Hazira Harbour Siltation Study

A new harbour in the Gulf of Khambhat, India

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Preface

This Master of Science thesis presents my graduation project leading to the completion of my study of Civil Engineering at the Delft University of Technology, Faculty of Civil Engineering and Geosciences, Department of Hydraulic Engineering. It has been carried out in association with Delta Marine Consultants, a subsidiary of HBG Civiel.

The study concerns the siltation of the proposed harbour at Hazira, which is situated in the region Gujarat at the Northwest coast of India. Shell intends to develop this harbour and has requested Delta Marine Consultants to provide the engineering services. The annual siltation rate inside the new harbour basin determines, for a large part, the economic feasibility of this harbour because it is an ongoing necessity.

I would like to thank prof. dr. ir. M.J.F. Stive (Delft University of Technology), dr. ir. Z.B. Wang (Delft University of Technology and WL | Delft Hydraulics), ir. J.S. Reedijk (Delta Marine Consultants) and, especially, dr. ir. J.C. Winterwerp (Delft University of Technology and WL | Delft Hydraulics) for their time and commitment, and for sharing their knowledge and insight during this study. Finally, I would like to thank my family, Pieter and friends for their support and making the time I spent in Delft a very pleasant one.

Gouda, April 2002

Marloes Timmermans

Disclaimer

Summary

The Royal Dutch / Shell Group of Companies intends to implement a Liquefied Natural Gas import terminal in the new general port at Hazira. This terminal is expected to meet the fuel demands of the nearby steel and power plant, as well as to supply gas to more remotely situated industrial customers, as the domestic gas supply of India is not likely to keep pace with the demand. Shell has asked Delta Marine Consultants, a subsidiary of HBG Civiel, to provide engineering services for the proposed harbour.

Hazira is situated at the Northwest coast of India, at the Eastern side of the Gulf of Khambhat. The Gulf of Khambhat is known for its strong tidal currents and large tidal flats. Furthermore, it is affected by the Southwest monsoon. This period can be characterised by strong winds and high waves. Initially, the port is designed to accommodate the LNG berth. However, with some additional dredging and some other changes, space is created for additional berths in the future.

The decision has been taken to develop the proposed Hazira harbour in three separate phases, in which each phase can represent the final stage of the harbour. As a result, the flexibility of the port development is increased. The final stage reflects a semi-enclosed harbour basin.

Flow velocities within a semi-enclosed harbour are generally much smaller than the flow velocities in front of such a harbour. This leads to a reduction in transport capacity of the water when entering the basin. Consequently, part of the penetrating suspended sediment will settle in the basin.

In the present study, an estimate of the expected siltation rate is provided for the proposed Hazira harbour, in its final stage of development. Two layouts are compared with each other: (1) a semi-enclosed harbour in which only an LNG terminal is present, and (2) a fully operational port, which includes a bulk and a container terminal as well. Once the siltation rate is known, insight into the maintenance costs and hence, the economic feasibility of the new harbour can be assessed. Maintenance dredging, being an ongoing necessity, determines for a large part the cost structure of the harbour.

The annual siltation rate inside the semi-enclosed basin is proportional to the sediment influx per tide. The volume of water penetrating the basin per tide and its sediment content determine this influx. Furthermore, the annual deposit rate depends on the trapping efficiency of the basin and the dry bulk density of the deposits.

As a result of the highly dynamic character of the Gulf of Khambhat, the water carries a large amount of suspended sediment. The amount of total suspended solids shows considerable variations in time. Variations over a tidal cycle as well as more seasonal variations can be distinguished. At Hazira, near the surface, a decrease of total suspended solids is found at and just after slack water. The sediment particles start to settle and an increase in the amount of total suspended solids can be observed at the seabed. Apart from the tide, the sediment concentration is also expected to be influenced by the climate at the West coast of India. In the present thesis, an annual mean sediment concentration of 1000 mg/l is assumed.

The water exchange between the proposed Hazira harbour and the Gulf of Khambhat is mainly caused by: (1) filling and emptying by the tide, and (2) entrainment by longitudinal flow effects.

No water exchange due to entrainment by cross flow effects and due to salinity-induced density currents is. Furthermore, it is found that the influence of the water intake can be neglected. The exchange by the first mechanism, the tidal prism, equals $8.7*10^6$ m^3/tide for a port, which only accommodates the LNG berth and $6.9*10^6$ m^3/tide for the 'full port layout'. The tidal prism can easily be determined by the product of the tidal range and the storage area of the basin.

The exchange by entrainment due to longitudinal flow effects takes care of the greatest part of the total water exchange volume. This exchange mechanism is expected to generate an eddy in the entrance of the proposed Hazira harbour basin. At present, no studies concerning eddies that are generated by entrainment due to longitudinal flow effects are available. For quantifying this water exchange mechanism, an analogy with the existing harbour at IJmuiden is established. The water exchange caused by entrainment by longitudinal flow effects is found to be proportional to the ratio of a characteristic velocity at Hazira to a characteristic velocity at IJmuiden, the mean entrance depth and a characteristic width of the entrance. The velocity ratio, and hence the exchange volume, appears to be sensitive to the location that is chosen for comparison of the characteristic current velocities at Hazira and at IJmuiden. Comparison of the flow velocities just outside the harbour entrances, yields a water exchange caused by entrainment due to longitudinal flow effects that varies in between $2*10^7$ and $3*10^7$ m^3 per tide. However, when the undisturbed flow velocities, without influence of the harbour, are compared with each other, the water exchange caused by this mechanism varies in between $5*10^7$ and $6*10^7$ m^3 per tide. It is hypothesised that a velocity ratio of one reflects the lower boundary of the water exchange due to entrainment by longitudinal flow effects, whereas a velocity ratio of two can be seen as the upper boundary.

A sediment trapping efficiency that varies in between 30 and 50 percent is expected at the Hazira harbour basin. It depends on the water depth over the fall velocity as a measure of the time required for total settling, and the time available, the retention time. At Hazira, two sediment-trapping processes are anticipated, which may lead to a high trapping efficiency:

- It is hypothesised that the sediment concentration profile collapses when the water enters the Hazira harbour basin and a layer of high-concentrated fluid mud is formed near the seabed. This collapse may lead to the development of a sediment driven density current. The interaction between the collapse of the concentration profile and the density current that is generated by this collapse is expected to lead to rapid siltation in the Hazira harbour basin.
- The primary eddy, which is anticipated in the Hazira harbour entrance, is expected to drive a secondary eddy in the basin. Secondary eddies can be of great influence on the sediment transport and the trapping efficiency of the basin, as they can transport the sediment particles further into the harbour basin where the particles finally settle.

The dry bulk density is determined based on data of dredged materials of harbour basins of the port of Rotterdam in the Netherlands. It is assumed to be approximately 350 kg/m^3.

The foregoing leads to an expected siltation rate in the proposed Hazira harbour basin of a multitude of ten million cubic metres of mainly fine-grained sediment per year. Again, it depends on the location chosen for comparison of the current velocities at Hazira and at IJmuiden. The use of a three-dimensional flow model at the proposed Hazira harbour entrance is strongly recommended. Laboratory experiments can also lead to an improved understanding of the water exchange due to entrainment by longitudinal flow effects. Furthermore, research on the consolidation rate of the deposited materials, as well as additional survey on the sediment concentration is recommended.

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1 Introduction

Given that the domestic gas supply of India is not likely to keep pace with the demand, the Indian government has planned a number of new Liquefied Natural Gas (LNG) terminals. The Royal Dutch / Shell Group of Companies intends to develop one of the proposed LNG import terminals. This terminal is situated at Hazira, in the region Gujarat at the Northwest coast of India. Shell has asked Delta Marine Consultants to provide engineering services for this proposed harbour.

The decision has been taken to develop the proposed Hazira harbour in three separate phases. In the last development phase, a semi-enclosed harbour is constructed. It generally holds that flow velocities within a semi-enclosed harbour basin is much lower than the flow velocities in front of such a harbour. Consequently, the transport capacity of the water when entering the basin reduces and part of the incoming suspended sediment settles in the basin.

The proposed harbour is situated at the Hazira Peninsula, at the Eastern side of the Gulf of Khambhat, an inlet of the Arabian Sea. As a result of the highly dynamic character of the Gulf of Khambhat, the water carries a large amount of suspended sediment and high sedimentation rates are expected.

The objective of the present study is to provide a reliable estimate of the expected siltation rate for the proposed Hazira harbour, in the final development stage. Once the siltation rate is known, insight into the maintenance costs and hence, the economic feasibility of the new harbour can be assessed. Maintenance dredging, being an ongoing necessity, determines for a large part the cost structure of the harbour.

The annual siltation rate in the semi-enclosed Hazira harbour basin is determined through three different quantities: the sediment influx per tide, the part of incoming sediment that settles in the basin and the dry bulk density of the deposits.

The outline of this thesis is as follows: Chapter 2 deals with the problem analysis of the Hazira harbour project, which results in the objective of the present study. The characteristics of the area around the proposed harbour and its morphological impact are dealt with in chapter 3. Chapter 4 classifies estuaries according to their salinity structure and it determines the salinity distribution in the Gulf of Khambhat. The annual siltation rate inside the semi-enclosed Hazira harbour basin is proportional to the sediment influx per tide. The sediment content of the penetrating water and the total volume of water exchange determine this influx. These parameters are discussed in chapter 5 and 6 respectively. It is expected that a part of the penetrating sediment settles in the basin. Chapter 7 deals with the sediment trapping processes that can be distinguished. After determining the dry bulk density of the deposits, all the phenomena that contribute to the total rate of exchange have been quantified. Chapter 8 provides an estimate of the annual deposit rate of the proposed Hazira harbour. It also deals with the sensitivity of the different quantities. Chapter 9, discussion, compares the results of the present study with two siltation studies as carried out by Svašek, Coastal & Harbour Engineering Consultants. Furthermore, the assumed amount of total suspended solids in the vicinity of Hazira is compared with the mean sediment concentration measured in other port studies in the Gulf of Khambhat. Finally, chapter 10 deals with the conclusions and recommendations with regard to this siltation study.

2 Problem analysis

2.1 Introduction

The Royal Dutch / Shell Group of Companies intends to implement a Liquefied Natural Gas (LNG) import terminal at Hazira, in the region Gujarat at the Northwest coast of India. This terminal is expected to serve the fuel demands of the nearby steel and power plant, as well as to supply gas to more remotely situated industrial customers. Shell has asked Delta Marine Consultants (DMC) to carry out a detailed design of the proposed port.

This chapter explains the need for good insight in the annual maintenance dredging rate of the planned harbour. After description of the background of the Hazira harbour project in section 2.2, the different layouts, as proposed by DMC, are described in section 2.3. From this analysis, the problem definition is formulated in section 2.4. Finally, section 2.5 discusses the main objective of this study and gives an overview of the approach used in this research project.

