08
15-5-1995	13:30:00	5,08
15-5-1995	13:45:00	7,62
15-5-1995	14:00:00	5,08
15-5-1995	14:15:00	5,08
15-5-1995	14:30:00	7,62
15-5-1995	14:45:00	20,32
15-5-1995	15:00:00	7,62
24-5-1995	14:00:00	7,62
24-5-1995	14:15:00	30,48
24-5-1995	14:30:00	10,16
24-5-1995	14:45:00	2,54
	Total	118 mm



(**a**) Rain data & hydrograph for largest rainstorm in 1993.

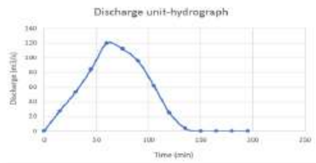
Date	time	mm/15min
30-5-1996	1:45:00	7,62
30-5-1996	2:00:00	5,08
30-5-1996	2:15:00	7,62
30-5-1996	2:30:00	15,24
30-5-1996	2:45:00	2,54
30-5-1996	3:00:00	10,16
30-5-1996	3:15:00	2,54
30-5-1996	3:30:00	2,54
30-5-1996	3:45:00	10,16
30-5-1996	4:00:00	17,78
30-5-1996	4:15:00	12,7
30-5-1996	4:30:00	5,08
	Total	92 mm



(**b**) Rain data & hydrograph for largest rainstorm in 1993.

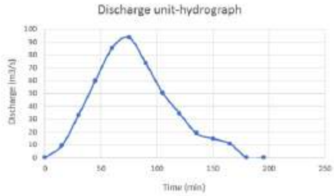
Figure B.8: Rain data & discharge hydrograph

Date	time	mm/15min
1-10-1997	12:30:00	22,86
1-10-1997	12:45:00	15,24
1-10-1997	13:00:00	22,86
1-10-1997	13:15:00	25,4
1-10-1997	13:30:00	15,24
1-10-1997	13:45:00	2,54
	Total	112 mm



(a) Rain data & hydrograph for largest rainstorm in 1997.

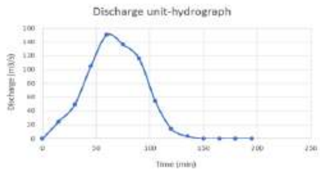
Date	time	mm/15min
25-5-1998	13:00:00	7,62
25-5-1998	13:15:00	17,78
25-5-1998	13:30:00	17,78
25-5-1998	13:45:00	17,78
25-5-1998	14:00:00	12,7
25-5-1998	14:15:00	2,54
25-5-1998	14:30:00	2,54
25-5-1998	14:45:00	7,62
	Total	87 mm



(b) Rain data & hydrograph for largest rainstorm in 1998.

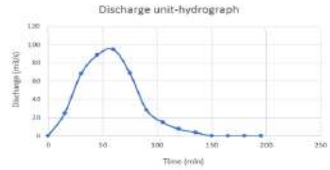
Figure B.9: Rain data & discharge hydrograph

Date	time	mm/15min
15-7-1999	8:45:00	20,32
15-7-1999	9:00:00	15,24
15-7-1999	9:15:00	43,18
15-7-1999	9:30:00	27,94
15-7-1999	9:45:00	7,62
15-7-1999	10:00:00	2,54
	Total	106 mm



(a) Rain data & hydrograph for largest rainstorm in 1999.

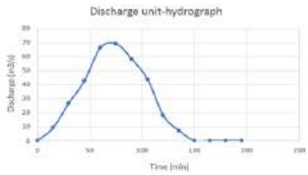
Date	time	mm/15min
21-6-2000	16:15:00	20,32
21-6-2000	16:30:00	30,48
21-6-2000	16:45:00	10,16
21-6-2000	17:00:00	5,08
21-6-2000	17:15:00	2,54
21-6-2000	17:30:00	2,54
	Total	71 mm



(b) Rain data & hydrograph for largest rainstorm in 2000.

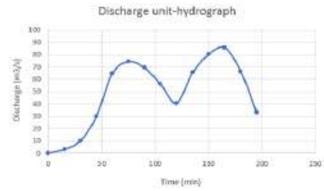
Figure B.10: Rain data & discharge hydrograph

Date	time	mm/15min
15-5-2001	15:15:00	7,62
15-5-2001	15:30:00	12,7
15-5-2001	15:45:00	10,16
15-5-2001	16:00:00	17,78
15-5-2001	16:15:00	7,62
15-5-2001	16:30:00	5,08
	Total	61 mm



(a) Rain data & hydrograph for largest rainstorm in 2001.

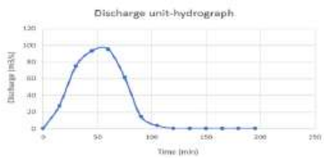
Date	time	mm/15min
27-4-2003	3:30:00	2,54
27-4-2003	3:45:00	5,08
27-4-2003	4:00:00	15,24
27-4-2003	4:15:00	25,4
27-4-2003	4:30:00	5,08
27-4-2003	4:45:00	2,54
27-4-2003	5:00:00	7,62
27-4-2003	5:15:00	15,24
27-4-2003	5:30:00	22,86
27-4-2003	5:45:00	10,16
27-4-2003	6:00:00	12,7
	Total	113 mm



(b) Rain data & hydrograph for largest rainstorm in 2003.

Figure B.11: Rain data & discharge hydrograph

Date	time	mm/15min
1-6-2004	12:25:00	22,86
1-6-2004	12:40:00	33,02
1-6-2004	12:55:00	7,62
1-6-2004	14:10:00	2,54
	Total	67 mm



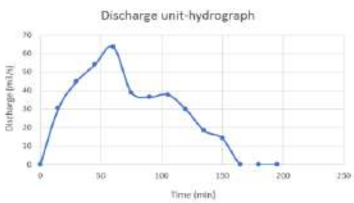
(a) Rain data & hydrograph for largest rainstorm in 2004.

Date	time	mm/15min
6-6-2005	9:25:00	20,32
6-6-2005	9:40:00	20,32
6-6-2005	9:55:00	2,54
6-6-2005	10:10:00	2,54
6-6-2005	10:25:00	22,86
6-6-2005	10:40:00	15,24
6-6-2005	10:55:00	2,54
	Total	86 mm

(b) Rain data & hydrograph for largest rainstorm in 2005.

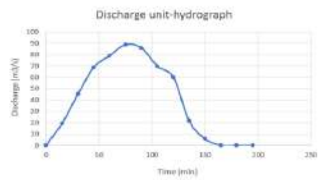
Figure B.12: Rain data & discharge hydrograph

Date	time	mm/15min
6-8-2006	14:00:00	25.4
6-8-2006	14:15:00	5.08
6-8-2006	14:30:00	7.62
6-8-2006	14:45:00	7.62
6-8-2006	15:00:00	7.62
6-8-2006	15:30:00	2.54
8-8-2006	16:00:00	10.16
	Total	66 mm



(a) Rain data & hydrograph for largest rainstorm in 2006.

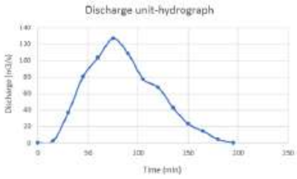
Date	time	mm/15min
13-6-2009	15:00:00	16
13-6-2009	15:15:00	17
13-6-2009	15:30:00	16
13-6-2009	15:45:00	6
13-6-2009	16:00:00	27
13-6-2009	16:15:00	11
13-6-2009	16:30:00	4
	Total	97 mm



(b) Rain data & hydrograph for largest rainstorm in 2009.

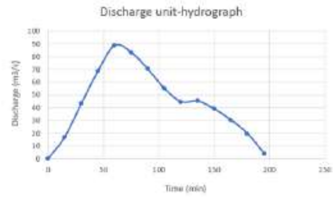
Figure B.13: Rain data & discharge hydrograph

Date	time	mm/15min
19-5-2010	16:30:00	2
19-5-2010	16:45:00	28
19-5-2010	17:00:00	29
19-5-2010	17:15:00	13
19-5-2010	17:30:00	21
19-5-2010	17:45:00	14
19-5-2010	18:00:00	6
19-5-2010	18:15:00	7
19-5-2010	18:30:00	3
	Total	123 mm



(a) Rain data & hydrograph for largest rainstorm in 2010.

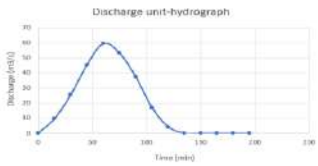
Date	time	mm/15min
28-5-2011	15:00:00	14
28-5-2011	15:15:00	18
28-5-2011	15:30:00	17
28-5-2011	15:45:00	14
28-5-2011	16:00:00	10
28-5-2011	16:15:00	9
28-5-2011	16:30:00	6
28-5-2011	16:45:00	7
28-5-2011	17:00:00	11
28-5-2011	17:15:00	3
	Total	106 mm



(b) Rain data & hydrograph for largest rainstorm in 2011.

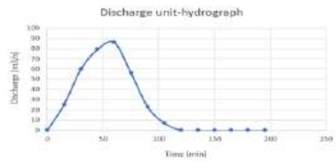
Figure B.14: Rain data & discharge hydrograph

Date	time	mm/15min
14-9-2013	1:55:00	8
14-9-2013	2:10:00	11
14-9-2013	2:25:00	14
14-9-2013	2:40:00	9
14-9-2013	2:55:00	3
	Total	45 mm



(a) Rain data & hydrograph for largest rainstorm in 2013.

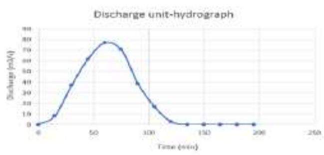
Date	time	mm/15min
14-11-2014	14:10:00	21
14-11-2014	14:25:00	23
14-11-2014	14:40:00	11
14-11-2014	14:55:00	5
	Total	62 mm



(b) Rain data & hydrograph for largest rainstorm in 2014.

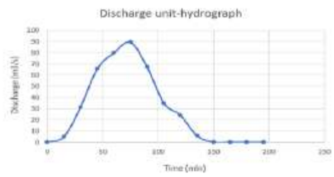
Figure B.15: Rain data & discharge hydrograph

Date	time	mm/15min
14-6-2015	18:35:00	7
14-6-2015	18:50:00	22
14-6-2015	19:05:00	15
14-6-2015	19:20:00	10
14-6-2015	19:35:00	2
	Total	56 mm



(a) Rain data & hydrograph for largest rainstorm in 2004.

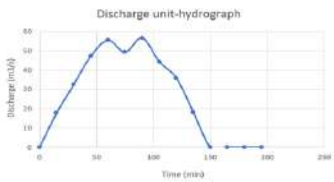
Date	time	mm/15min
6-6-2016	16:00:00	4
6-6-2016	16:15:00	21
6-6-2016	16:30:00	23
6-6-2016	16:45:00	7
6-6-2016	17:00:00	13
6-6-2016	17:15:00	4
	Total	72 mm



(b) Rain data & hydrograph for largest rainstorm in 2005.

Figure B.16: Rain data & discharge hydrograph

Date	time	mm/15min
2-8-2017	14:10:00	15
2-8-2017	14:25:00	8
2-8-2017	14:40:00	11
2-8-2017	14:55:00	5
2-8-2017	15:10:00	12
2-8-2017	15:30:00	13
	Total	64 mm



(a) Rain data & hydrograph for largest rainstorm in 2017.

Figure B.17: Rain data & discharge hydrograph

B.3 Water quality

During the research all collected samples were kept in plastic and glass containers, correctly identified and stored in ice-cold refrigerators until they were delivered to the laboratory accredited by the CNA of Panama. The conservation methods for each parameter (indicated in Executive Decree No. 75 of June 4, 2008, Technical Regulations DGNTI-COPANIT and last update of Standard Methods) have been followed.

To qualify the water and compare the results of the phsico-chemical variables and microbiological tests for all parameters a method is used from the Executive Decree No. 75 of June 4, 2008. It is indicated that, individually for each parameter, the most restrictive model has been used in the assessment and follows the principle "one outside, all outside". So if one parameter does not comply with its tolerable limit the whole water body does not comply with the corresponding quality status. In this method the following criteria have been followed, see Table B.2:

Criteria of degree of compliance of recreational water parameters	Class of water quality
Meets the legal limits of low risk (direct contact) of Executive Decree No. 75	Good
Meets the legal limits of medium risk (without direct contact) of Executive Decree No. 75	Moderate
Does not comply with the legal limits of low risk (without direct contact) of Executive Decree No. 75	Bad

Table B.2: Classification of water quality (Programa Sanaemiento de Panama, 2018).

In Executive Decree No. 75 tolerable limits for several of the water quality parameters are indicated for water with direct contact and without direct contact. These values can be found in Table B.3.

Parámetros	Unidad	BAJO RIESGO	RIESGO MEDIO
		Contacto directo	Sin contacto directo
Bacteriológico			
Coliformes fecales	UFC / 100 ML	=<250 Coliformes fecales/100 mL (o 200 estreptococo fecales /100mL) (5)	251 – 450 Coliformes fecales (o 201 – 500 estreptocos fecales /100mL) (5).
Fisicoquímicos			
pH(1)	unidad de pH	6.5-8.5	6.5-8.5
Temperatura	ΔT°C	3(4)	3(4)
Transparencia (disco Secchi)(2)	M	>1.2	0-1.2
Sólidos flotantes	-	Ausentes	Ausentes
Sólidos suspendidos	mg/L	<50	<50
Sólidos disueltos	mg/L	<500	<500
Color	Pt-Co	<100	100-150
Turbiedad	NTU	<50	50-100
Oxígeno disuelto(2)	mg/L	>7	6-7
Demanda bioquímica de oxígeno (DBO_5)	mg/L	<3	3-5
Orgánicos			
Grasas y aceites	mg/L	<10	<10
Parámetros	Unidad	BAJO RIESGO	RIESGO MEDIO
		Contacto directo	Sin contacto directo
Hidrocarburos	mg/L	<0.05	0.05-0.2
Hidrocarburos aromáticos policíclicos	mg/L	<0.2	0.2-1
Plaguicidas(cada uno)	mg/L	Ausente	<0.005
Detergentes (SAAM)(3)	mg/L	<1.0	<1.0
Inorgánicos y metales			
Cianuro	mg/L	<0.01	<0.01
Arsénico	mg/L	<0.1	<0.1
Cadmio	mg/L	<0.03	<0.03
Cromo(VI)	mg/L	<0.05	<0.05
Mercurio	mg/L	<0.01	<0.01
Plomo	mg/L	<0.05	0.05-0.2

Table B.3: Water quality standards from Executive Decree No. 75 (Decreto Ejecutivo No. 75, 2008).

B.4 Bathymetry bay of Panama

The bathymetry of the entire bay of Panama City, see Figure B.18 was retrieved from a survey done by Boskalis in previous works. The complete bathymetry map was loaded into Delft3D for modeling purposes.