2.2 Project background

The Republic of India is the world's sixth largest energy consumer. To keep up with the increasing demand, the Indian government plans major energy infrastructure investments. The Indian consumption of natural gas has risen faster than any other fuel in recent years. From only $17.0*10^9$ m^3 per year in 1995, natural gas use was nearly $22.7*10^9$ m^3 in 1999 and is projected to reach $36.8*10^9$ m^3 in 2005 and $51.0*10^9$ m^3 in 2010. Increased use of natural gas in power generation is accountable for most of the increase, as the Indian government is encouraging the construction of gas-fired electric power plants in coastal areas where they can easily be supplied with LNG by the sea.

Given that the domestic gas supply is not likely to keep pace with demand, India is expected to import most of its gas requirements in the future, either via pipeline or LNG tanker. The Royal Dutch / Shell Group of Companies is going to develop one of the proposed terminals as planned by the India's Foreign Investment Promotion Board (FIPB). Shell is expected to implement an LNG import terminal with all the marine facilities in the new general port at Hazira. Hazira is situated in the region Gujarat at the Northwest coast of India. This planned terminal meets the fuel demands of the nearby steel and power plant, as well as supplying gas to more remotely situated industrial customers. In the following map of India (Figure 2.1), the location of Hazira is pointed out.

Figure 2.1 India

The LNG terminal is initially sized to handle 2.5*10^9 kg/year, expanding first to a throughput of 5*10^9 kg/year, but on the longer term a throughput of 10*10^9 kg/year is required. Initially the port is designed to accommodate the LNG berth. With additional dredging and some other changes, the design of the port has to create space for additional berths in the future.

2.3 Proposed harbour layout

Delta Marine Consultants (DMC) has been asked by Shell Global Solutions, at the beginning of 2000, to provide engineering services for the proposed Hazira harbour. Initially, this has resulted in a detailed design, including an LNG jetty and an LNG terminal platform protected by a breakwater and a Northern spur. This layout is shown in appendix A. In this layout, the anticipated terminal for general cargo ships and bulk carriers, as well as the planned container and roro terminal, are already reflected.

As the development of a semi-enclosed harbour for an LNG terminal is rather expensive, Shell decided to study the development of the proposed harbour in three different phases. Each phase can represent a final stage of the harbour, which increases the flexibility of the port development. DMC has also carried out the design of the several phases, in which the third phase is approximately similar to the initial design presented in appendix A.

In phase 1 the LNG terminal and a berth for an LNG carrier are constructed. Phase 2 consists of the development of a terminal for general cargo ships and bulk carriers. In the final phase, the construction of the protective breakwater and the container terminal is planned. The three separate phases are shown in Figure 2.2.

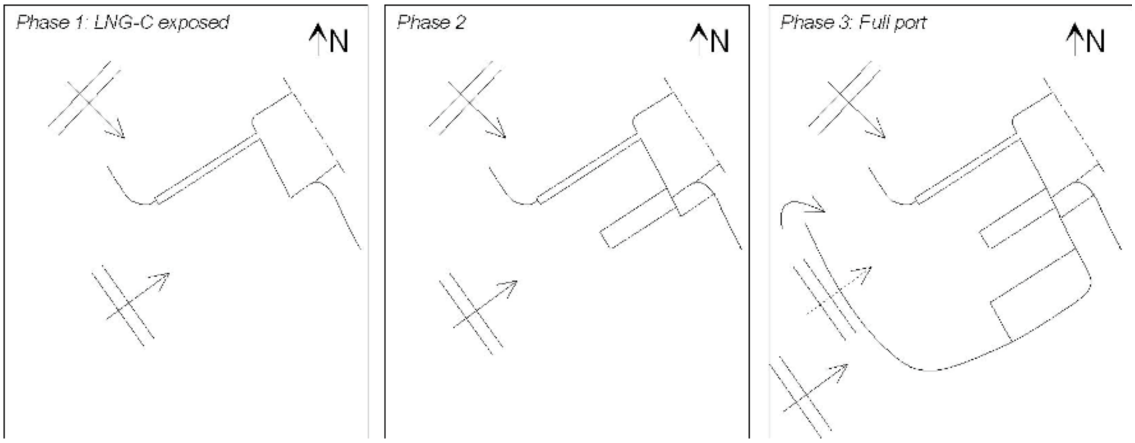


Figure 2.2 Harbour development phases and wave loading

In the initial design, the breakwater is expected to protect the marine structures against the Southwest monsoon winds and waves. In the updated design, it is unknown how long the construction works of phase 1 and 2 remain unprotected. Among other things, this depends on the willingness of other companies to invest in the harbour. The works of phases 1 and 2 have to withstand the SW monsoon and cyclone generated waves, which can come from the Southwest and the Northwest.

2.4 Problem definition

Flow velocities within a semi-enclosed harbour are generally much smaller than the flow velocities in front of such a harbour. This leads to reduction of transport capacity of the water when entering the basin. Consequently, part of the penetrating suspended sediment settles in the harbour basin.

The annual siltation rate inside a basin is proportional to the sediment influx per tide. The water exchange volumes per tide and the mean sediment content of the penetrating water determine this influx. Furthermore, the trapping efficiency of the basin and the dry bulk density of the deposits influence the sedimentation rate in the basin.

Perception of the annual siltation rate is an important figure for the economic feasibility of a harbour, as maintenance dredging is an ever-returning necessity.

2.5 Objective of this study

The objective of this study is to provide an estimate of the expected siltation rate in the proposed Hazira harbour basin, in the final stage of development. The initial design, in which only an LNG terminal is present, is compared with the situation of a fully operational port, which includes the bulk and container terminals as well. Once the siltation rate is known, insight in the maintenance costs and hence, in the economic feasibility of the harbour, can be provided.

Figure 2.3 gives an overview of the subjects of study to fulfil the main objective of this study.

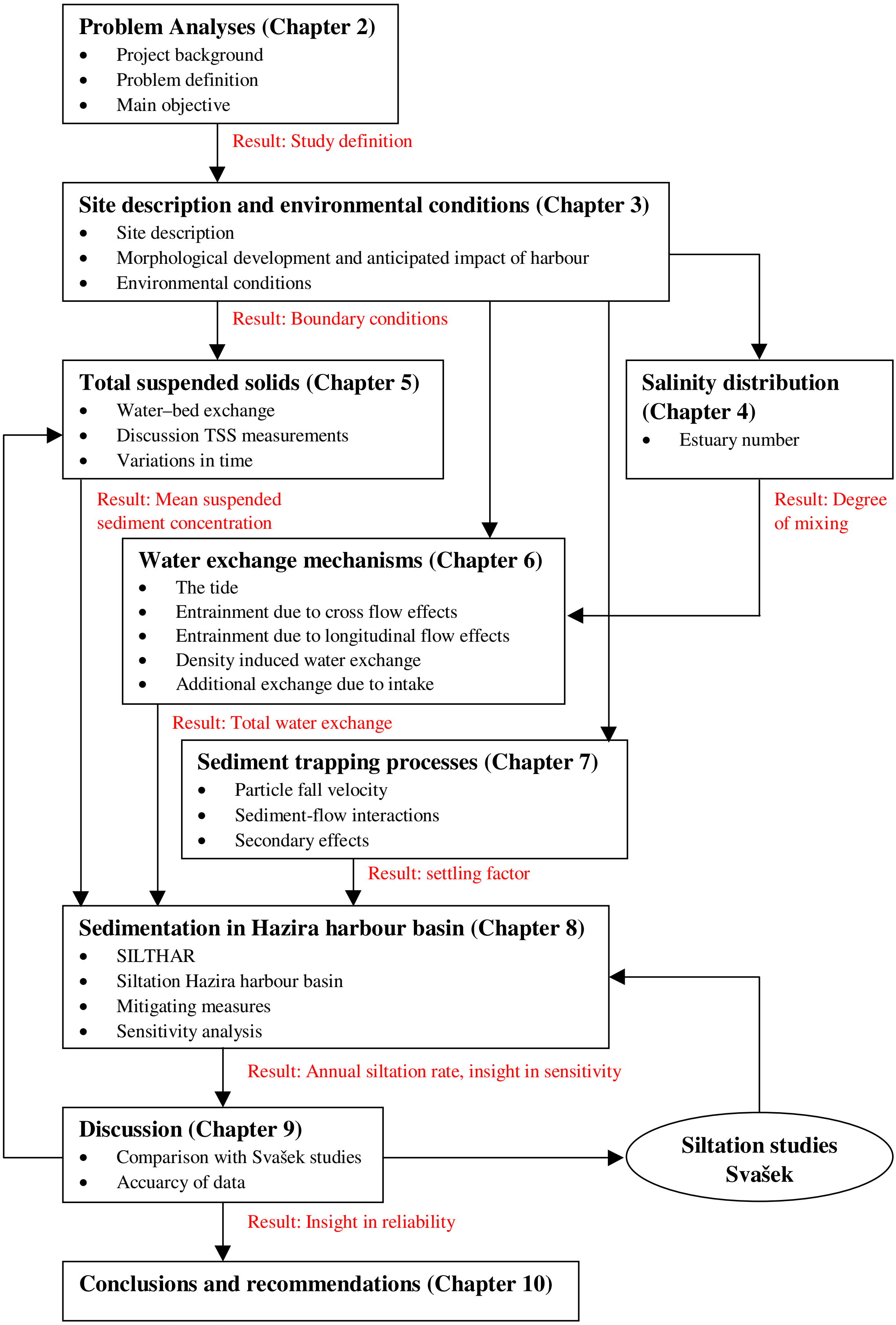


Figure 2.3 Set-up of the study

3 Site description and environmental conditions

3.1 Introduction

This chapter deals with the general characteristics of the area around the proposed Hazira harbour and the expected morphological development after construction of this harbour. This forms the basis for the present siltation study.

Section 3.2 gives a description of the site location. Subsequently, the important environmental conditions in the vicinity of Hazira are discussed in section 3.3. Finally, the expected impact of the proposed harbour in relation to the autonomous development of the coastal area is assessed in section 3.4.

3.2 Site description

The proposed harbour is situated at the Hazira Peninsula at the Eastern side of the Gulf of Khambhat, a sea arm of the Arabian Sea. See Figure 3.1. The Gulf of Khambhat can be characterised as a large cone-shaped estuary with a length of 210 kilometres and a width of 200 kilometres its mouth. The estuary is tide dominated. Due to partial reflection of the tidal wave in the Gulf and due to its geometry, the vertical range increases in Northern direction. The tidal range is the highest at the head of the Gulf of Khambhat, where the maximum spring tidal range is about 9 metres.

The estuary width at the Hazira peninsula is approximately 60 kilometres. The adjacent coastal region is characterised by a wide, shallow and partly drying foreshore. Tidal channels intersect the several shallow banks in the Gulf of Khambhat. All the channels follow a NNE-SSW direction. The channels and shoals may move under the influence of the strong tidal currents. Flood and ebb channels are recognisable. Flood channels are generally shallower than ebb channels and tend to have dead ends. Ebb channels are deeper and more continuous. The channel directly in front of the harbour entrance, the Sutherland Channel, is a flood dominated channel. This channel has depths from 25 to 30 metres below mean sea level.

At one kilometre South of the proposed harbour, a 1500 metres long dam, which is connected to a land based drilling platform, is situated.

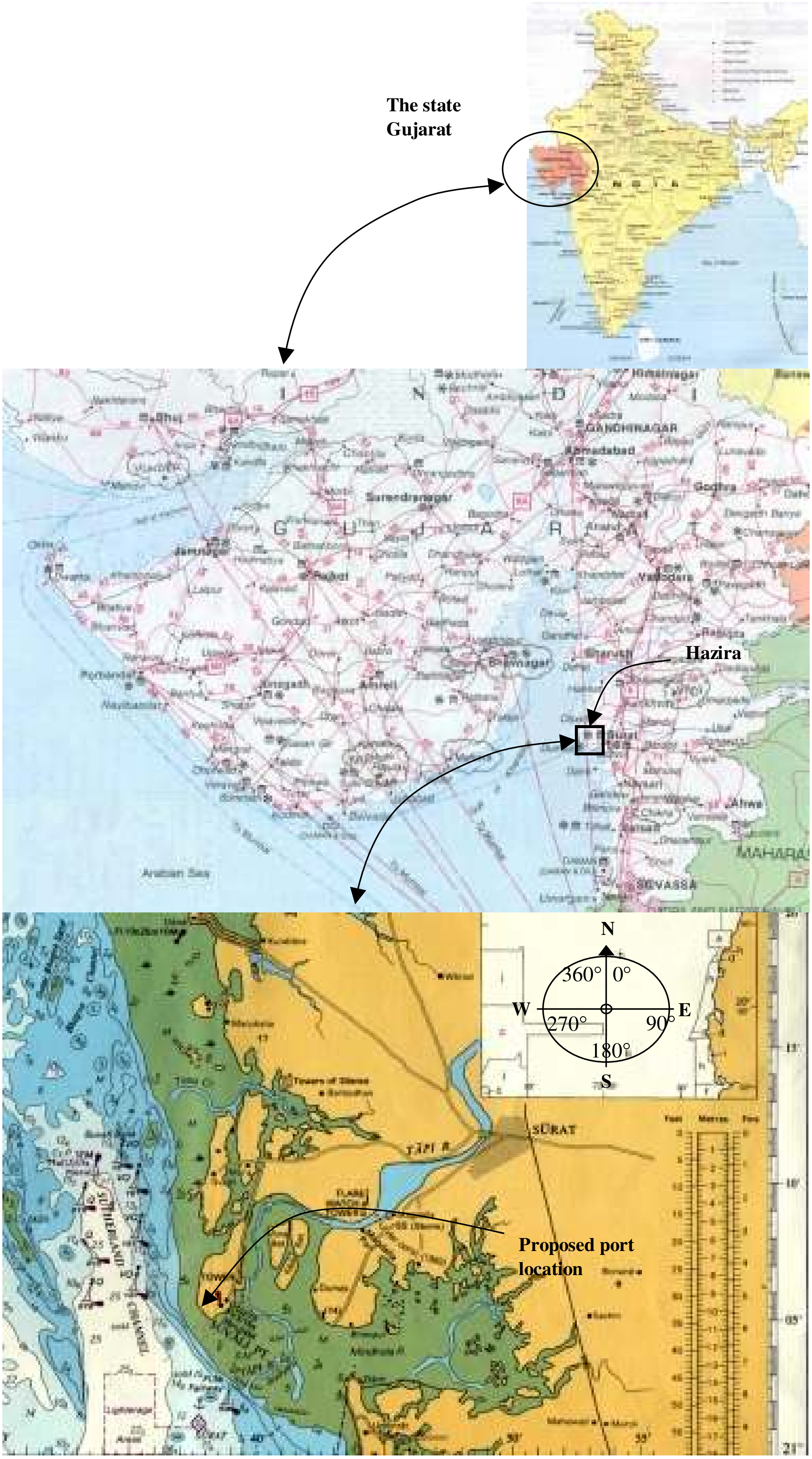


Figure 3.1 Site location

3.3 Environmental conditions

This section summarises the most important environmental conditions around the proposed harbour location. Emphasis is laid upon processes that may influence the siltation. Only characteristics concerning the salinity profile in the Gulf of Khambhat are discussed later on in this report.

3.3.1 Seasons

The weather at the West coast of India has a very seasonal character. The year can be divided into four main seasons, each with specific characteristics. Table 3.1 is based on information from the West Coast of India Pilot (1975, rev. 1986).

Table 3.1 Typical seasons in India

Name of the season	Period	Characteristics
NE monsoon	December-March	Light-moderate NE winds
Hot season	April – mid June	Light SW winds
SW monsoon	Mid June - September	Strong SW winds
Interim period	October and November	Light variable winds

On average, one or two cyclones (Beaufort 12) occur over the Arabian Sea each year. Only eight cyclones have hit the area surrounding Hazira between 1877 and 1982. Six of these cyclones have been of severe nature with wind speeds exceeding 24.2 m/s.

At the onset of the SW monsoon, from late May to the first half of June, and at the onset of the NE monsoon, from late October through the first three weeks of November, the frequency of tropical storms (Beaufort 8-11) reaches a maximum. The influence of tropical storms and cyclones on the sediment transport at the area surrounding Hazira is believed to be relatively small because they are of short duration.

During the SW monsoon period strong winds, high waves and high river discharges occur. As they persist for a longer period, it is expected that the sediment transport increases during this period.

3.3.2 Tidal elevations

The tidal wave enters the Gulf of Khambhat from the South. The tidal range increases going further to the North of the Gulf. In the tidal station Arnala, in the South, the mean tidal range equals 1.5 metres and 3.4 metres for neaps and springs respectively, whereas it amounts to 4.8 and 8.8 metres respectively at the station Bhavnagar in the Northwestern part of the Gulf. The tidal levels at Hazira are given in Table 3.2. All the levels are reduced to Chart Datum.

Table 3.2 Tidal levels at Hazira

	Tidal elevations
Mean High Water Spring	CD+7.31 m
Mean High Water Neap	CD+5.78 m
Mean Low Water Neap	CD+3.06 m
Mean Low Water Spring	CD+1.53 m

Mean Sea Level is 4.4 meter above Chart Datum, hence MSL = CD + 4.4 meter.

The values are derived from site investigations done by Fugro Geos. In general, these values are found to be consistent with the formerly adopted tidal data as derived from the Admiralty Chart no. 1486 and Oceonics data. The tide at Hazira has a considerable daily inequality. This has been summarised in Table 3.3.

Table 3.3 Daily inequality

	Tidal elevations
Highest Astronomical Tide	CD+8.97 m
Mean Higher High Water	CD+7.18 m
Mean Lower High Water	CD+6.03 m
Mean High Low Water	CD+2.35 m
Mean Lower Low Water	CD+1.67 m
Lowest Astronomical Tide	CD+0.02 m

Mean high water is equal to CD+6.6m and mean low water is equal to CD+2.0m. This gives a mean tidal range at Hazira of 4.6 metres. The tidal range increases to the North of the Gulf due to partial reflection of the tidal waves.

3.3.3 Currents

The tidal currents are expected to be proportional to the tidal range. Due to the strong vertical tide, considerable tidal currents occur in the area surrounding Hazira. As stated in section 3.2, the Sutherland Channel, directly in front of the Hazira harbour, is a flood-dominated channel. This implies that the maximum flood velocities are generally higher than the ebb velocities. Fugro Geos has measured the current velocities at four different locations C1-C4 as indicated in appendix B. Table 3.4 reflects the maximum observed current velocities in the Sutherland Channel (location C3).

Table 3.4 Maximum observed current velocities 2.5 m below surface Sutherland Channel

	Maximum flood	Maximum ebb
Spring tide	2.3 m/s	1.8 m/s
Neap tide	1.6 m/s	1.2 m/s

Flood currents are directed North-Northwest (335°-360°) while ebb currents follow a South-Southeastern (165°-180°) direction at the area around the proposed harbour location. Maximum flood occurs 2 hours before high water. Maximum ebb takes place 4 hours after high water.

Although the main breakwater of the proposed harbour is aligned with the flood current in the Sutherland Channel (see Figure 3.1 and appendix A), the presence of the harbour causes higher velocities in the Sutherland Channel due to the contraction and bending of streamlines around the harbour.

Svašek, Coastal & Harbour Engineering Consultants, states that special attention must be given to wind-driven currents during the monsoon period. During the SW monsoon a clockwise circulation is recognisable in the Arabian Sea. However, this clockwise circulation is probably no longer recognisable at Hazira. Svašek assumes that the interaction of the wind shear stress with the depth contours in the Gulf generates an anti-clockwise circulation in the Gulf of Khambhat. See Figure 3.2.

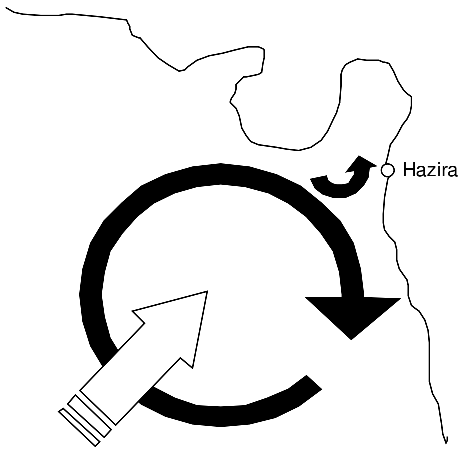


Figure 3.2 Assumed circulation during the SW monsoon

In more shallow areas, the currents follow the wind direction with typical speeds of about three percent of the wind velocity. A mean hourly wind of 14.5 m/s during the SW monsoon period is exceeded nearly one percent of the time.

The effect of the wind-driven currents on the sediment transport on the proposed Hazira harbour may be seen in measurements of wind, current and tide. See appendix C. However, these measurements of July and November 2000 indicate that the effect of the monsoon winds on the tidal currents is limited. Wind set-up creates a hydraulic slope that decreases the tidal range. A smaller tidal range reduces the flood velocities, while the wind drift increases the flood velocities because their direction is the same. As a result, the net effect is negligible.

More data of a longer period is needed to determine whether the monsoon has a significant influence on the maximum current speed or not.