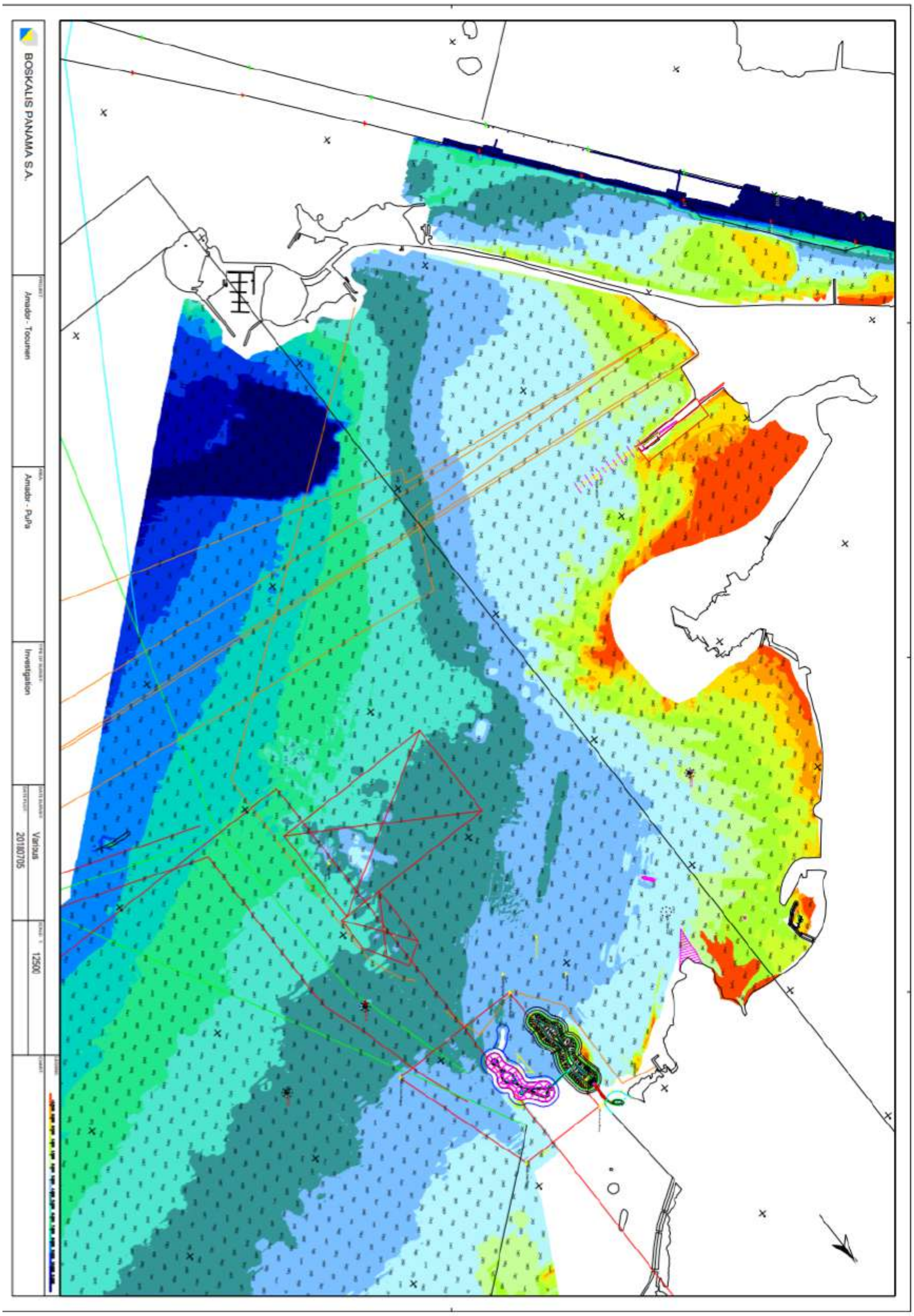


Figure B.18: Bathymetry in the bay of Panama City [Boskalis, 2018].

B.5 Soil conditions

Information on soil conditions was retrieved from earlier works done ICA Panama for a land reclamation project in 1998. Now, two islands are located at the site of this drilling. The soil conditions are expected to be similar to those at the project location.

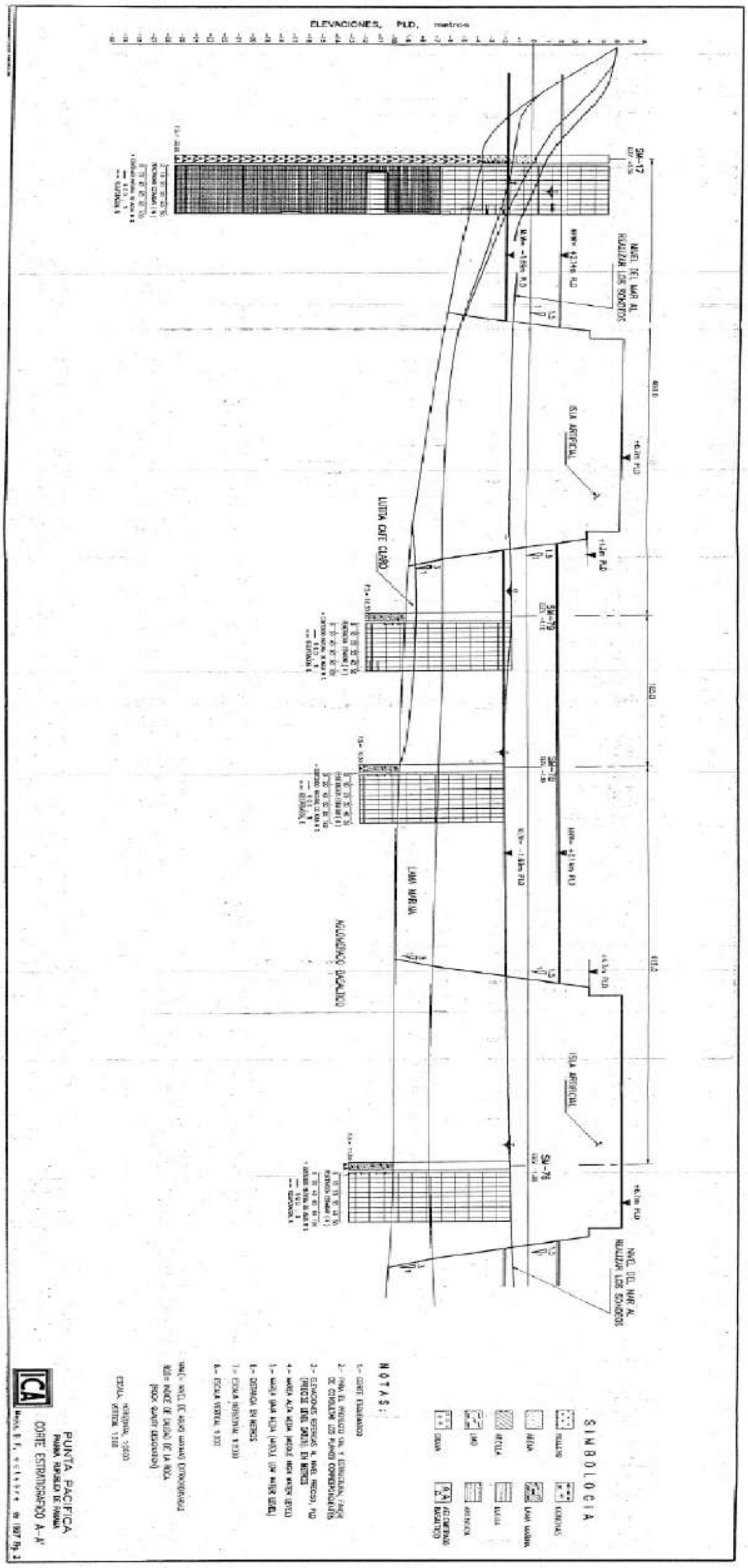


Figure B.19: Soil conditions at land reclamation Punta Pacifica [ICA Panama, 1998].

Appendix C

Delft3D

This Appendix describes the set up of the Delft3D flow model with a complete overview of all the results from the Water Quality model.

The Delft3d model consists of several modules. The method and actions that were performed inside each module will be discussed here.

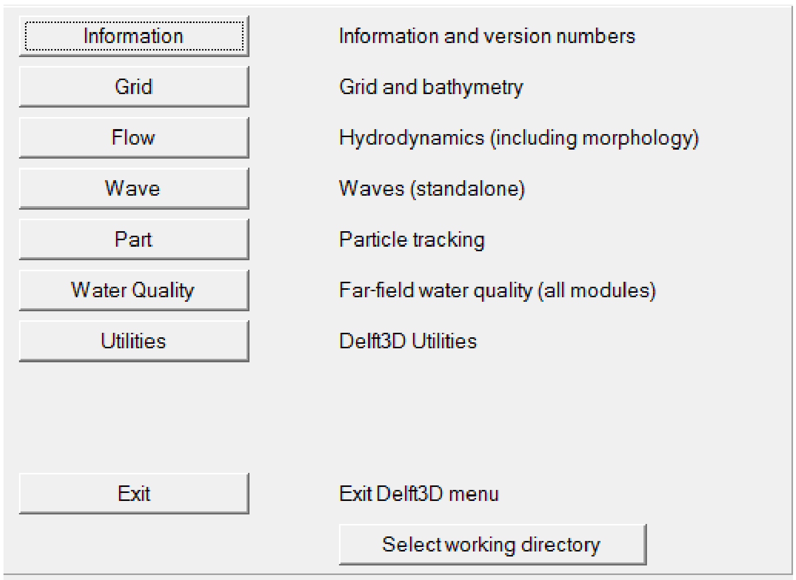


Figure C.1: Modules in Delft3D

C.1 Flow model

Before the water quality can be modeled, first a flow model needs to be made that simulates the flow and currents inside the bay of Panama. To get an accurate model the processes at large and small scale need to be included. Therefore a large, coarse model and a more detailed model at Panama City is made.

C.1.1 Coarse model

The first step is to make a large coarse grid that covers the entire bay of Panama, see Figure C.2. This is done to get a better look on the processes at larger scale before looking more closely at the conditions near Panama City. The grid, depth and boundary conditions are made using the Delft Dashboard software. This software contains coarse bathymetry data and tidal information. The spatial setup of the grid size is 320 cells in horizontal direction and 260 in vertical direction, such that each is more or less 1x1 km in size. For this grid the bathymetry or depth file is made using the software. The final step is to assign the boundary conditions. Here the tidal conditions are given to the south border of the model, as can be seen as the blue line on the bottom of the grid in Figure C.2.

The results of this first, coarse, model are only intended to have a look at the currents an behaviour of the flows in the bay of Panama. This information will also be needed for the boundary conditions of the more detailed model. In figure C.3 the results can be seen for outgoing tide C.3a and incoming tide C.3c. Figure C.3b and C.3d show the transition between incoming and outgoing tides. As can be seen from the results the currents inside the bay of Panama are in an upwards or downwards orientation for most of the time. Only at the transition between incoming and outgoing tide the flow patterns change notably. However, it can also be seen that a flow along the coast is present at the location of Panama City (indicated with a red dot). So the tide

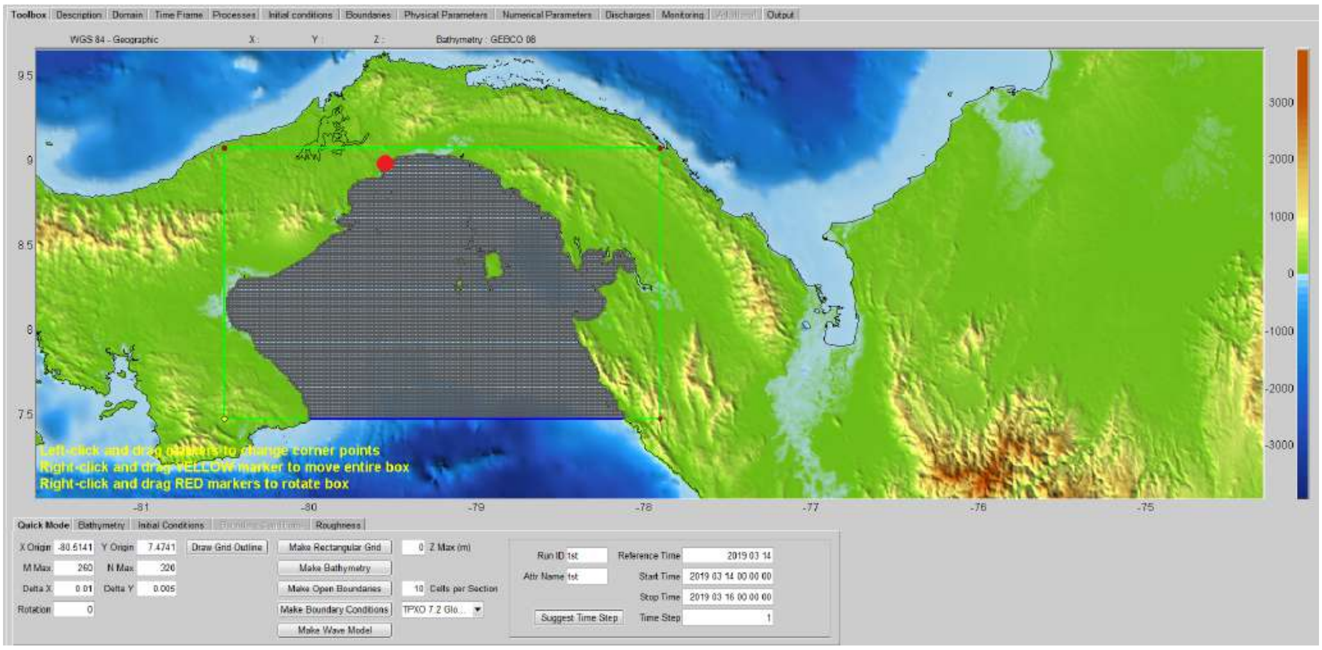
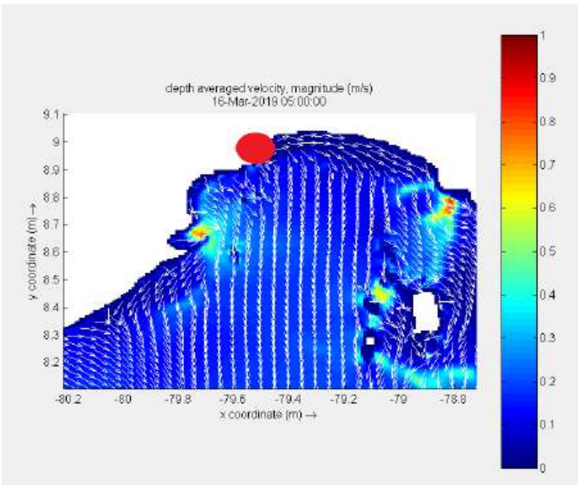
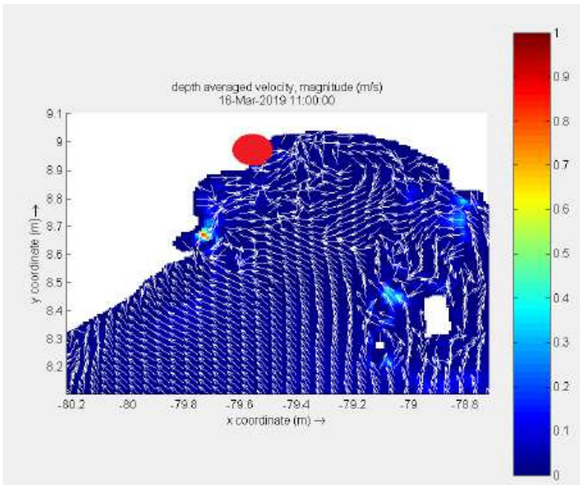