3.3.4 Wave conditions

The monthly variation in the mean significant wave height H_s from June 2000 to May 2001 in the vicinity of Hazira is reflected in Figure 3.3

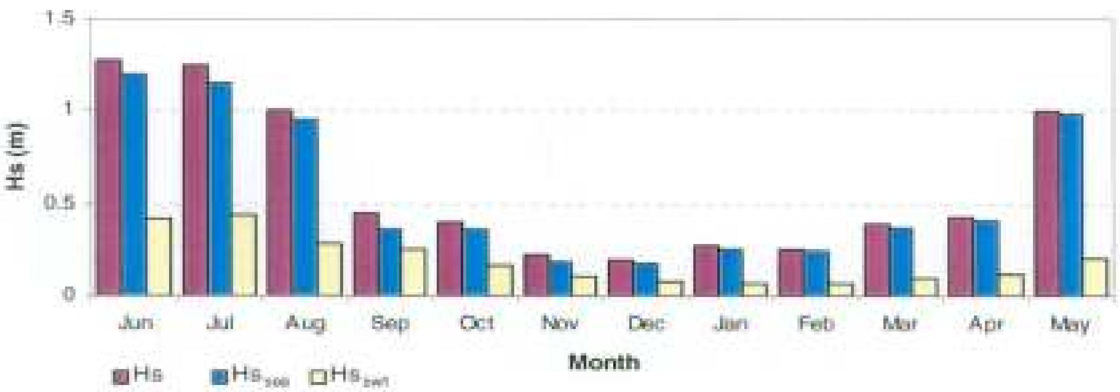


Figure 3.3 Mean significant wave height June 2000 - May 2001

The mean significant wave H_s height peaks at 1.2 metres in June and July 2000, with a maximum value of Hs of 2.3 metres occurring on 14 July. After the monsoon period, the wave heights decrease rapidly with a minimum of 0.2 metres in December 2000. After December, the mean wave heights increase again up to 1 metre in May. Measurements of sea and swell significant wave heights (Hs_{sea} and Hs_{sw1} respectively) show that Hs_{sea} follows almost the same seasonal pattern. The swell significant wave shows less variation and has lower values.

Comparison of waves and wind of the monsoon of the year 2000 with a model of the British Meteorological Office, shows that especially the nearshore swell climate in 2000 was less severe than on average. A reason can be the more Westerly incoming waves. The presence of shallow banks in the vicinity of Hazira makes Westerly incoming waves attenuate more.

Svašek has performed a study concerning the wave climate in the area surrounding Hazira. This study includes the metocean measurements carried out by Fugro Geos in the vicinity of the planned harbour. Svašek has characterised the nearshore wave climate at Hazira based on offshore measurements by the British Meteorological Office over a period of 12 years. These offshore measurements are translated to the area surrounding Hazira by applying reduction factors, which are determined by the mathematical program SWAN.

The yearly mean significant wave height near the planned harbour is about 0.75 metre. However, the nearshore wave climate shows a highly seasonal character. In the SW monsoon period, the wave heights reach their maximum values. During this period, an offshore significant wave height of 8 metres with an occurrence frequency of 1/100 year is assumed. This wave comes from a direction of 210°. This reduces to a significant wave height at Hazira of approximately 4.5 metres during springs. Cyclone waves with a wave height of 12 metres offshore result in a H_s at Hazira of approximately 5.5 metres at springs.

3.3.5 Sediment characteristics

Fugro has surveyed the seabed in front of the planned port location. The thickness of the top layer varies in between two and five metres and mainly consists of fine to coarse dense sands. Occasionally fragments of gravel and shell are found. Below this layer, a medium dense to dense silt layer is found. The deeper layer at about 20 metres below Chart Datum, shows very dense sand that is cemented at some locations.

At some locations at the Hazira foreshore, silt is found. These sediments are supplied by rivers and creeks, like the Tapi River and the Tena Creek (see Figure 3.1). The Tapi River has formed an alluvial delta of which the major part dries during low water. The sediments are easily brought into suspension due to the combined action of flow and waves. The sediments are carried over long distances due to its low settling rate. In sheltered areas, accumulation of these fine-grained sediments takes place. This accumulation can also be found South of the dam at the end of the monsoon period.

3.3.6 Rainfall

From observations by the Surat Airport, just South of the Hazira peninsula, it follows that the annual rainfall is approximately 1200 mm and the number of rainy days is about 50 each year. 95 Percent of the annual rainfall falls during the monsoon period. The rainfall at Surat from

May 2000 to July 2001 is reflected in Figure 3.4. In this Figure, the great differences between the seasons are clearly visible.

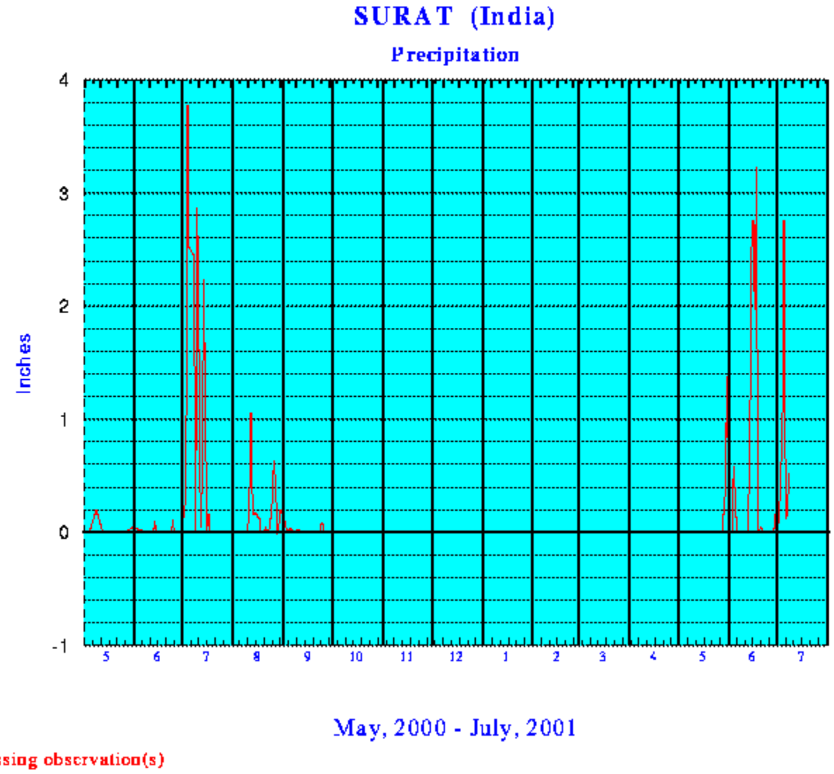


Figure 3.4 Rainfall Surat May 2000 – July 2001

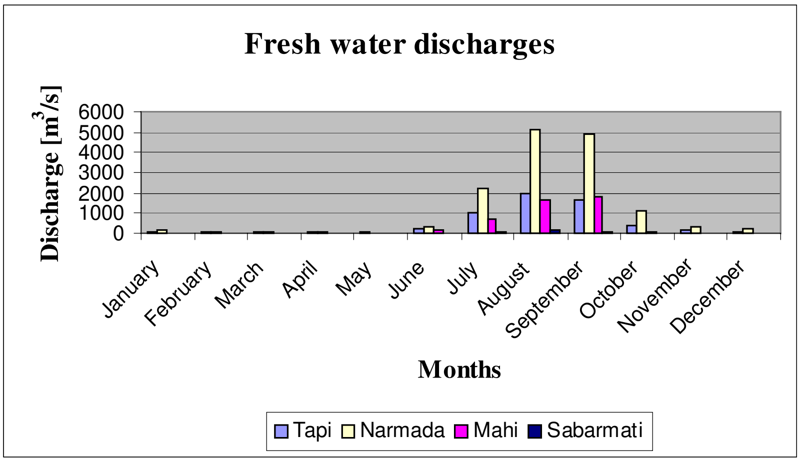
3.3.7 River discharge

To the North and the South of the proposed harbour location, several fresh water sources can be found. The four most important rivers that empty into the Gulf of Khambhat are the Tapi River, the Narmada River, the Mahi River and the Sabarmati River. See Figure 3.5.

Figure 3.5 Fresh water inlets of the Gulf of Khambhat

The Tapi River is in the vicinity of the proposed port. It runs along the East side of the Hazira peninsula and its mouth is situated about 7 kilometres to the South of Hazira.

The fresh water discharge data of the four rivers is obtained from the "Global River Discharge Database" (1998). This database is developed by "The Institute for the Study of Earth, Oceans and Space" from the University of New Hampshire, Durham (USA). It uses the UNESCO river archives and the series of publications entitled "The Discharge of Selected Rivers of the World". These references have been compared with each other and a reliable summary discharge data set has been developed. The discharge of the four rivers as well as the total fresh water discharge into the Gulf of Khambhat is reflected in Figure 3.6 and in Table 3.5.



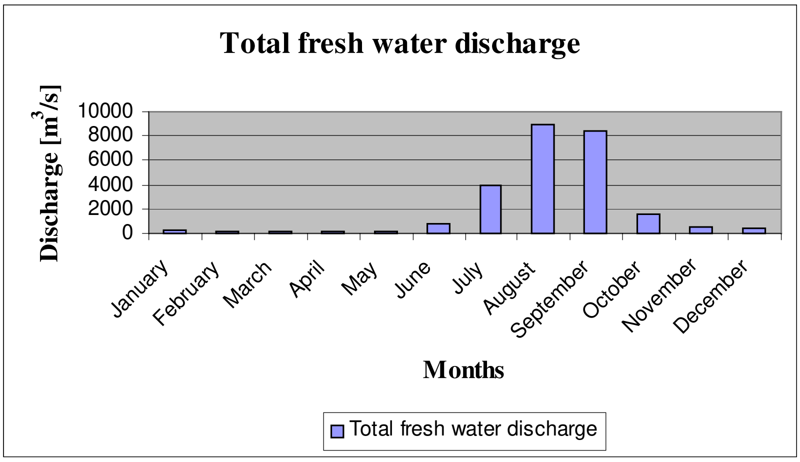


Figure 3.6 Fresh water discharge into the Gulf of Khambhat

Table 3.5 Total fresh water discharge

Month	Total fresh water discharge [m^3/s]
January	231
February	185
March	174
April	135
May	102
June	811
July	3990
August	8916
September	8413
October	1630
November	491
December	364

All the rivers show the same seasonal pattern. During the monsoon period, or the rainy season, the river discharges highly increase. The Narmada River takes care of the largest part of the fresh water discharge in the estuary. It is situated about 65 kilometres to the North of the proposed harbour. The influence of the Sabarmati River on the total discharge can be neglected. See Table 3.6:

Table 3.6 Relative influence of the four rivers

River	Mean part of total yearly discharge [%]
Tapi	33
Narmada	53
Mahi	12
Sabarmati	2

3.3.8 Salinity

Fugro Geos has determined the water salinities in the vicinity of the proposed Hazira harbour in three different ways:

- Salinity and temperature measurements at 0.3-0.5 m above seabed with sampling intervals of 10 minutes by using the Sea-Bird MicroCat CT recorder.
- Salinity and temperature measurements over the depth at the locations of the proposed turning circle, LNG terminal area and bulk terminal area. These profiles have been measured using the Sea-Bird MicroCat CT on 10 second sampling.
- Monthly water samples have been collected from different locations near the proposed Hazira harbour. The collected water samples have been sent to England for analysis.

In this section, the results of the different measuring methods are discussed.

Measurements 0.3-0.5 metres above the seabed

Detailed monthly salinity profiles for the period from June 2000-May 2001 have been provided by using the Sea-Bird MicroCat CT (Conductivity/Temperature) recorder for location C3 and C4 at 0.3-0.5 metres above seabed. In June 2000, the measurements have been carried out at locations S1, S2 and S3 (see appendix B for locations).

The CT recorder has been integrated with an Accoustic Doppler Current Profiler (ADCP), which can provide undisturbed flow velocities at multiple locations in the water column, with an acoustically released marker buoy and with a homer beacon, which monitors the equipment. The configuration is shown in Figure 3.7 and Figure 3.8.

From the monthly salinity profiles results that the tide is an important factor for the level of salinity. Sharp reductions in salinity are found twice each tidal cycle, reducing salinity by approximately 10 parts per thousand at some points.

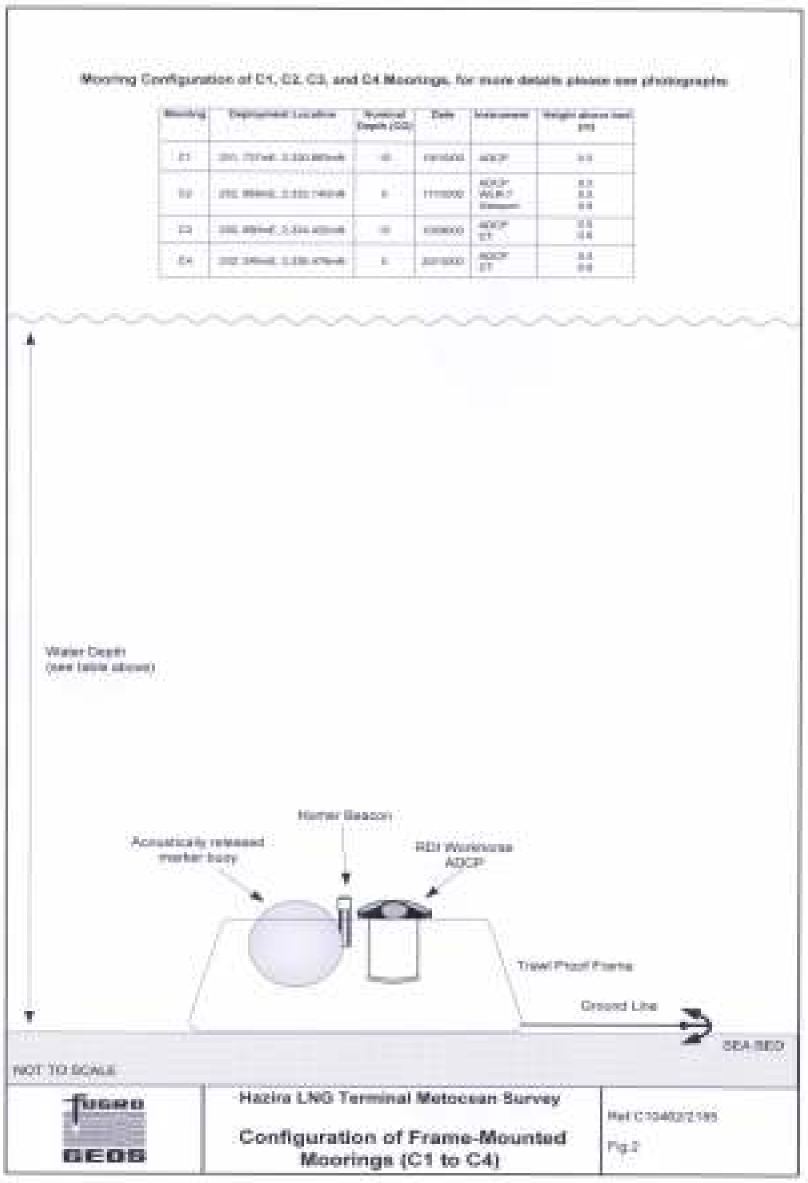


Figure 3.7 Configuration

Figure 3.8 Mooring C3 immediately prior to deployment

Fugro Geos has observed sharp reductions in salinity coinciding with periods of maximum flood and ebb. The, by Fugro Geos, assumed salinity profile, which follows from the monthly data reports, is shown in Figure 3.9.

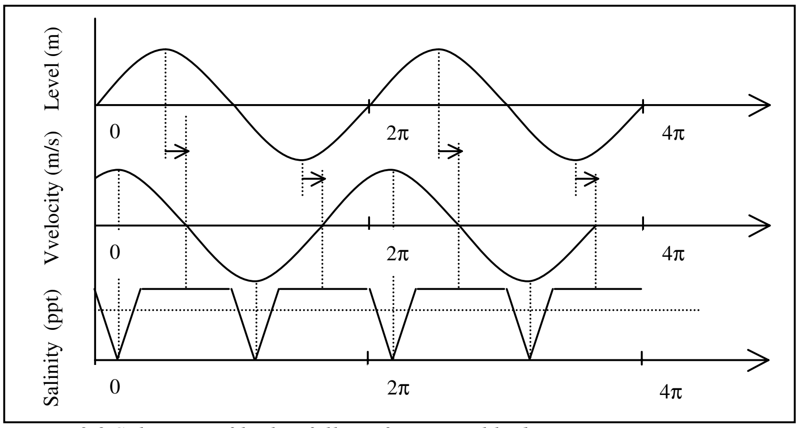


Figure 3.9 Salinity profile that follows from monthly data reports

Salinity measurements over the depth profile

In October and December 2000 and in February, April and May 2001, the MicroCat has been suspended on a cord with a lead clump weight. This has been done at the locations of the proposed turning circle, the LNG terminal and the bulk terminal, at the same time that the water samples have been taken. The locations are shown in Figure 3.10. After two minutes in the water near the surface to equibrilate, the instrument has been lowered at one-metre increments every minute. The depth of the MicroCat has been estimated from the length of the cord deployed, and the measurements at each depth interval have been averaged to obtain the results of conductivity and temperature. In June 2000, the Sea-Bird MicroCat has been deployed at three different locations (S1, S2, and S3) and has measured conductivity and temperature at the surface and seabed. The measured data is summarised in appendix D1. In this appendix, the salinity profiles over the depth are also visualised for the three locations.

Figure 3.10 Locations profiling measurements

From these measurements it can be concluded that there is no difference in salinity over the depth. The estuary is fully mixed at the location of the proposed port and salinity differences over the tide can be neglected. Furthermore, it is shown that the differences in salinity between the different locations are negligible as well. From October 2000 till May 2001, the salinity increases and in June the salinity decreases again. There is no data available from the monsoon period but it is expected that the salinity decrease further in this period due to the increase of fresh water discharge. The fully mixed condition at Hazira is in line with the findings of chapter 4, in which it is concluded that the whole Gulf of Khambhat is fully mixed during the year.

The accuracy of the measured data needs to be discussed here. Even if the estuary is not fully mixed at the observed locations, the salinity on the bed of the estuary could never be less than the salinity close to the surface. Analysing the data of June 2000 at location S1 yields that there is an error in the measurements of the Sea-Bird MicroCat of at least 0.5 parts per thousand. At that location, the Microcat measured a higher salinity at the surface than at the seabed.

Water samples

Every month, water samples have been collected from different locations in the vicinity of Hazira. The collected water samples have been sent to England for analysis. They have been taken one metre below surface and one metre above seabed. Comparison with the measurements over the depth is possible because they are taken at the same moment. Salinity data according to the water quality analysis is summarised in appendix D2.

The data from October 2000 is not in accordance with the profiling measurements by the Sea-Bird MicroCat (see data October 2000 in appendix D1). It seems that temperature data is reflected. In general, the water quality analysis gives lower salinity values than the data obtained by measurements over the dept by the Sea-Bird MicroCat. The accuracy of the data has to be discussed again. In some months, the salinity at the surface is higher than just above seabed, which is not possible and is due to errors in the measuring method. The December 2000 data, at the location of the turning circle, show that the error in salinity is at least 0.4 parts per thousand (see appendix D2).

Conclusions concerning tidal differences in salinity

In this section, the salinity results at 0.3-0.5 metres above seabed are combined with the data measured over the depth. Both data is obtained by using the Sea-Bird MicroCat CT Recorder. The salinity data obtained at 0.3-0.5 metres above seabed is taken at location C3 and C4, whereas the data over the depth is measured at the location of the proposed turning circle, the LNG terminal and the bulk terminal.

Fugro Geos has concluded that there is a great tidal influence upon the salinity structure just above seabed. Fresh water reductions with maximum flood and ebb velocities have been observed. On the other hand, the depth profiles of salinity indicate a fully mixed estuary and do not show the presence of any fresh water at all. Combination of the data implicates that the estuary would suddenly become fresher over the full depth twice per tidal cycle. This is unrealistic.

In the monthly data reports, Fugro Geos observed sharp reductions in salinity at 0.3-0.5 metres above seabed coinciding with periods of maximum flood and ebb (see Figure 3.9). Figure 3.11

and Figure 3.12 show that this assumption does not hold. In general, the increments are observed at, or just after, slack water. In these figures, the green and red coloured current profiles are the observed and predicted speed respectively.

Comparing the fixed station measurements just above seabed with the profiling measurements over the depth profile shows that the profiling salinity data corresponds with the maximum values of the salinity data at 0.3-0.5 metres above seabed (see Table 3.7, Figure 3.11 and Figure 3.12).

Table 3.7 Salinity data depth profiles

Month	Salinity [ppt]
October 2000	33.0-33.1
December 2000	34.2-34.4
February 2001	35.0-35.1
April 2001	35.5-35.6
May 2001	36.0-36.2

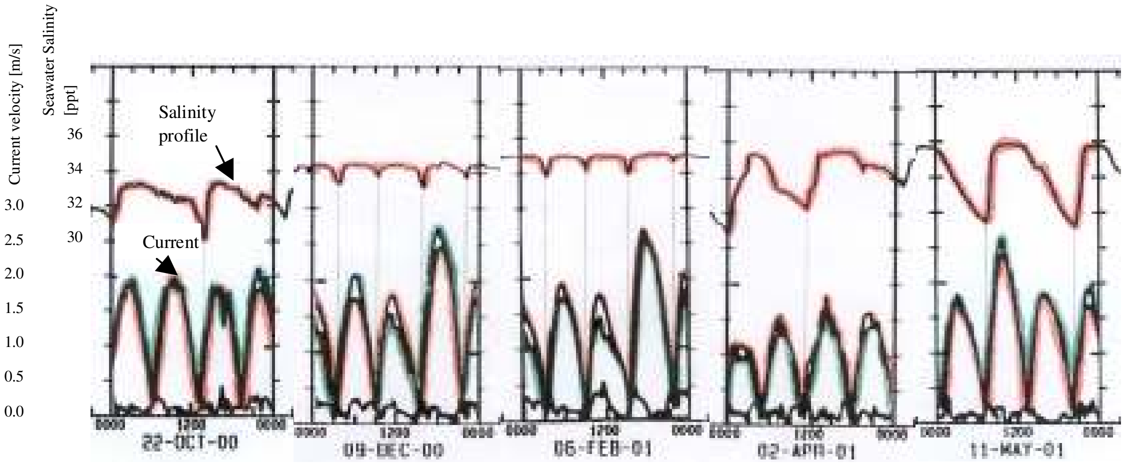


Figure 3.11 Relation between sea water salinity and current velocity, location C3

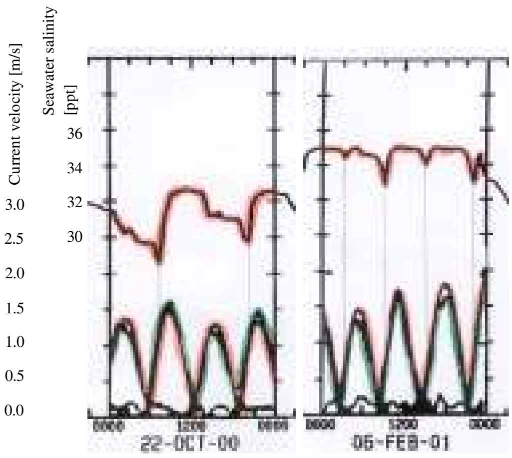


Figure 3.12 Relation between sea water salinity and current velocity, location C4

As a result, it is assumed that the fresh water reductions are not real. By neglecting the reductions in the monthly data reports, there are no differences in salinity over the tide. In January 2001, the Anderaa WRL-7 Tide Gauge has measured salinity data. The data corresponds with the former assumption; timeseries of waterlevel, temperature and salinity show a constant salinity profile with an average of about 34.5 parts per thousand.