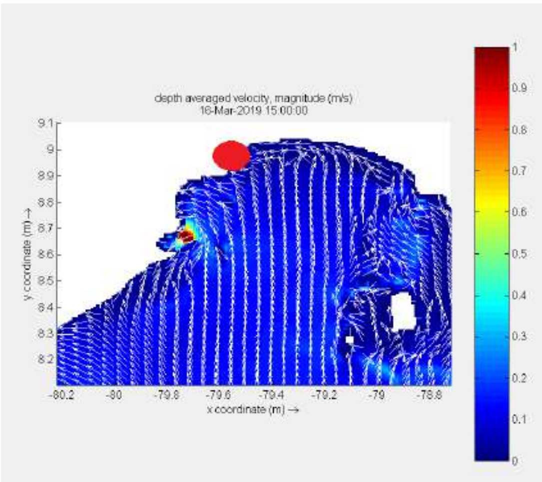
Figure C.2: Coarse grid



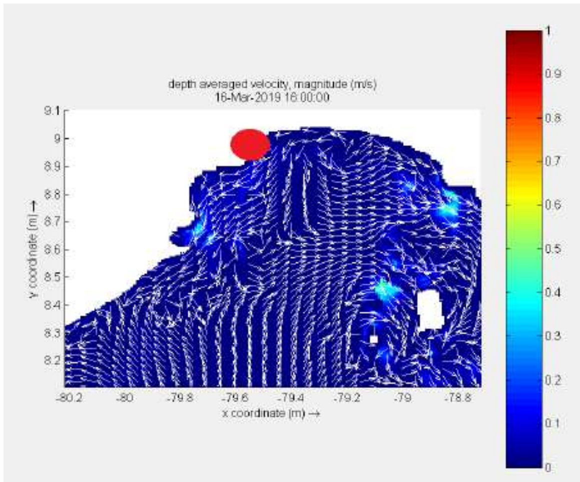
(a) Coarse grid model, outgoing (low) tide



(b) Coarse grid model, intermediate tide



(c) Coarse grid model, incoming (high) tide



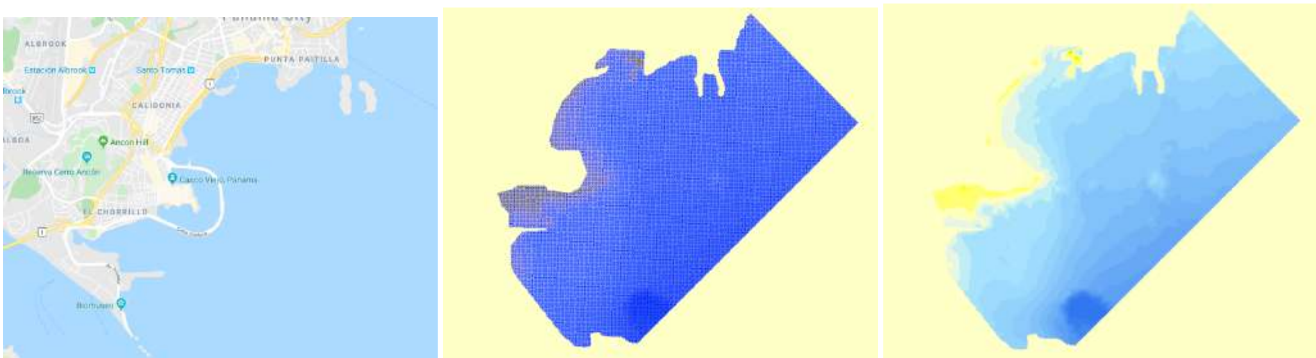
(d) Coarse grid model, intermediate tide

Figure C.3: Flow patterns inside bay of Panama for incoming and outgoing tide

will also cause a horizontal flow, though the depth average velocity is generally low around 0.1 tot 0.2 m/s.

C.1.2 Fine detailed model

With a coarse grid of 1 km by 1 km it is very hard to look at local conditions. Therefore, to get a good look on the conditions at the location of interest, a grid with detailed information is made. This is done using the detailed bathymetry data of the area. The area at interest is shown if Figure C.4a. The bathymetry data is loaded into the Delft3D module QUICKIN. Next a fine grid is constructed that fits neatly over the bathymetry data, see Figure C.4b. In this case the grid cells are 5x5 meters. When the grid is fitted to the data it can create a depth file. This means that the depth for each grid cell is determined using triangular interpolation. The final result is shown in Figure C.4c. Using this depth file simulations can be made for the detailed model.



(a) (b) (c)

Figure C.4: Fine model

C.1.3 Nesting

The detailed model has to be nested in the large coarse model before simulations can be done. This is to get the right boundary conditions on the detailed models boundaries. These boundary conditions are simulated in the large coarse model and will serve as the starting conditions for the detailed model. The nesting is done using the nesting tool (Figure C.5) in Delft3D and consists of two phases: nesting 1 and nesting 2.

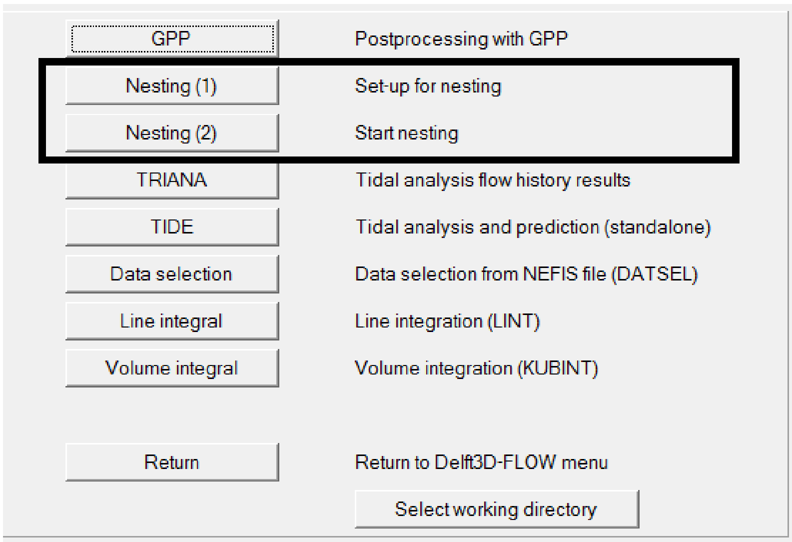


Figure C.5: Nesting tool in Delft3D

Nesting 1: In the nesting 1 phase the detailed model is fitted into the large coarse model. On the edges of the detailed model boundaries are assigned.

Nesting 2: Here the results of the large coarse model are assigned at the boundaries of the detailed model. In later simulation these will act as the boundary conditions.

C.1.4 Detailed flow model input parameters

- Domain:

- Time frame:
- Initial conditions:
- Boundaries:
- Monitoring:
- Output:

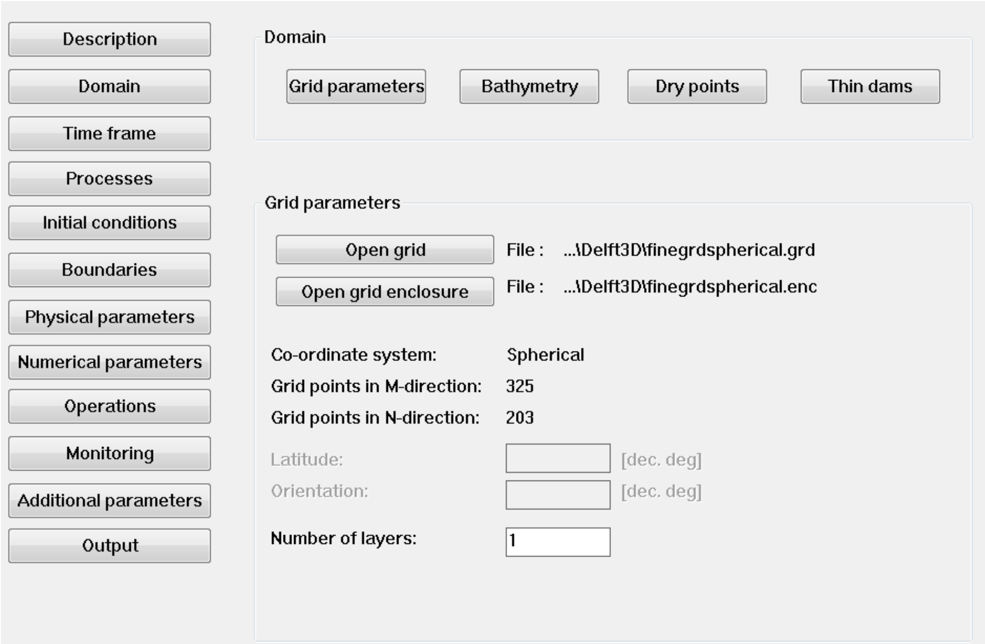
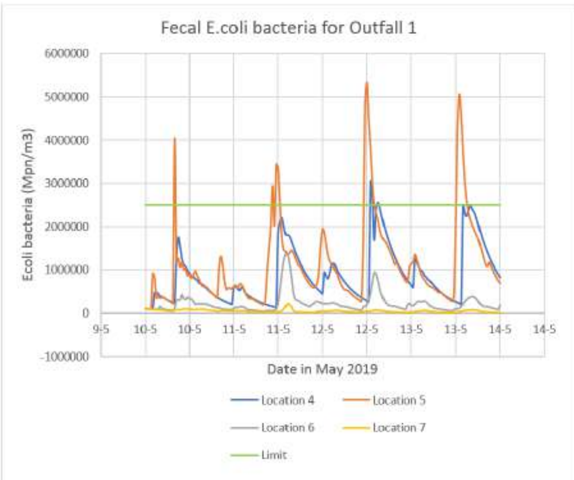


Figure C.6: Input parameters for flow model in Delft3D

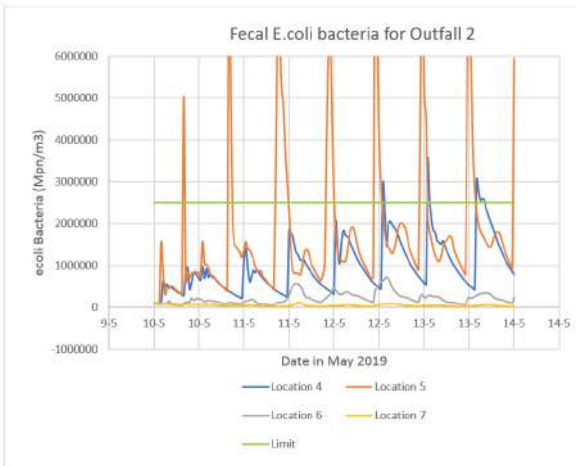
C.2 D-Water Quality model

The results for all the modeled outfall locations can be seen here. For each substance the results of the 4 outfall locations are shown.

E.coli bacteria
Here the results for E.coli bacteria are presented.

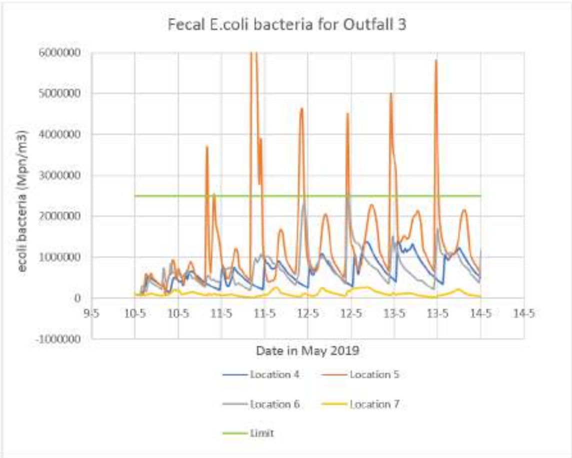


(a) Fecal E.coli bacteria outfall 1

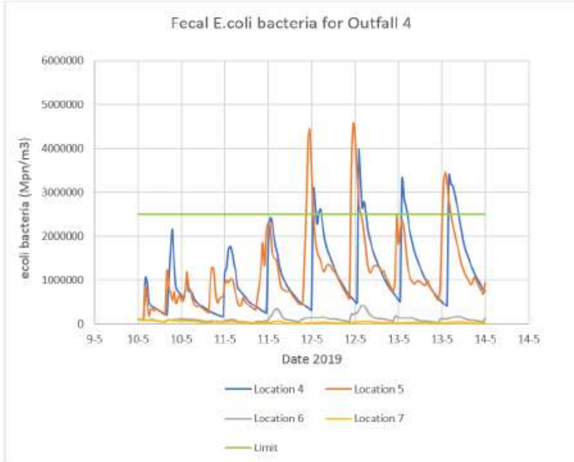


(b) Fecal E.coli bacteria outfall 2

Figure C.7: Results for fecal E.coli bacteria outfall 1 and 2



(**a**) Fecal E.coli bacteria outfall 3

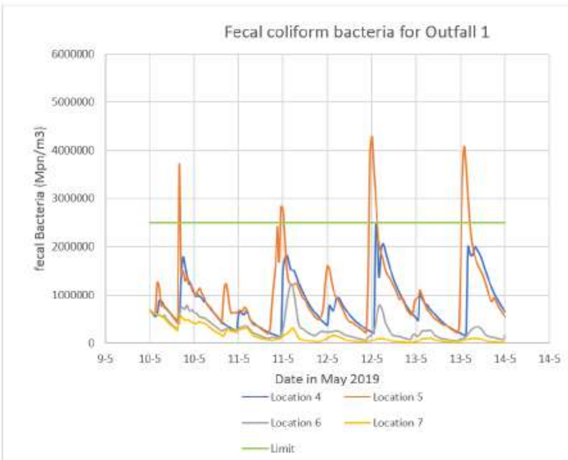


(**b**) Fecal E.coli bacteria outfall 4

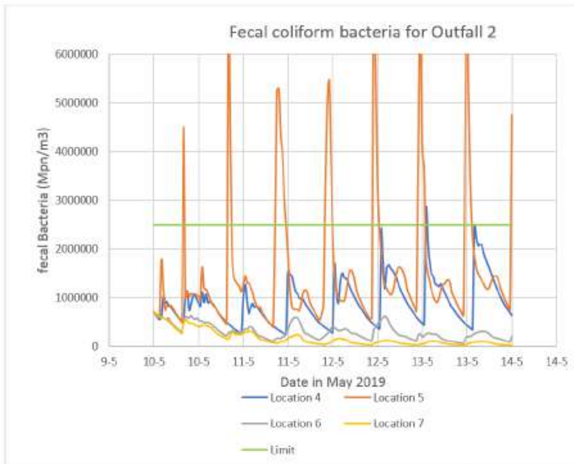
Figure C.8: Results for fecal E.coli bacteria outfall 3 and 4

Fecal coliform bacteria

Here the results for Fecal coliform bacteria are presented.