After inquiry, Fugro Geos has confirmed the problem with the measured salinity data by the MicroCat CT recorder just above seabed. After overlaying plotted salinity data from the Valeport 730D directional Wave Recorder and from turbidity measurements, Fugro has concluded that the fresh water peaks should indeed be neglected.

Reasons for the unrealistic data are:

- the horizontal set up of the CT recorder (see Figure 3.8)
- the installation of the recorder too close to the seabed

High sediment content and insufficient flushing are probably contaminating the recorder. After increase of current velocity, the recorder is rinsed out again. The high sediment content during slack water decreases the turbidity, which results in a decrease of salinity as well.

Hence, the main conclusion of this section can be drawn:

In spite of former assumptions, there are no salinity differences over the tide.

The salinity over the full year can now be derived from the monthly data reports by neglecting sharp reductions in salinity. This results in a salinity pattern as shown in Figure 3.13.

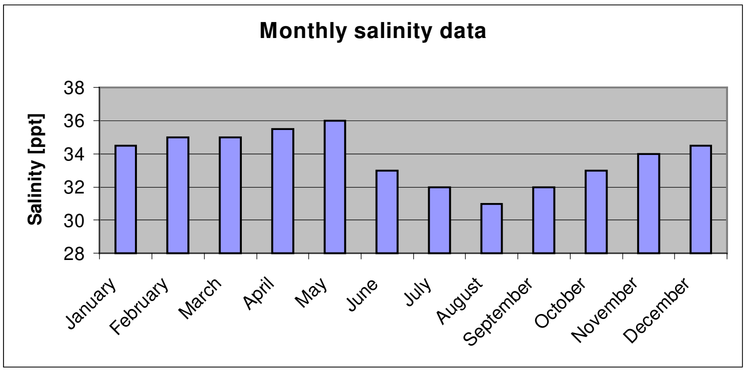


Figure 3.13 Assumed salinity profile

As follows from Figure 3.13, the salinity decreases in the monsoon period due to the increased influence of the fresh water discharges. After this period the salinity increases again up to the maximum value of 36 parts per thousand in May. In May, the river discharges are minimal.

3.4 Morphological development

In this section the morphological processes in the planned port area are discussed based on a study performed by Svašek. The impact of the port has to be seen in relation to the autonomous development of the coastal area around Hazira without any port.

3.4.1 Autonomous development

The morphological development of the area surrounding Hazira is mainly tide-dominated and the sandbanks in front of the coast follow the current direction. Erosion and deposition of sediments take place with increasing and decreasing transport capacity of the water respectively. At morphological equilibrium, the ratio between the tidal volume and the cross sectional area of each of the tidal channels is constant.

Along the coastline, the tidal influence diminishes and wave driven transport becomes dominant. This wave action disturbs the equilibrium between the tidal volume and the cross sectional area of the tidal channels. In areas that are sheltered for currents and waves, sediment deposition is expected.

From an analysis of historical bathymetric data, satellite images and model simulations, it follows that the coastline appears to be nearly stable. To the South of the proposed harbour, the foreshore is expected to be slightly erosive. Probably, the counteracting forces of the breaking waves and the tide can explain this relatively stable coastline. The main wave direction is from the Southwest, which results in a Northerly sediment transport due to breaking waves. The tidal flow along the coastline, on the other hand, is ebb dominated and is directed to the South.

The influence of the sediments that originate from the Tapi River is expected to be limited. Probably, the strong flood current in the Sutherland Channel carries the fine sediments away without affecting the coast.

3.4.2 Anticipated impact of the harbour

The morphological development of the coastal area surrounding Hazira is determined by changes in the current and wave pattern that results from the construction of the proposed harbour. Especially, changes in wave driven transport by the harbour is an important feature as this transport mechanism is dominant in the shallow foreshore.

To the North of the proposed harbour, some accretion of fine-grained sediments is expected due to the interruption of the wave driven transport. This may result in filling up of creeks. Further to the North, wave action increases again and probably causes some erosion. It is hypothesised that the planned port has no influence on the meandering processes of the Tena Creek. These processes may also cause some erosion or accretion.

The dam to the land based drilling platform influences the morphological processes South of the planned harbour. In this area, the tidal influence is small and accumulation of suspended sediments is expected. This may lead to an extension of the mangrove area. Mangroves offer a sheltered zone for accumulation of sediments due to the dense network of trunks and roots. However, this section remains open for waves from Southwesterly directions. In the long run, this probably leads to a new equilibrium shoreline, further into the sea, directly South of the planned harbour.

The creeks and the Tapi River tend to follow their own meandering patterns. Therefore, it is hypothesised that the dam and the Hazira harbour do not influence them.

The foregoing conclusions are presented in appendix E.

4 Salinity distribution

4.1 Introduction

The salinity distribution in an estuary may be of great influence on the sedimentation processes. A common classification of estuaries by salinity structure leads to three different types of estuaries. Section 4.2 describes these different mixing regimes. In section 4.3 the salinity distribution in the Gulf of Khambhat is discussed. Use is made of the estuary number, which is also described in this section.

4.2 Characteristics of estuaries

4.2.1 Definition of an estuary

Estuaries are turbulent bays where fresh water from rivers meets salt water from the sea. Estuaries are semi-enclosed coastal bodies of water that are connected to the open sea and within which seawater is measurably diluted.

Often, estuaries are basins of non-marine origin that have been invaded by the sea. Complex processes concerning sediment and water movements may arise due to the interaction of two chemically and physically different water masses. An estuary can be divided into three different parts; the inland end where the river enters is called the head, the middle part is the fully estuarine area and the seaward end is called the mouth.

Tidal currents and wave action at the seaward end and river flow at the head of the estuary affect estuaries. These physical processes are closely interrelated with the sediment movement and are located differently in the estuary. See Figure 4.1 and Figure 4.2.

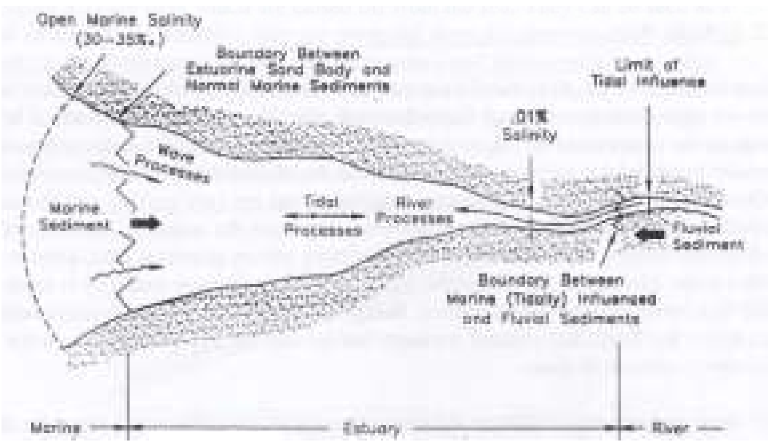


Figure 4.1 Physical processes in estuaries

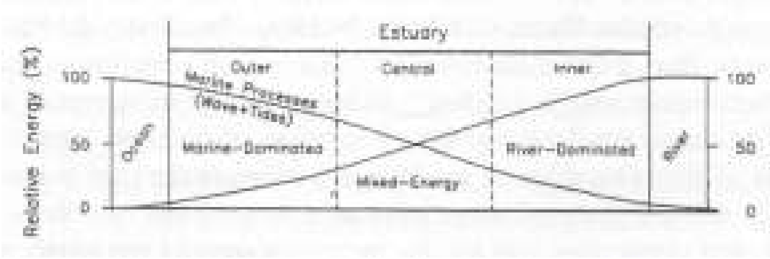


Figure 4.2 Schematic overview of the physical processes

The Gulf of Khambhat is an estuary with a wide mouth and a narrow head. This geometry produces a large tidal range. A given amount of water is transported from the sea to the narrower part of the estuary, resulting in an increase in the high tide level. At the head of the Gulf of Khambhat, the tidal range at maximum spring tide amounts to 9 metres.

4.2.2 Salinity classification

The salinity, i.e. the amount of totally dissolved salts, of fresh and salt water differs essentially. Salt water has a salinity of 30-35 parts per thousand, whereas the salinity of fully fresh water is zero. This leads to density gradients between fresh and salt water. The effect of density gradients depends on the degree of mixing that occurs. A common classification by salinity structure leads to three different types of estuaries: a stratified, a partly mixed and a well-mixed estuary. See Figure 4.3.

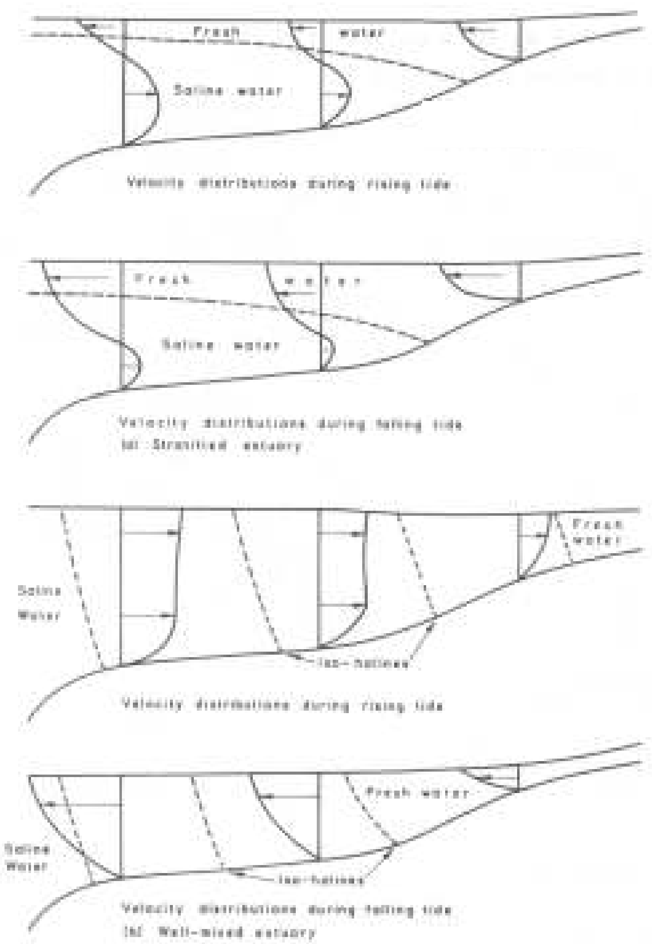


Figure 4.3 Iso-halines and velocity profiles for a typical stratified and well-mixed estuary

In a stratified estuary no significant amount of vertical mixing occurs. Stratified flow occurs in estuaries with a weak tidal action. The fresh water layer, which has a lower density, is on top of the salty layer and flows towards the mouth of the estuary. Fine sediments may be carried landward to a point of zero net movement. With high river discharges this point moves seawards and conversely, it moves landward with low river discharges.

In a partially mixed estuary, tidal currents are relatively strong in comparison to those in stratified estuaries. Tidal currents are the dominant mixing factors in a partially mixed estuary. Mixing takes place at the interface between the upper fresh water layer and the lower salt water layer which leads to brackish water at the interface. In a vertical water column, the salinities are usually the highest near the seabed and lowest at the surface. The iso-halines slope down from the seaward side to the river mouth of the partially mixed estuary.

If the mixing by waves and/or tidal currents is very intense, an almost homogeneous salinity profile over the depth occurs. In the horizontal plane, the salinity varies from pure fresh water close to the mouth of the river to undiluted salt water at the seaside of the estuary.

The distribution of salinities may be a very dynamic process. It can vary in a short period like a single tidal cycle. An estuary may also experience seasonal shifts in circulation patterns.

4.3 Mixing in the Gulf of Khambhat

After description of the three different types of estuaries in the preceding section, the salinity distribution in the Gulf of Khambhat is determined. Therefore, use is made of the estuary number, which has been developed by Harleman and Abraham (1966).

Section 4.3.1 deals with a general description of this estuary number. Subsequently, the estuary number of the Gulf of Khambhat is determined in section 4.3.2. Finally, the limitations of this criterion are discussed in section 4.3.3.

4.3.1 General

Several attempts have been made to classify estuaries according to their degree of mixing. Ippen and Harleman (1961) have suggested the use of a stratification number, which is based on salinity intrusion parameters. The derivation of this parameter is described in appendix F. As pointed out in this appendix, for a unit mass of fluid the stratification number (S) is defined as:

$$S = \frac{G}{J} = \frac{\text{rate of dissipated energy per unit mass of fluid}}{\text{rate of potential energy gain per unit mass of fluid}} \tag{4.1}$$

This number is based on the conservation of mass equation for salt. The numerator reflects the tidal energy dissipation in the estuary. The denominator represents the gain in potential energy, as water becomes more saline moving downstream to the ocean. This stratification number can indicate the degree of mixing; a well-mixed estuary has a stratification number S ≥ 100.

The stratification number is a useful parameter in flume tests, but in real estuaries the rate of energy dissipation from tidal data is difficult to evaluate. For this reason, the state of mixing is often defined as the flood number. This number reflects the ratio of the fresh water discharge in a tidal cycle to the tidal prism. The tidal prism can be defined in two ways:

1. the change in volume of water in the estuary between low and high water
2. the volume of water entering the estuary during a flood period

The first definition may not represent the actual volume of water in the estuary at any time for two reasons. Firstly, the volume of fresh water entering the estuary from the landside is included in this definition. Secondly, the first definition does not take into account the time lag between the high and low water stages along the estuary, as they often occur at the same time in long estuaries. Therefore, the second definition is used in this section. The limitation of this definition is that the water entering the estuary during the flood cycle does not contain the full ocean salinity but is expected to have a somewhat lower salinity. Assuming a linear harmonic relationship for the tidal velocity, the tidal prism P_t can be defined as:

$$P_t = \frac{2}{\pi} u_0 A \frac{T}{2} \qquad (4.2)$$

where $\frac{2}{\pi} u_0$ represents the mean velocity over the flood period at the estuary entrance and A represents the entrance cross section of the estuary.

To check the validity of the flood number, Ippen and Harleman (1966) compared the flood number with the stratification number for different flume tests carried out by the Waterways Experiment Station, U.S. Army Corps of Engineers, Vicksburg, Mississippi (1961). See Figure 4.4.

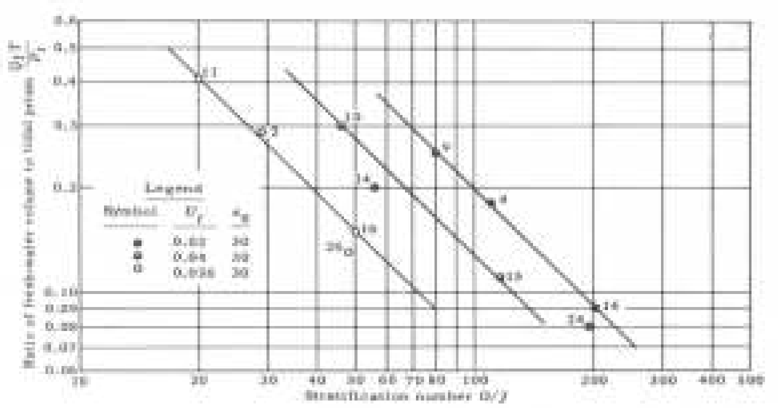


Figure 4.4 Relation between stratification number and the ratio of the fresh water volume to the tidal prism

From Figure 4.4, it follows that different values of the stratification number are obtained for the same value of the flood number. Therefore, the ratio of the fresh water discharge to the tidal prism does not provide sufficient insight with respect to the degree of mixing.

During reanalysis of the flume data of the Waterways Experiment Station, Harleman and Abraham (1966) found out that it was possible to relate a dimensionless parameter to the stratification number. This parameter is called the estuary number and is relatively easy to evaluate in real estuaries.

The estuary number E is defined by:

$$E = \frac{P_t F_0^2}{Q_f T} \tag{4.3}$$

where:

P_t = the volume of the tidal prism [m^3]

F_0 = Froude number, $\frac{u_0}{\sqrt{gh}}$, u_0 is the maximum flood current at the estuary mouth and h is the mean depth at the estuary mouth [-]

Q_f = fresh water discharge [m^3/s]

T = tidal period [s]

The estuary number is proportional to u^3. This makes sense because the tidal energy, on which the stratification number is based, is proportional to u^3 as well.

Figure 4.5 shows a linear relation in the logarithmic plot between the estuary number and the stratification number. This figure proves the validity of the estuary number as a parameter of the relative state of mixing.

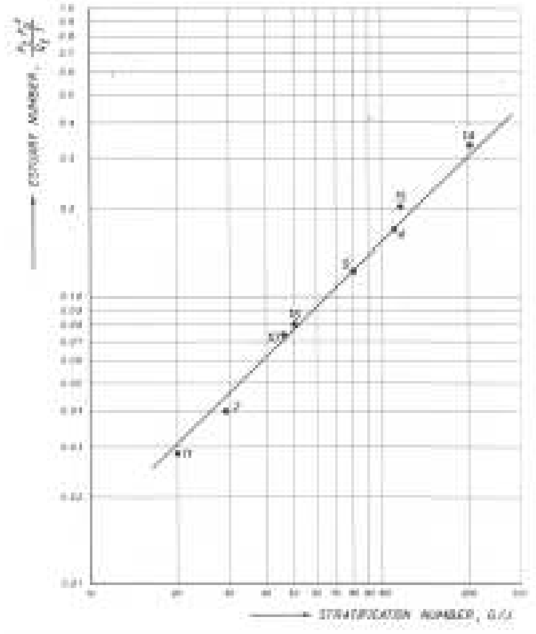


Figure 4.5 Relation between stratification number and estuary number

4.3.2 Estuary number of the Gulf of Khambhat

This section determines the estuary number of the Gulf of Khambhat. The numerator of equation (4.3) reflects the influence of the tide, while the denominator deals with the influence of the fresh water discharges. The tests carried out in the tidal flume are grouped into three classes:

- *Stratified flow*; the volume of fresh water flowing out to the sea during a tidal period is of the same order of magnitude as the tidal prism. There is a sharp interface between the salt water and the fresh water layer. The following applies to this situation:

$$E \leq 0.005 \tag{4.4}$$

- *Partially mixed flow*; there is no well-defined interface between the fresh and salt-water layer.

$$0.005 < E < 0.2 \tag{4.5}$$

- *Mixed flow*; the volume of fresh water flowing out to sea during a tidal period is very small compared to the tidal prism. There is no interface between the fresh and salt-water layer.

$$E \geq 0.2 \tag{4.6}$$

As stated by equation (4.3), the tidal prism, the Froude number, the fresh water discharge and the tidal period determine the estuary number. In this section, these parameters are discussed for the case of the Gulf of Khambhat.

Tidal Prism

The tidal prism is defined as the volume entering the mouth of the estuary during rising tide. See equation (4.2). As a result, the maximum flood current at the estuary mouth and the cross sectional entrance area of the estuary need to be determined.

As stated in chapter 3, the Gulf of Khambhat can be characterised as a very large cone-shaped estuary with a length of 210 kilometres and a width of 200 kilometres at the mouth. Ships enter the estuary between Diu and Umargam (see Figure 4.6). As stated in the West Coast of India Pilot (1975, revised 1986), the tidal streams are North during flood and South during ebb, except where the direction is altered by sandbanks. The flood currents entering the Gulf at Umargam attain a rate of 0.8 to 1.0 m/s. The flood stream South of Diu is directed to the East with a maximum of 0.5 to 0.8 m/s. Subsequently, the flood stream continues in Northeastern direction following the coastline.

The flow pattern of the Gulf during flood is shown in Figure 4.6. The mean maximum flood currents at Diu and Umargam are 0.6 and 0.9 m/s respectively. The mean maximum flood velocity, averaged across the estuary mouth, equals about 0.8 m/s.

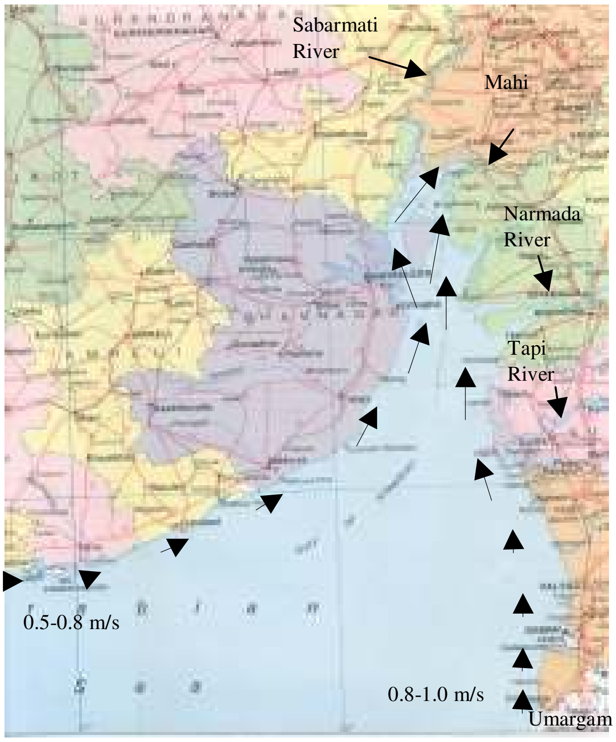


Figure 4.6 Flood currents in the Gulf of Khambhat

The mean velocity over the flood period at the estuary entrance yields (see equation (4.2)):

$$\bar{u} = \frac{2}{\pi} u_0 = 0.5 m/s \tag{4.7}$$

The Gulf of Khambhat is deep in its Southern part but much shallower in its Northern part. In the Northern part, sandbanks shift under influence of tidal bores and floods.