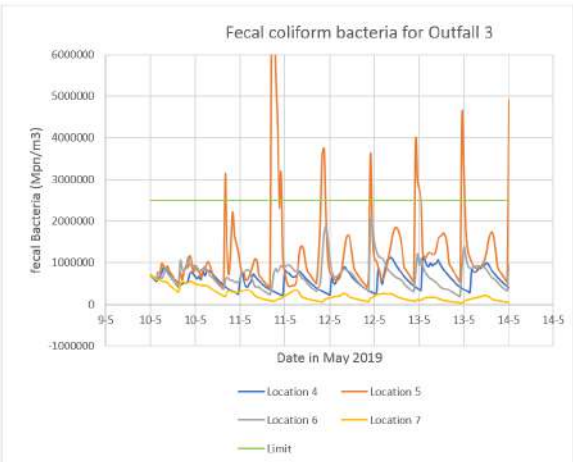


(**a**) Fecal coliform bacteria outfall 1

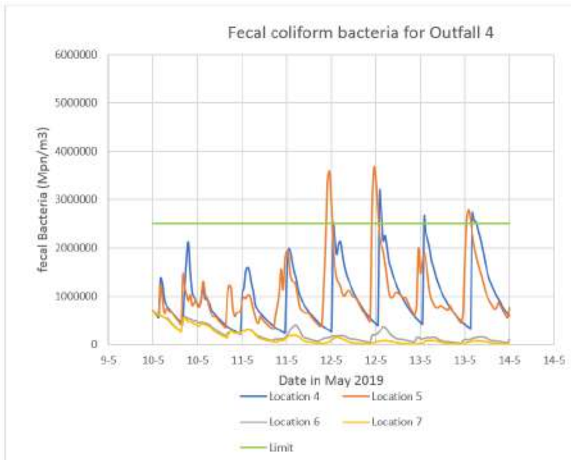


(**b**) Fecal coliform bacteria outfall 2

Figure C.9: Results for fecal coliform bacteria outfall 1 and 2



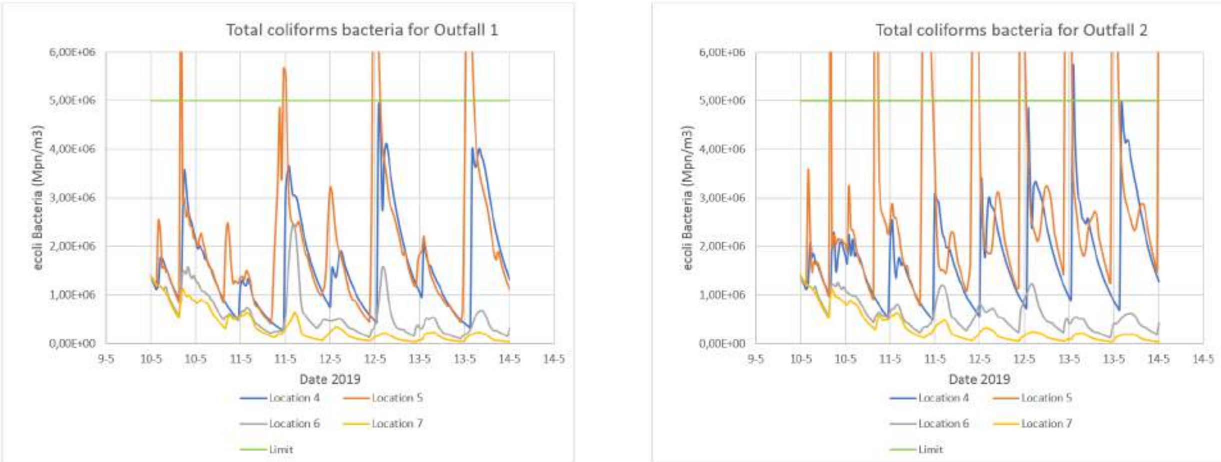
(**a**) Fecal coliform bacteria outfall 3



(**b**) Fecal coliform bacteria outfall 4

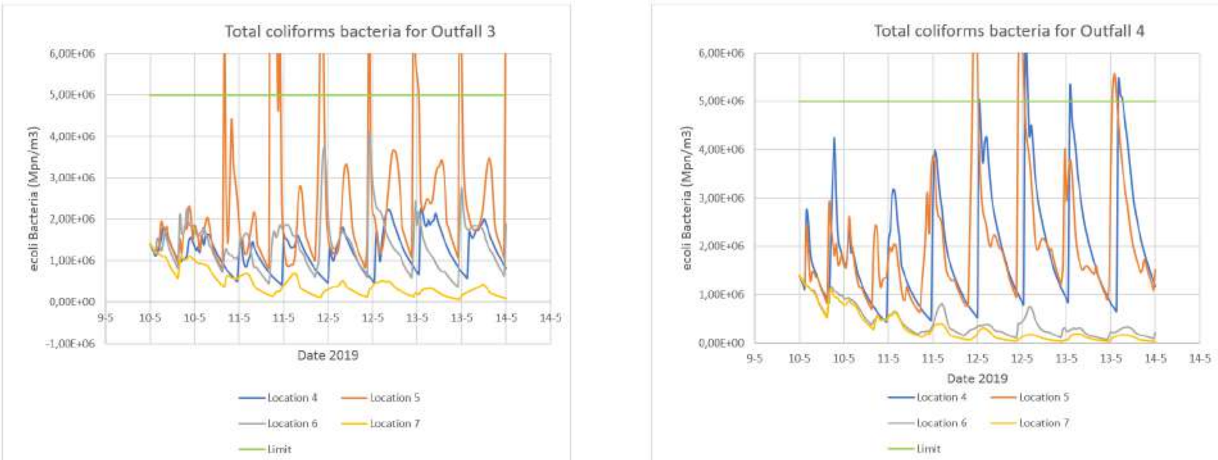
Figure C.10: Results for fecal coliform bacteria outfall 3 and 4

Total Fecal coliforms bacteria
Here the results for total Fecal coliform bacteria are presented.



(a) Total Fecal coliform bacteria outfall 1 **(b)** Total Fecal coliform bacteria outfall 2

Figure C.11: Results for total fecal coliforms bacteria outfall 3 and 4



(a) Total Fecal coliform bacteria outfall 3 **(b)** Total Fecal coliform bacteria outfall 4

Figure C.12: Results for total fecal coliforms bacteria outfall 3 and 4

Appendix D

Calculations

This Appendix gives an elaboration on the used formulas and calculations which are used in the report.

D.1 Water level inside the river under dry conditions

The water level inside the river follows the tide at sea. In case an outfall pipeline is in place, the discharge area should be sufficient in order to be able to discharge all the water. This depends on the water level inside the river and the amount of water and can be shown in terms of the following formula.

First, the length over which water is inside the river is expressed in terms of the water height inside the river divided by the slope.

$$l = \frac{h}{i} \tag{D.1}$$

Secondly, the cross section area is determined based on the dimensions of the river. This is as follows:

$$A = 15 * h + 0.6 * h \tag{D.2}$$

Now Equations D.1 and D.2 can be combined in order to express the total volume of water inside the river in terms of the water level h.

$$V = \frac{1}{2} * A * l = \frac{1}{2} * (15 * h + 0.6 * h) * \frac{h}{0.0022} = 3545 * h^2 \tag{D.3}$$

The volume of water V_i inside the river at a certain time step i depends on the volume of water at the time step previous to the one under consideration V_{i-1} plus the incoming water $V_{in\Delta t}$ from upstream minus the amount of water that is discharged through the pipeline $V_{out\Delta t}$. This leads to the following equation:

$$V_i = V_{i-1} + V_{in\Delta t} - V_{out\Delta t} \tag{D.4}$$

The volume of water that is discharged through the outfall pipelines depends on the cross sectional area of the pipeline and the speed of the water. This flow speed can be determined by looking at the water levels and using the hydraulic head balance. For calculations, a time step of 15 minutes or 900 seconds was used.

$$V_{out\Delta t} = v_2 * A_{pipe} * 900 \tag{D.5}$$

The incoming amount of water depends on the rivers natural discharge under normal conditions. Again, a time step of 15 minutes or 900 seconds was used.

$$V_{in\Delta t} = Q_{river} * 900 \tag{D.6}$$

The water height in the river can finally be described by rewriting formula D.3.

$$h_i = \sqrt{\frac{V_i}{3545}} + 1.7 \tag{D.7}$$

D.2 Bending moment reinforcement calculation

The required reinforcement steel needed in the slabs depends on the generated bending moments as a result of the loads acting on the structure. The reinforcement steel area which is needed, was determined by using a moment balance equation inside the slab around the point where the concrete compression force acts in the slab. A presentation of this is given in the Figure and Equations below.

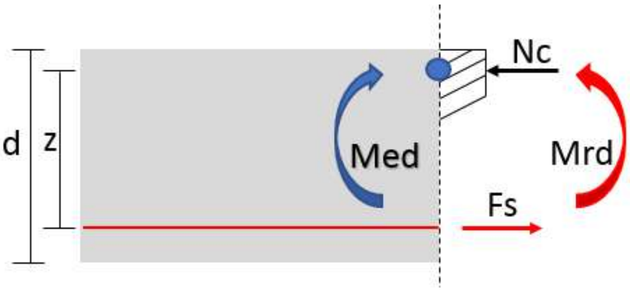


Figure D.1: Forces acting on cross section in the slab

$$\Sigma M_{bluedot} = -M_{ed} + F_s * z = 0 \quad \rightarrow \quad \Sigma M_{bluedot} = -M_{ed} + A_s * f_{yd} * z = 0 \tag{D.8}$$

$$A_s = \frac{M_{ed}}{f_{yd} * z} \tag{D.9}$$

$$A_s = \frac{1}{4} * \phi^2 * \pi * \frac{1000}{s} \quad \rightarrow \quad \phi = \sqrt{\frac{A_s * 4}{\pi * \frac{1000}{s}}} \tag{D.10}$$

In which:

M_{ed}	=	Design bending moment (Nmm)
F_s	=	Force in steel rebar, $F_s = A_s * f_{yd}$ (N)
z	=	Inernal lever arm 0.9*d (270 mm)
A_s	=	Steel rebar area (mm^2)
f_{yd}	=	Reinfocement steel yield strength (435 N/mm^2)
s	=	Spacing of rebar over widht (200 mm)
ϕ	=	Diameter of steel rebar (mm)

From formula D.9 it is clear that the design bending moment is needed in order to calculate the required steel area. To do so, the loads acting on the structure were calculated first for both the horizontal and vertical slab. A representation of the situation is given in Figure D.2.

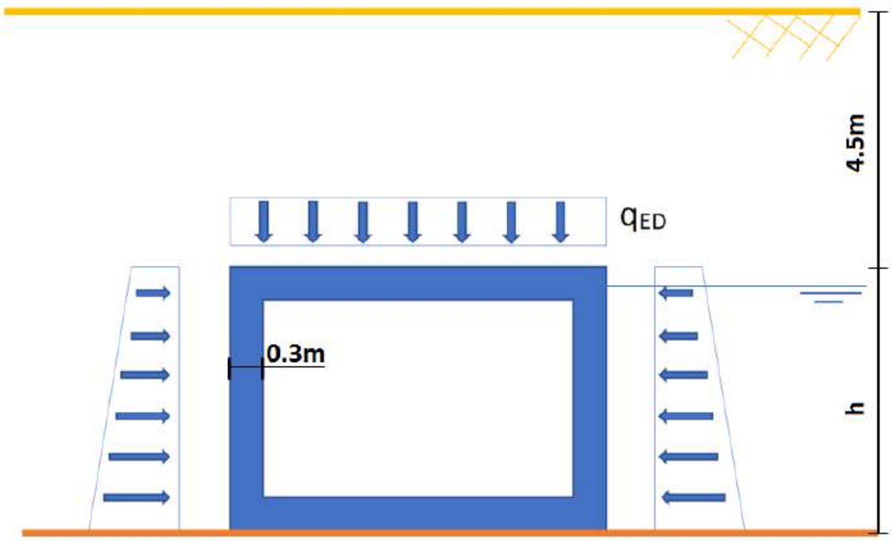


Figure D.2: Forces acting on the structure

D.2.1 Loads

Horizontal slab

The design load acting on the horizontal slab is defined by the weight of the soil plus a variable load as in:

$$q_{ed} = \gamma_G * q_{Gk} + \gamma_Q * q_{Qk} \tag{D.11}$$

In which:

q_{ed} = Design load on structure (kN/m^2)
γ_G = Permanent load factor (1.35)
q_{Gk} = Dead weight load (kN/m^2)
γ_Q = Variable load factor (1.5)
q_{Qk} = Variable load (kN/m^2)

To find the design load the dead weight load needs to be defined first. This was done using the following formula:

$$q_{Gk} = \gamma_s * d + \gamma_c * t \tag{D.12}$$

In which:

γ_s = Volumetric soil weight (kN/m^3)
d = Thickness of soil layer (m)
γ_s = Volumetric concrete weight (kN/m^3)
d = Thickness of concrete slab layer (m)

For these calculations the following parameters were used:

Wall thickness t (m)	Concrete weight γ_c (kN/m^3)	Soil weight γ_s (kN/m^3)	Sand layer d (m)	Variable load (kN/m^2)	Permanent load factor γ_G	Variable load factor γ_Q
0.3	25	20	4.5	5	1.35	1.5

Table D.1: Input parameters to compute the design load for horizontal slab.

Now, the dead weight load acting on the horizontal slab could be calculated using Formula D.12. The result is as follows:

$$q_{Gk} = 20 * 4.5 + 25 * 0.3 = 97.5 kN/m^2 \tag{D.13}$$

Using this result the final design load q_{ed} could be determined using Formula D.11. The variable load was estimated to be 5 kN/m^2. This gave the following result:

$$q_{ed} = 1.35 * 97.5 + 1.5 * 5 = 139.1 kN/m \tag{D.14}$$

Vertical slab

The design load acting on the vertical slab was determined differently from the horizontal slab. This is because the soil acts differently in a horizontal way. In this situation the load is defined by adding the hydrostatic water pressure to the active soil pressure.

$$q_{ed} = \gamma_w * d + K_a * \gamma_s * d \tag{D.15}$$

In which:

$$K_a = \text{Active soil coefficient (0.33)}$$
$$\gamma_d = \text{Volumetric weight of soil (kN/m}^3\text{)}$$
$$\gamma_w = \text{Volumetric weight of water (kN/m}^3\text{)}$$
$$d = \text{Depth of the soil layer acting on structure (m)}$$

Since the water level varies with the tide, it was decided to look for the least favorable situation which occurs when the water level is highest and equal to the beach level at 4.5 meters above the structure. The load will increase linearly over its height and will therefore be largest at the base of the structure. The calculations were done for the top and base of the structure for both design variants leading to the following:

Top structure of 60 and 150 m^3/s design

$$q_{ed,top} = 10 * 4.5 + 0.33 * 20 * 4.5 = 75\ kN/m \tag{D.16}$$

Bottom structure 60 m^3/s design

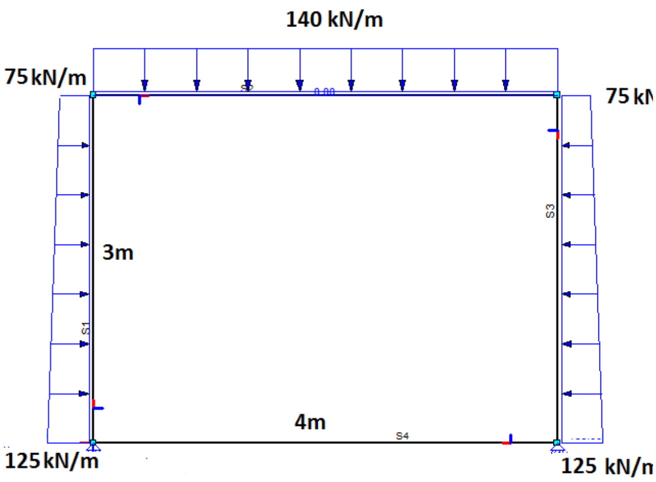
$$q_{ed,top} = 10 * 7.5 + 0.33 * 20 * 7.5 = 125\ kN/m \tag{D.17}$$

Bottom structure 150 m^3/s design

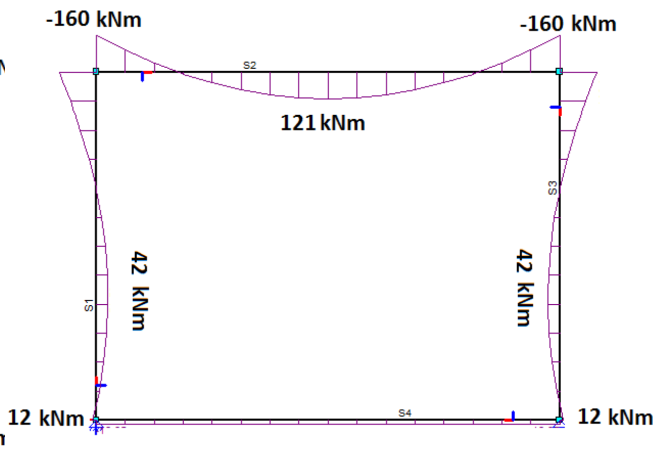
$$q_{ed,top} = 10 * 8.5 + 0.33 * 20 * 8.5 = 142\ kN/m \tag{D.18}$$

D.2.2 Bending moments

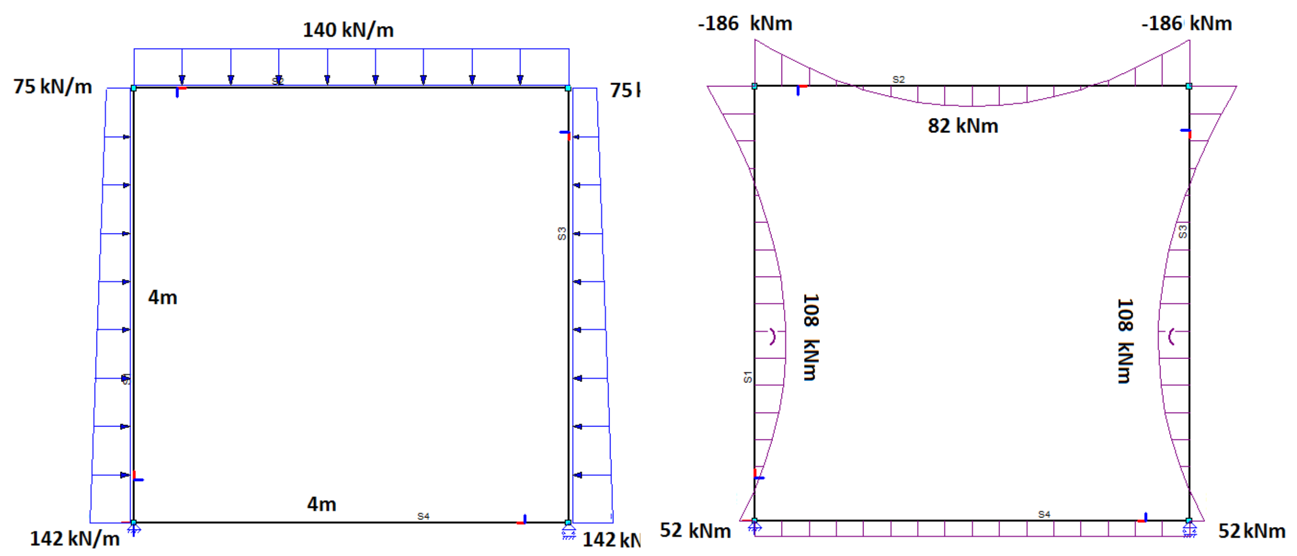
The bending moments were found by implementing the loads on the structure in MatrixFrame. The results, together with the loads acting on the structures can be seen in the figures below.



(a) Design load acting on structure for 60 m^3/s design variant



(b) Resulting design moments for 60 m^3/s design variant



(a) Design load acting on structure for 150 m^3/s design variant **(b)** Resulting design moments for 150 m^3/s design variant

Figure D.4: Forces acting on the structure for 60 and 150 m^3/s design variants

Now that the bending moments in the slabs are known, the required reinforcement steel can be calculated according to:

$$A_s = \frac{M_{ed}}{f_{yd} * z} \tag{D.19}$$

- For this design reinforcement steel of class B500B is used, which has a yield strength f_{yd} of 435 N/mm^2.
- The internal lever arm in the slab can be found with 0.9*d and is equal to 270 mm for a 300 mm thick slab.

The required reinforcement steel could now be calculated, here an example is given for the horizontal slab of the 150 m^3/s design.

$$A_s = \frac{186 * 10^6}{435 * 270} = 1538\ mm^2 \tag{D.20}$$

The diameter corresponding to this area was found using:

$$A_s = \frac{1}{4} * \phi^2 * \pi * \frac{1000}{s} \quad \rightarrow \quad \phi = \sqrt{\frac{A_s * 4}{\pi * \frac{1000}{s}}} \tag{D.21}$$

The loads acting on the structure were determined per running meter. Therefore, the spacing s is defined per 1000 mm. A spacing of 200 mm was chosen for the design. This leads to a diameter of:

$$\phi = \sqrt{\frac{1538 * 4}{\pi * \frac{1000}{200}}} = 20\ mm \tag{D.22}$$

The results for all the slabs for both design variants can be found in the tables D.2 and D.3 on the next page.

60 m³/s	Max. design moment M_{ED} (kNm)	Internal lever arm z (mm)	Spacing s (mm)	Required steel area A_s (mm²)	Steel bar diameter ϕ (mm)
Horizontal slab	160	270	200	1362	19
Vertical slab	42	270	200	360	10

Table D.2: Input parameters and resulting rebar dimensions for the 60 m^3/s design

150 m³/s	Max. design moment M_{ED} (kNm)	Internal lever arm z (mm)	Spacing s (mm)	Required steel area A_s (mm²)	Steel bar diameter ϕ (mm)
Horizontal slab	186	270	200	1538	20
Vertical slab	108	270	200	919	16

Table D.3: Input parameters and resulting rebar dimensions for the 150 m^3/s design

D.3 Shear force calculation

The shear force in the structure is a result of lifting the culvert sections. The forces, as shown in Figure D.5, are generated by the self weight of the sections and are subjected at the location where the straps are attached to the structure. Here, the calculations for the shear force of the 150 m^3/s design are elaborated.

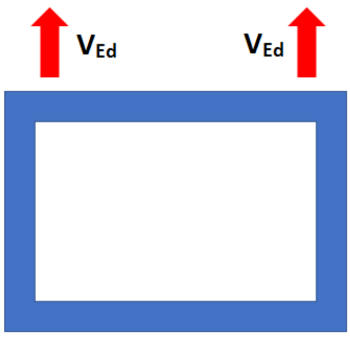


Figure D.5: Forces acting on culvert section during lifting

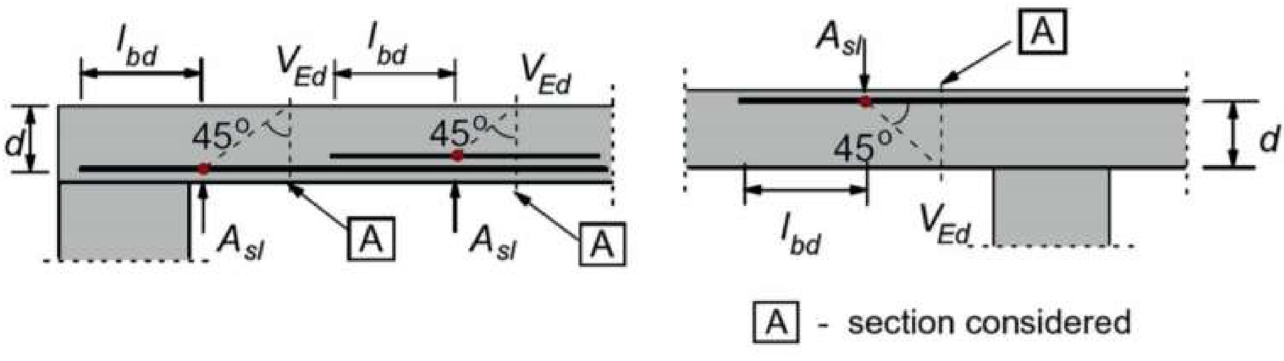


Figure D.6: Principle of shear force without shear reinforcement(Manual Hydraulic Structures, 2019)

The following Formula is applied to check whether the structure can withstand the forces without applying extra shear reinforcement (Voorendt, 2019).

$$V_{Rd,c} = [C_{Rd,c} * k * (100 * \rho_1 * f_{ck})^{1/3} + k_1 * \sigma_{cp}] * b_w * d \tag{D.23}$$

With a minimum of:

$$V_{Rd,min} = (v_{min} + k_1 * \sigma_{cp}) * b_w * d) \tag{D.24}$$

In which:

$$f_{ck} = \text{characteristic compressive strength of concrete (N/mm}^2\text{)}$$
$$k = 1+\sqrt{\frac{200}{d}} \leq 2.0\ (-)$$
$$\rho_1 = \text{reinforcement ratio in longitudinal reinforcement } \frac{A_s l}{b_w * d} \leq 0.02\ (-)$$
$$b_w = \text{smallest width of cross-section in the tensile area (mm)}$$
$$\sigma_{cp} = \text{compressive stress in the croncrete } \frac{N_{ed}}{A_c} \leq 0.2 * f_{cd}\ \text{(N/mm}^2\text{)}$$
$$N_{ed} = \text{Axial force in the cross section (N)}$$
$$A_c = \text{Effective area of the concrete cross-section 0.9*d (mm}^2\text{)}$$
$$k_1 = \text{Coefficient: } 0.15\ (-)$$
$$C_{Rd,c} = \text{Coefficient: } 0.18/\ \gamma_c = 0.12\ (-)$$
$$v_{min} = 0.035 * k^{3/2} * f_{ck}^{1/2}\ \text{(N/mm}^2\text{)}$$

Calculating all parameters

- The characteristic compressive strength f_{ck} for concrete class C40/50 is equal to 45 MPa.
- The factor k is defined by $1+\sqrt{\frac{200}{d}} \leq 2.0$, where d is the thickness of the slab which is 300 mm in this case. This results in a value of$1+\sqrt{\frac{200}{300}} = 1.86$ for this slab thickness and is lower than the maximum value of 2.
- The reinforcement ratio follows from the reinforcement area with respect to the concrete area: $\frac{A_s l}{b_w * d} \leq 0.02$. In this case a width in the tensile area b_w of 1000 mm was considered. The amount of reinforcement steel A_{sl} with a diameter of 25 mm and a spacing of 200 mm in that width is $A_s l = \frac{1}{4} * 25^2 * \pi * \frac{1000}{200} = 2455 mm^2$. The reinforcement ratio ρ_1 then becomes $\frac{2455}{1000*300} = 0.01$.
- The compressive stress σ_{cp} as a result of the axial force in the concrete is found by dividing the axial force N_{ed} by the effective concrete area $A_c = 0.9 * 300 * 1000 = 2.7 * 10^5 mm^2$ and should be smaller or equal than $0.2 * f_{cd}$ in which f_{cd} is the design compressive strength and is equal to 26.67 N/mm^2 for concrete class C40/50. The compressive stress becomes: $\frac{320000}{270000} = 1.19 N/mm^2$. This is smaller than the required $0.2 * 26.67 = 5.3$, so its ok.
- The load V_{ed} follows from the self weight of the structure. With a total weight of 25.8 tonnes and two lifting locations (in each top corner) the load will be 129 kN for the 150 m^3/s design variant.
- Finally the coefficients k_1 and $C_{Rd,c}$ are given to be 0.15 and 0.12 respectively.

Now all the variables can be implemented into Formula D.8 and D.9 to determine the design value for the shear resistance, which gives the following result:

$$V_{Rd,c} = [0.12 * 1.86 * (100 * 0.01 * 45)^{1/3} + 0.15 * 1.19] * 1000 * 0.9 * 300 = 263 kN \tag{D.25}$$

With a minimum of:

$$V_{Rd,min} = (0.035 * 1.86^{3/2} * 45^{1/2} + 0.15 * 1.19) * 1000 * 0.9 * 300 = 195 kN \tag{D.26}$$

The lowest value should be used which is the case for $V_{Rd,min}$ and is equal to 195 kN.

Unity check
Finally, the unity check was made to check whether the structure is safe. In case the unity check is smaller than 1 the structure is safe and no additional shear reinforcement is needed. The following check was made: $\frac{V_{ed}}{V_{Rd,min}} = \frac{129}{195} = 0.66$. This means that no additional shear reinforcement is needed in the slab.

Appendix E

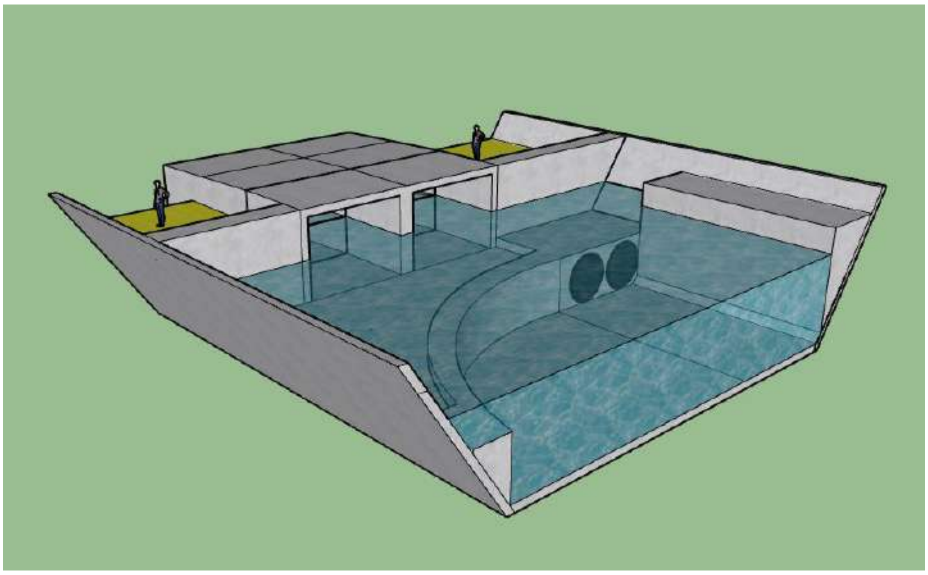
Design

This Appendix shows some additional design pictures of the 60 and 150 m^3/s design variant at high water events along with technical drawings of the designs. In the end the process for placing the outfall pipelines is shown.

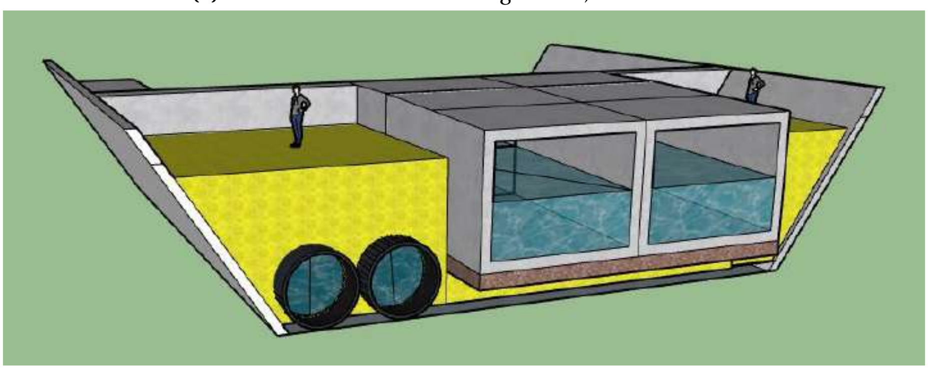
E.1 Design drawings

60 m^3/s

In this situation the inlet can be built inside the current river embankments without making any mayor changes. The two outfall pipes with inner diameter of 2 meters are located on the Western bank of the river. This height is also used as wall height that will catch the polluted water in the river. The water will only exceed this wall if either the high water level at sea is higher than CD +3.7 m or if the water level on the river side exceeds 2 meters due to heavy rainfall. In this case the water will flow through the culvert. Under the culvert a gravel layer of 25 cm is placed.



(a) 60 m^3/s inlet structure at high water, river side view

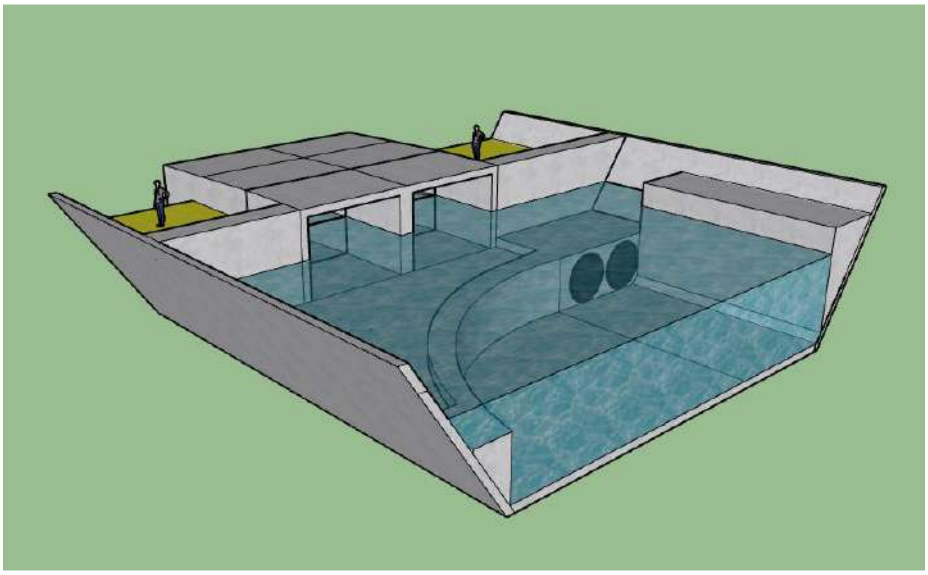


(b) 60 m^3/s inlet structure at high water, beach side view

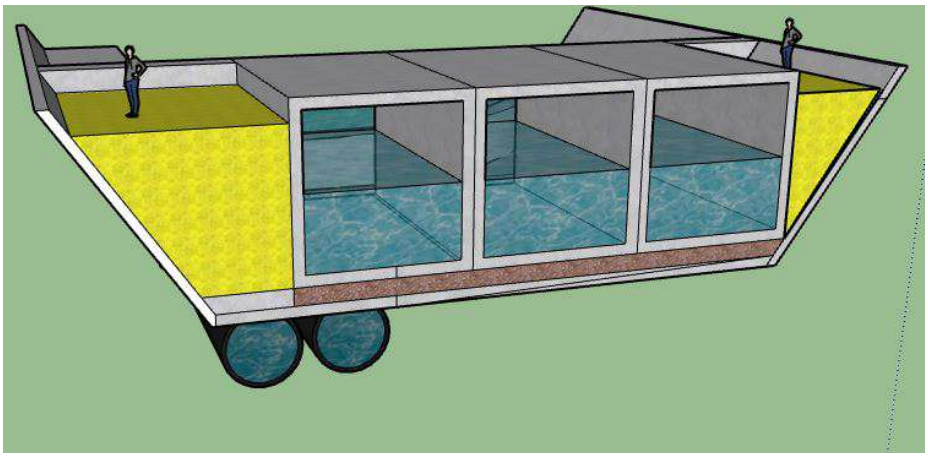
Figure E.1: Culvert inlet for 60 m^3/s design capacity at river mouth during large discharge

150 m^3/s

For this design some changes will have to be made to the existing river banks, because in case the three culvert sections are placed next to each other there will not be enough room for the outfall pipes. As a solution the outfall pipes will be placed 2 meters lower than the current bed level, this can clearly be seen in Figure 5.43a. For water levels the same conditions apply as for the 60 m^3/s design capacity, because the water retaining wall is still positioned at the same level. The gravel layer will also be 25 cm.



(a) 150 m^3/s inlet structure at high water, river side view



(b) 150 m^3/s inlet structure at high water, beach side view

Figure E.2: Culvert inlet for 60 m^3/s design capacity at river mouth during large discharge

Technical drawings

On the next two pages technical drawing made in AutoCad can be seen for the 60 and 150 m^3/s design variants. Some of the most important dimensions are given in the drawings.

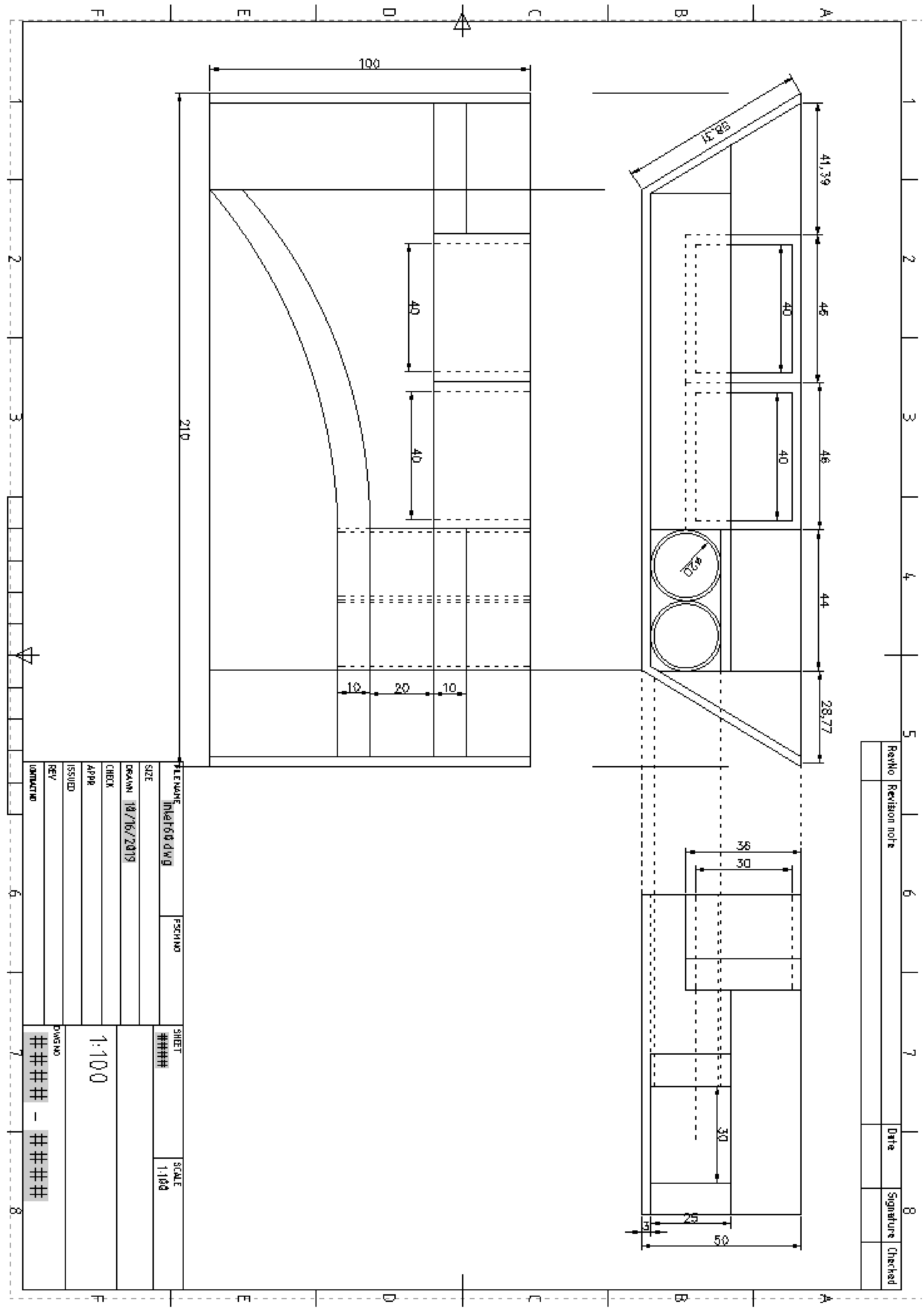


Figure E.3: Technical drawing 60 m^3/s inlet structure

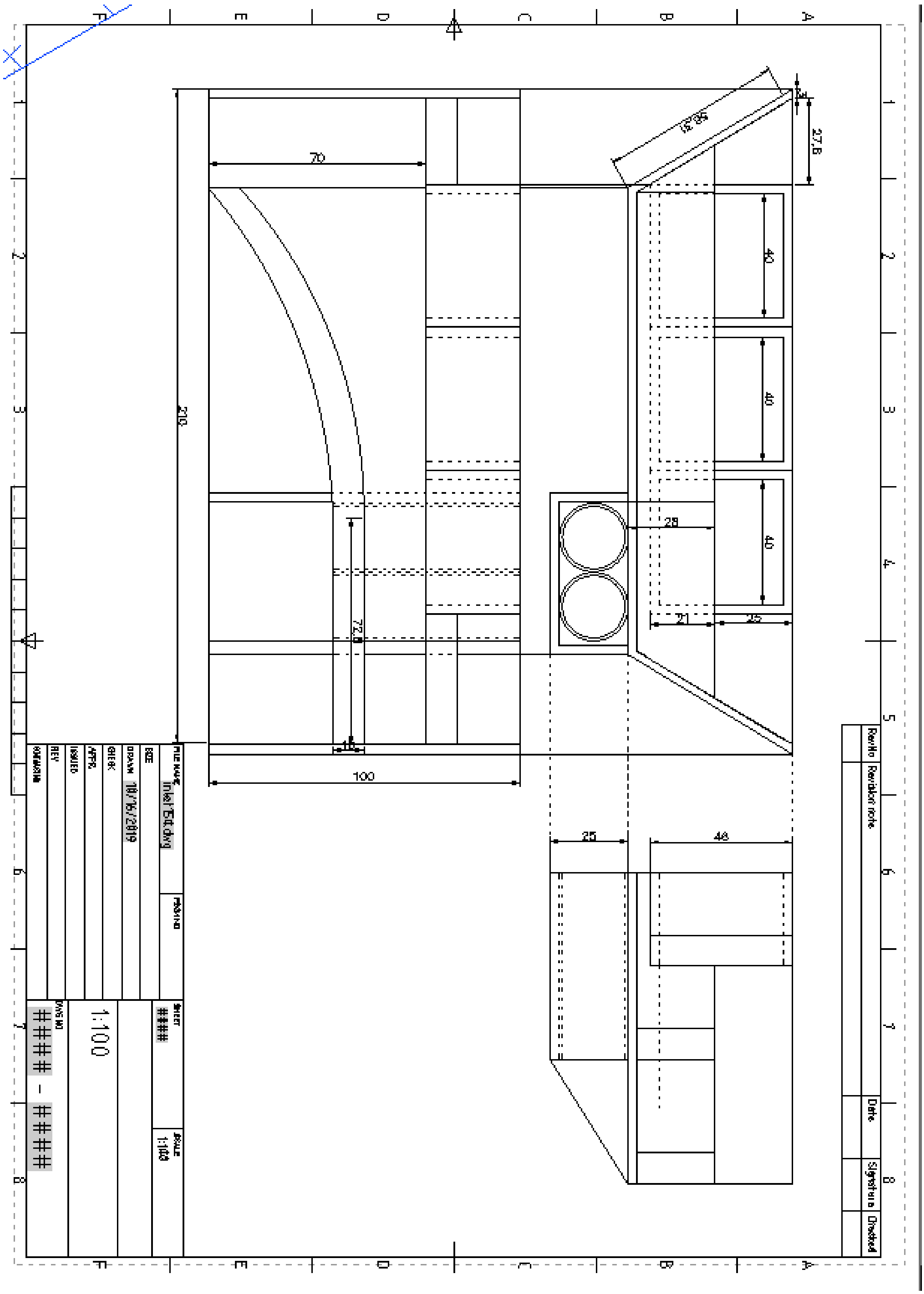
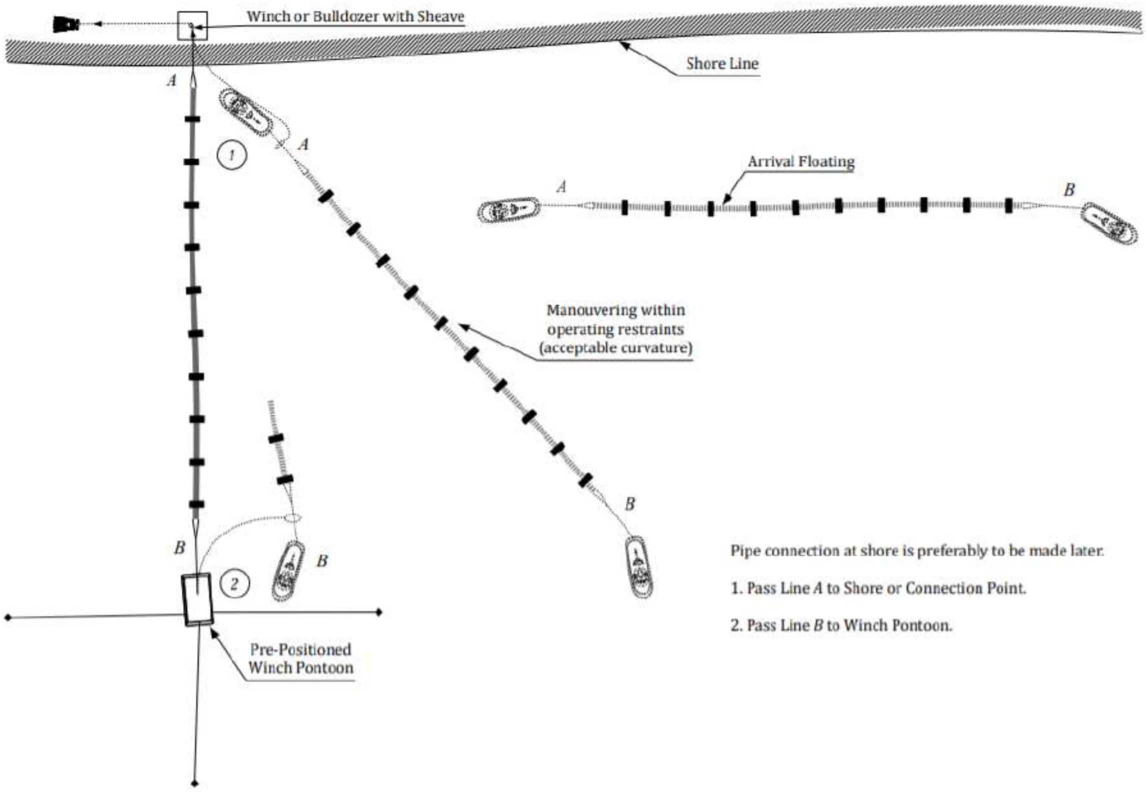


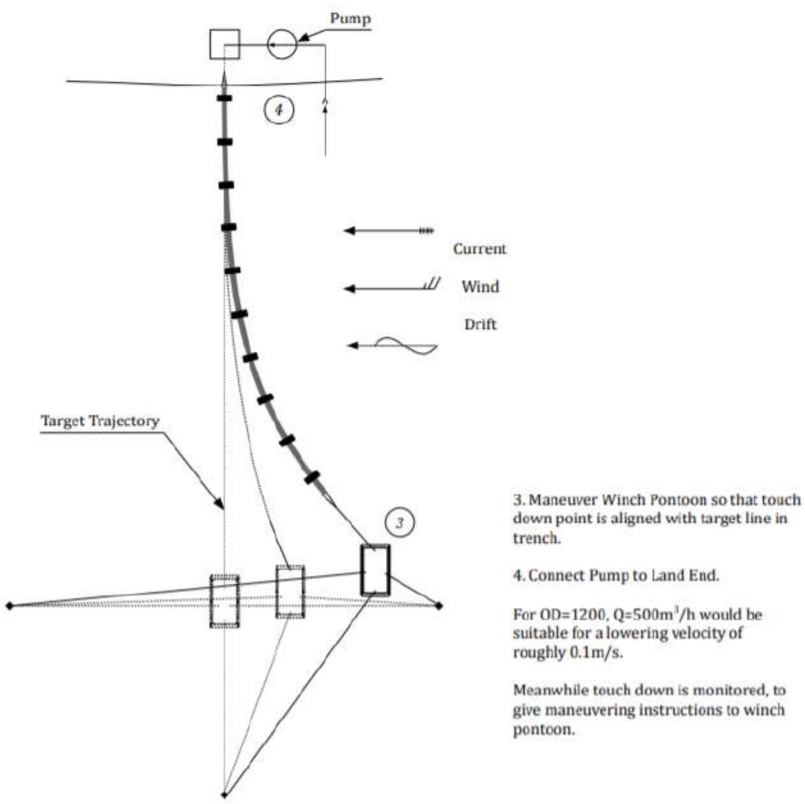
Figure E.4: Technical drawing 150 m^3/s inlet structure

E.2 Construction of outfall pipelines

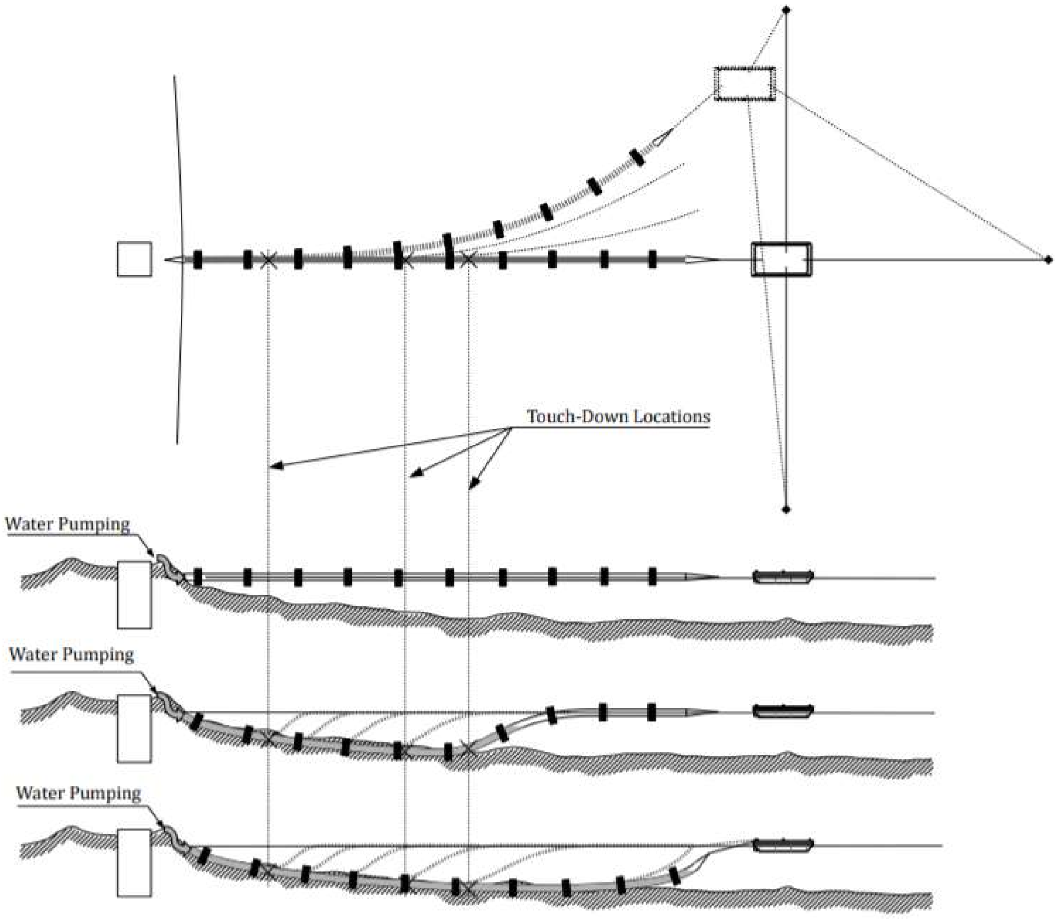
On the following pages a step by step description is given of the placement of the outfall pipeline at sea.



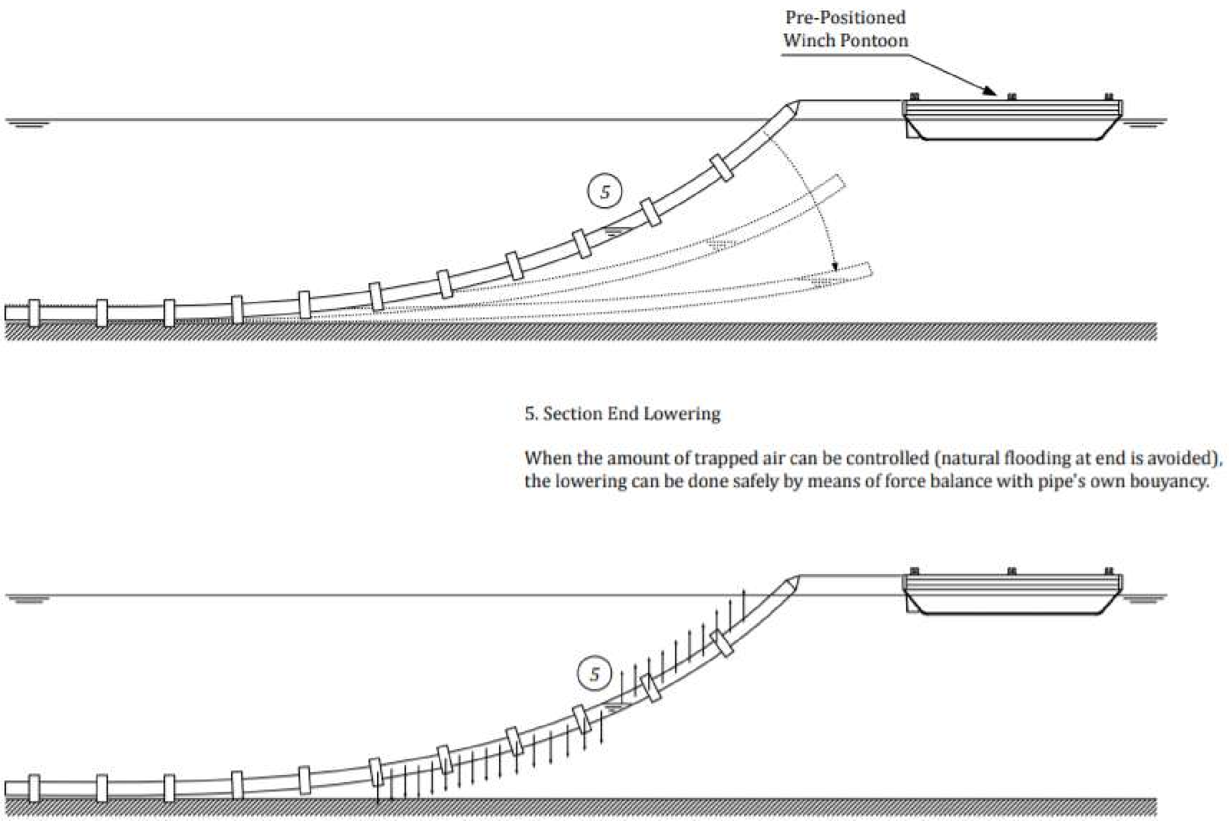
(**a**) step 1



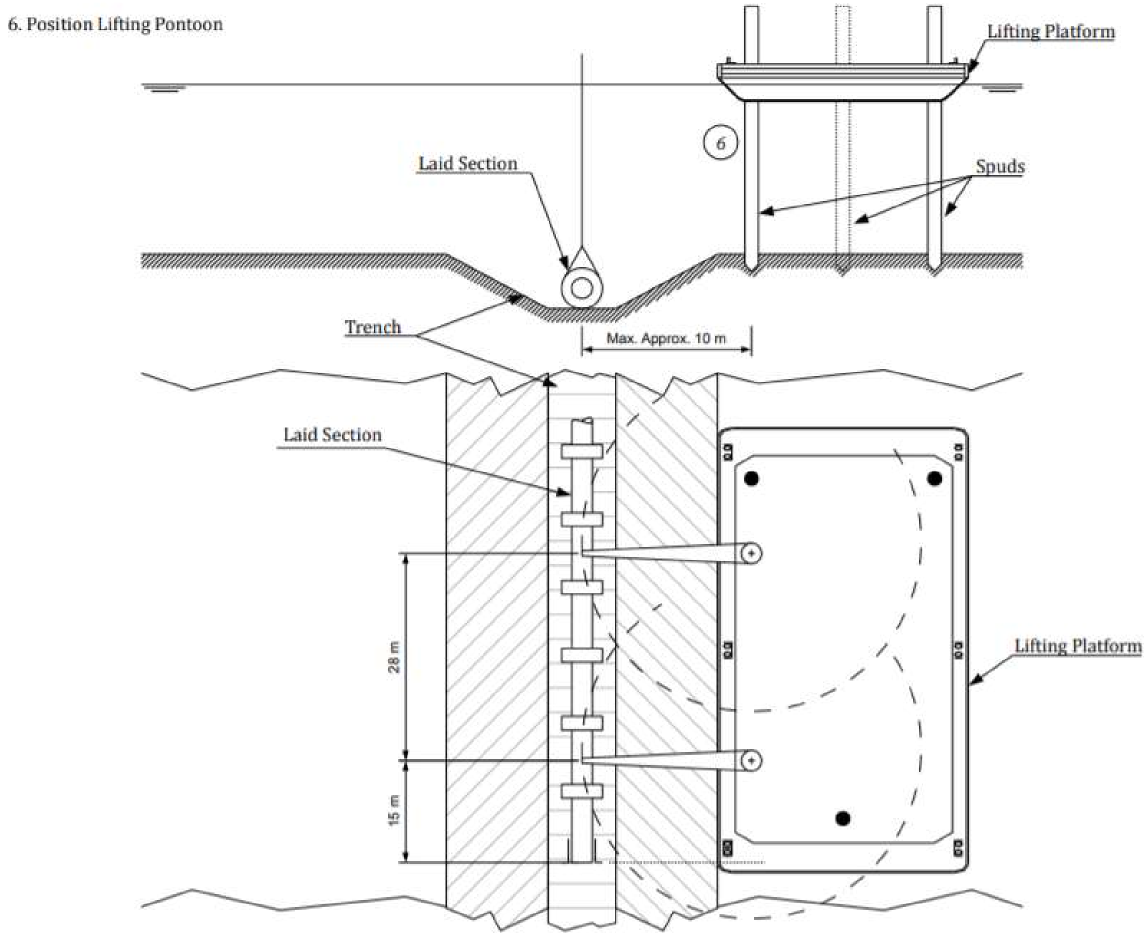
(**b**) step 2



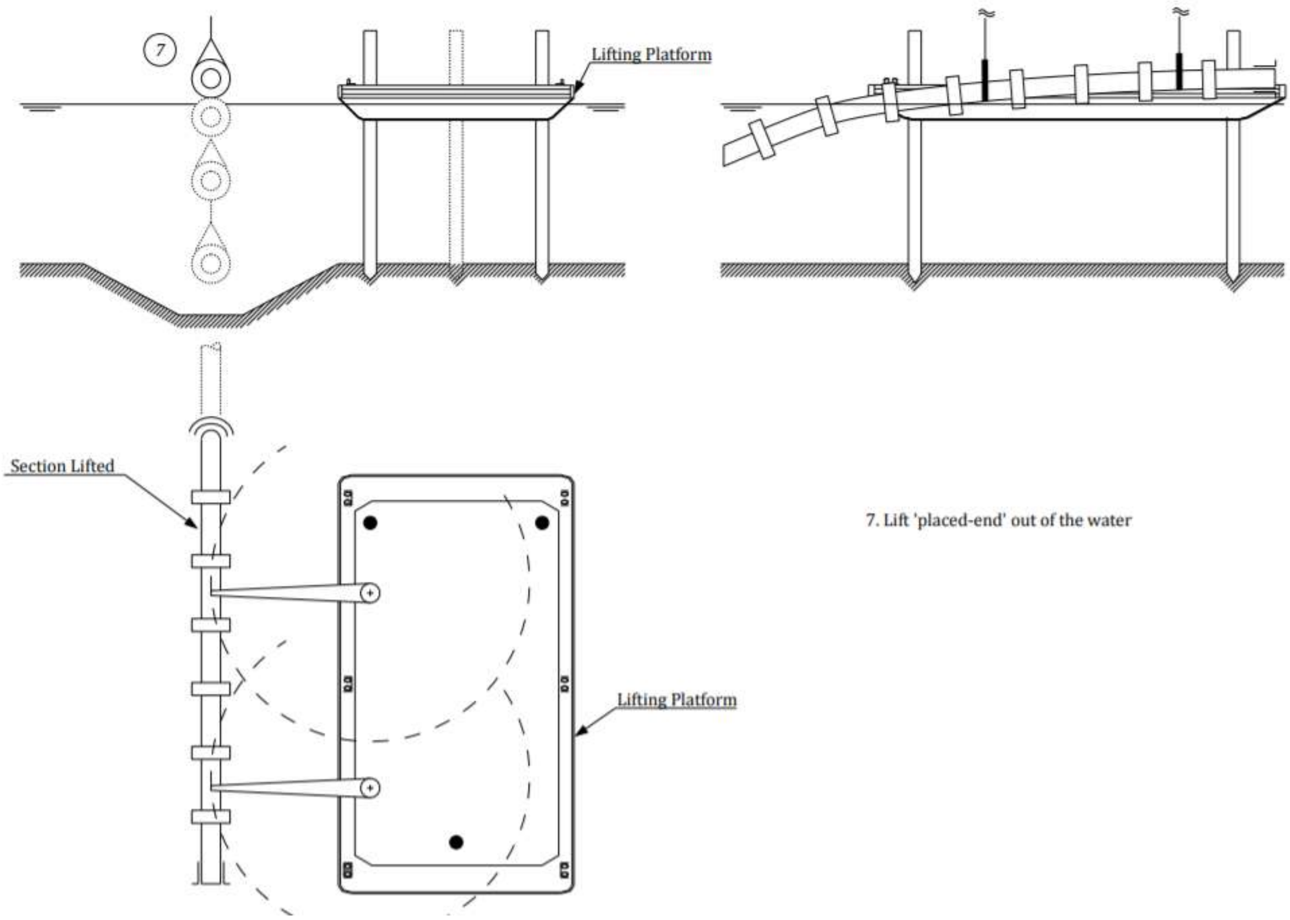
(a) step 3



(b) step 4

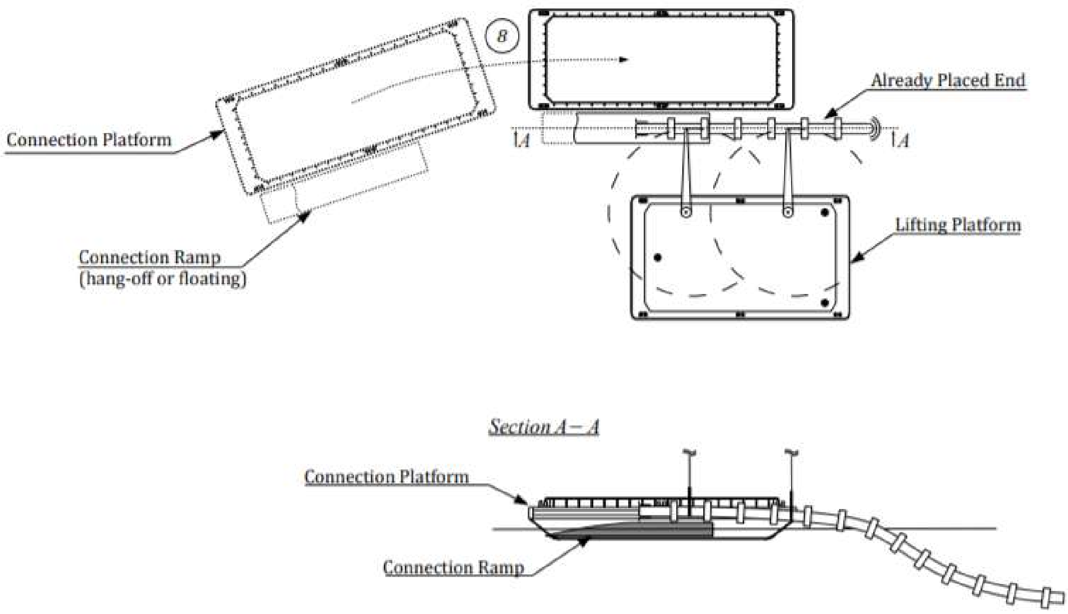


(a) step 5



(b) step 6

8. Position Connection Platform
Connection Ramp placed just
below lifted section.

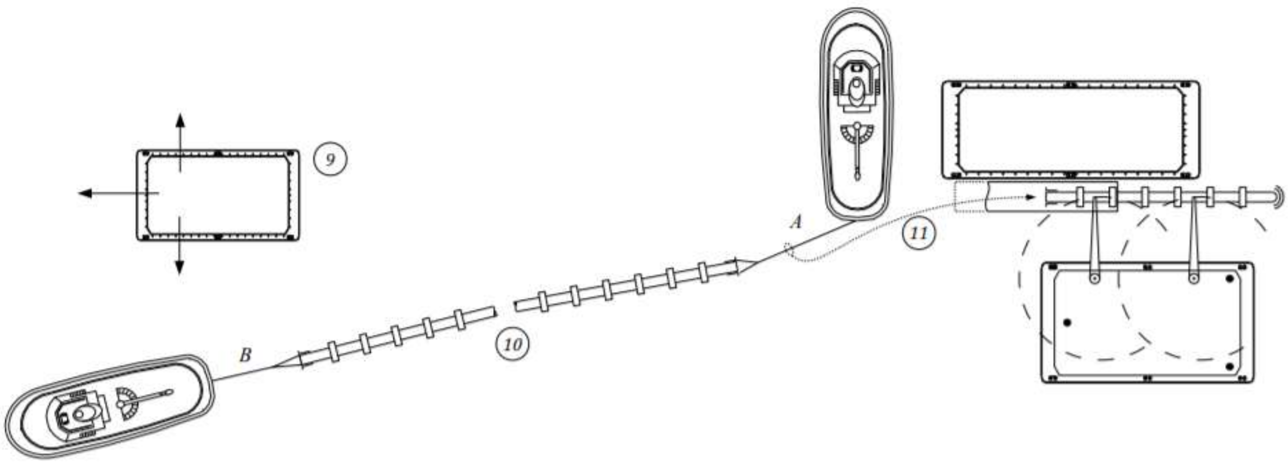


(a) step 7

9. Bring Winch Pontoon to new
end position

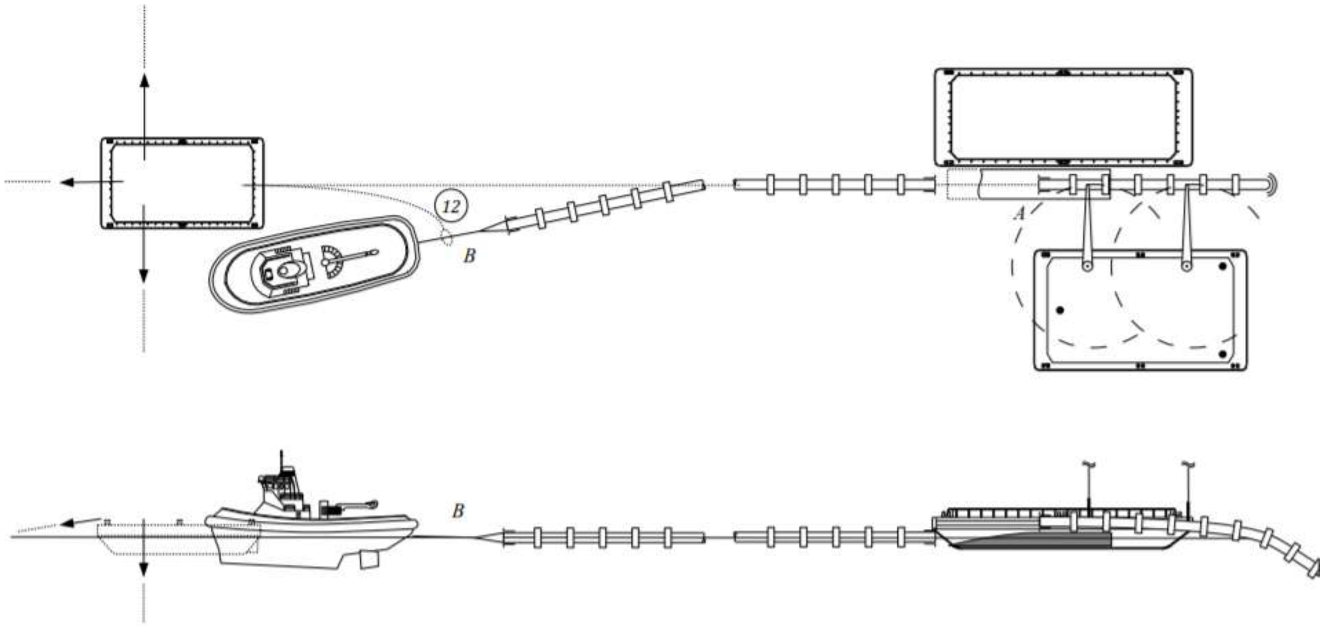
10. Bring new section

11. Pass wire *A*.



(b) step 8

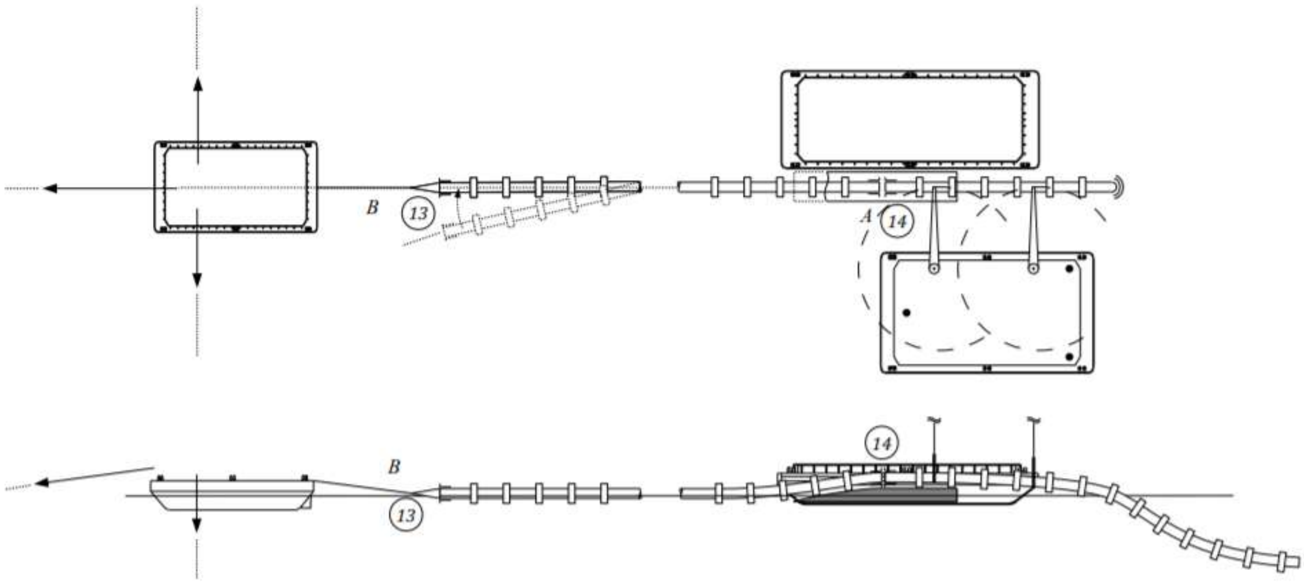
12. Pass wire *B* to Winch Pontoon



(a) step 9

13. Align end B, by maneuvering winch pontoon, so that section arrives inline at ramp

14. Pull end A on ramp and make connection

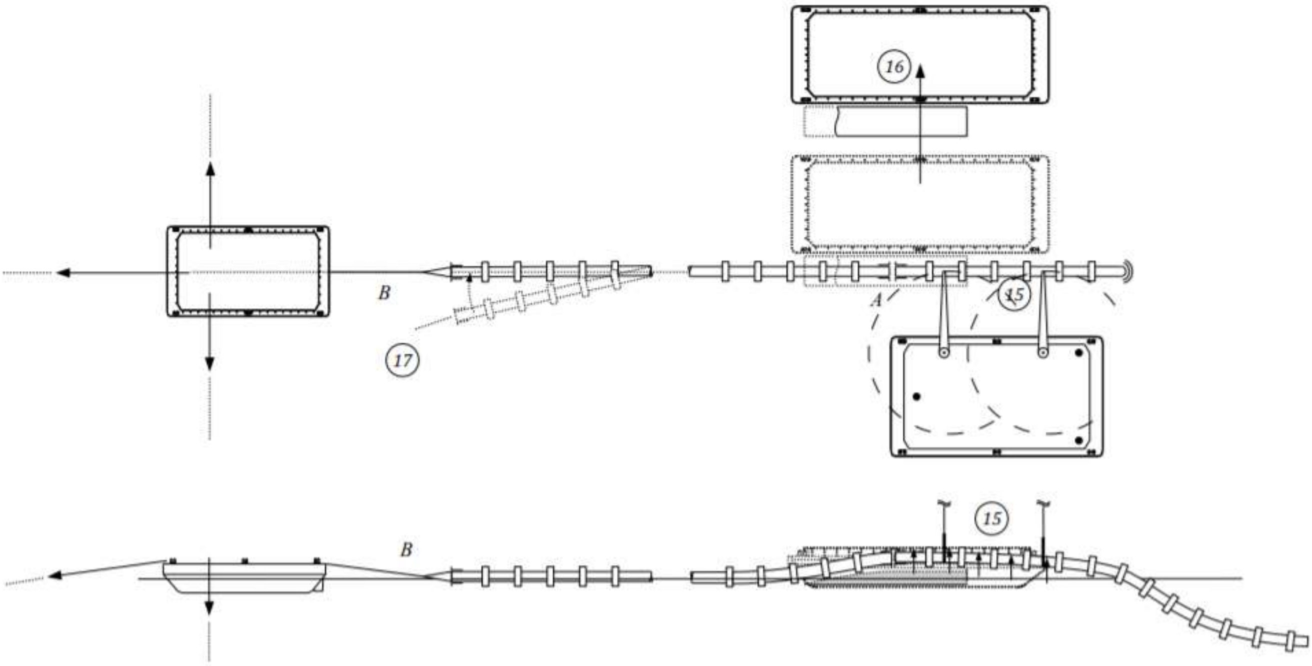


(b) step 10

15. Lift connected sections

16. Pull Connection Platform away from underneath Outfall

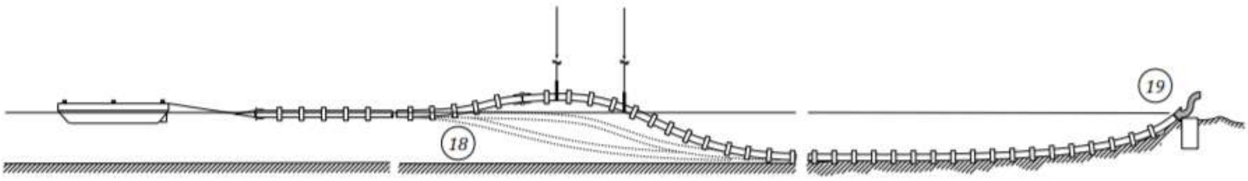
17. Check Alignment



(a) step 11

18. Lower lifted part

19. Start pumping again



(b) step 12