To compute the tidal prism, the estuary entrance is schematised to a rectangular cross section with a width of 200 kilometres and an average depth of 26 metres relative to MSL. The entrance area of the estuary is approximately $200*10^3$ m*26 m=$5.2*10^6$ m^2. See Figure 4.7.

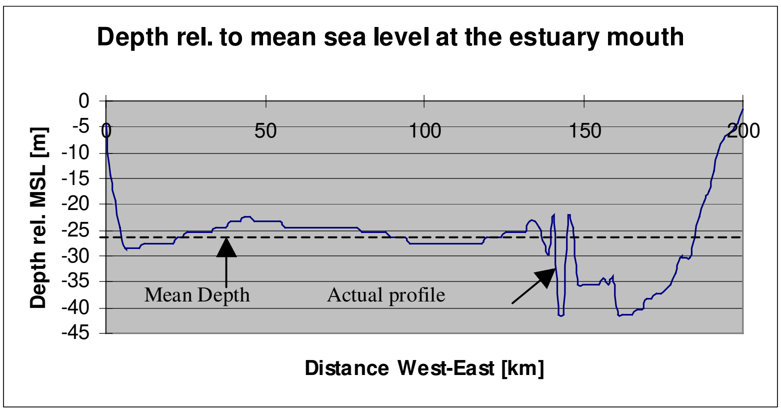


Figure 4.7 Depth profile at the estuary mouth

This profile is based on the Admiralty Chart 1486 "Approaches to the Gulf of Khambhat". Every 1.5 kilometres, the depth profile has been determined. Up to 130 kilometres from the West side of the estuary, the actual depth profile is quite constant. Subsequently, there are some irregularities and the cross section of the estuary mouth becomes deeper in its Western part.

The tide has a semi-diurnal character. This results in a tidal period of 12.41 hours. The tidal prism in the Gulf of Khambhat can now be estimated, as all the unknown parameters of equation (4.3) are known. See equation (4.8).

$$P_t = \frac{2}{\pi} u_0 A \frac{T}{2} \approx 5.7 * 10^{10}\ m^3 \tag{4.8}$$

Froude number

The Froude number at the estuary mouth reflects the ratio of the maximum flood current to the wave celerity. It amounts to 0.05. See equation (4.9).

$$F_0 = \frac{u_0}{\sqrt{gh}} = \frac{0.8}{\sqrt{9.81 * 26}} = 0.05 \tag{4.9}$$

Fresh water discharge

Chapter 3 describes the discharges of the four main rivers of the Gulf of Khambhat. The Tapi, the Narmada, the Mahi and the Sabarmati River all show the same seasonal pattern. During the monsoon period the river discharges are very large. After this season, the fresh water discharges decrease rapidly and reach their minimum in May.

Estuary Number

Since the tidal prism, the Froude number, the river discharges and the tidal period are known, the estuary number also can be determined (see equation (4.3)). Due to great differences in the monthly fresh water discharges, the estuary number of the Gulf of Khambhat shows considerable variation during the year as well. See Table 4.1.

Table 4.1 Estuary number of the Gulf of Khambhat

Month	Estuary number
January	12,8
February	16,0
March	17,0
April	21,9
May	29,1
June	3,7
July	0,7
August	0,3
September	0,4
October	1,8
November	6,0
December	8,1

From Table 4.1, it can be concluded that the Gulf of Khambhat is well mixed during the whole year. In all months, the estuary number is greater than 0.2.

However, it has to be mentioned that the non-rectangular geometry of the estuary clearly affects the velocity of the approach mentioned above. The next section deals with the limitations of the Harleman and Abraham criterion.

4.3.3 Limitations of the one-dimensional model

The one-dimensional criterion by Harleman and Abraham is a stationary criterion, based on a state of equilibrium in the estuary. This means that it is applicable for characteristic conditions around which variation is possible. In reality, river outflow and tidal conditions alter constantly.

Comparison of the one-dimensional criterion with tidal flume studies carried out by Delft Hydraulics, yields that such a model is only valid for partially and well-mixed estuaries. This means that the estuary number has to be larger than 0.005-0.02.

The estuary number is obtained from tidal flume experiments with a depth of approximately 15 centimetres and a width of 23 centimetres. The Gulf of Khambhat however, has a depth of 26 metres and a width of 200 kilometres at the estuary mouth. To prove that the estuary number can be applied to the Gulf of Khambhat, the correlation between the two basic salinity intrusion parameters and the estuary number is analysed. The salinity intrusion parameters (the dispersion coefficient D_0 and the characteristic length B) specify the boundary conditions at the estuary mouth as well as the shape of the longitudinal salinity distribution within the estuary. The derivation of these parameters is given in appendix F. The relationship with the estuary number can be given by (Harleman and Abraham, 1966):

$$\frac{D_0}{V_f B} = 0.055 * \left(\frac{h}{a}\right)^{2.7} * E^{1.2} \tag{4.10}$$

where D_0 is the dispersion coefficient, B a characteristic length and a the tidal amplitude.

The effect of changes in the estuary geometry such as its depth and tidal prism and/or changes in the fresh water flow rate and the tidal conditions at the mouth may be predicted from this correlation.

In Figure 4.8 the relationship between the salinity distribution parameters and the estuary number multiplied by the depth to tidal amplitude ratio (h/a) is given.

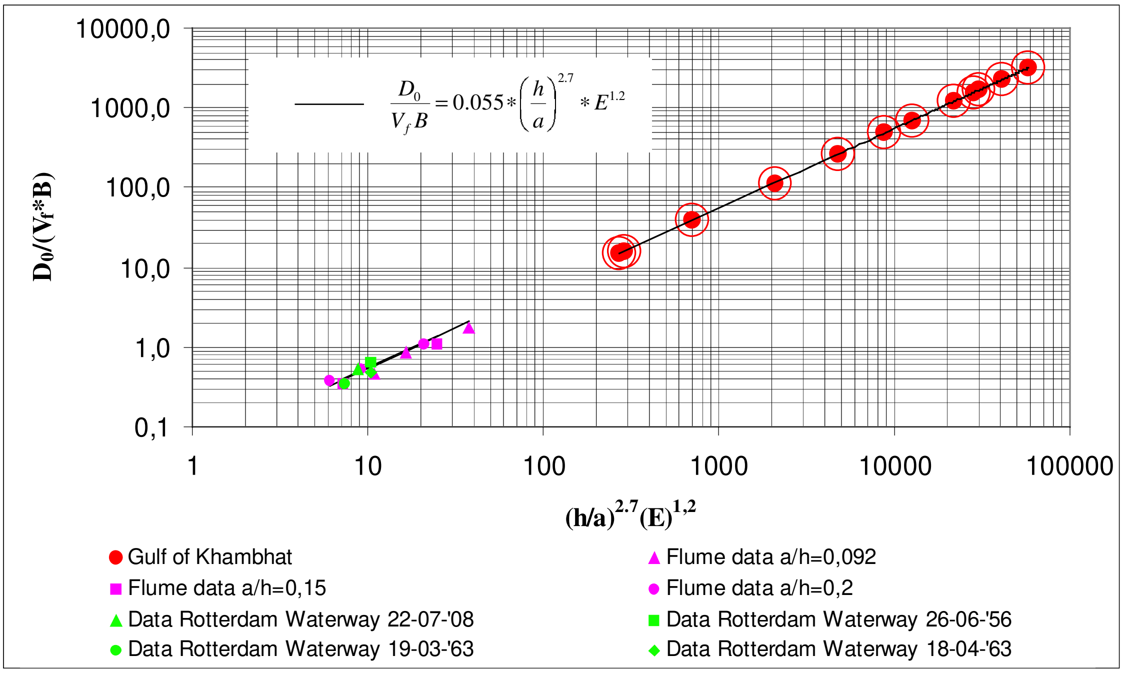


Figure 4.8 Correlation salinity distribution parameters to the estuary number and the depth to tidal amplitude ratio

In this figure the Rotterdam Waterway data is compared with the tidal flume data. These field and flume data show the same slope. Although the depth and width of the Rotterdam Waterway are respectively 100 and 3000 times as large as the tidal flume dimensions, the Rotterdam Waterway data matches well with equation (4.10).

Assuming the same relationship for the Gulf of Khambhat, requires extrapolation because the value of $\left(\frac{h}{a}\right)^{2.7} * E^{1.2}$ of the Gulf of Khambhat is not within the range of the data covered by the flume experiments. The great estuary numbers in the Gulf of Khambhat cause this. Nevertheless, it is expected that the estuary number can also be applied to the Gulf of Khambhat, because when the step from the tidal flume to the Rotterdam Waterway can be justified, the step from the Rotterdam Waterway to the Gulf of Khambhat can be considered justifiable as well.

A very important limitation of the Abraham and Harleman criterion (1966) is that it is only valid for an estuary without subsidiary arms, which can be approximated by a constant cross sectional area. This does not hold for the Gulf of Khambhat; the cross section varies in the longitudinal direction (see Figure 4.9). The next section deals with this limitation.

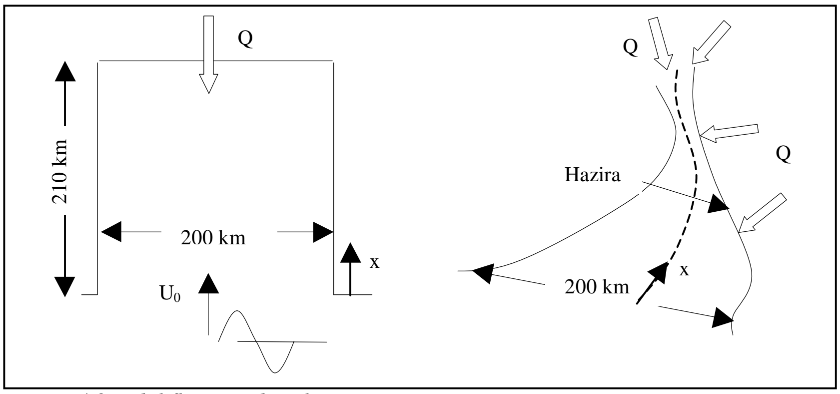


Figure 4.9 Tidal flume and real situation

Estuary numbers in the longitudinal direction

The criterion assumes a rectangular tidal flume. On the end of the flume, there are provisions for a fresh water discharge, while the beginning of the flume is connected to a large tidal basin. It is expected that, because of its cone shaped geometry, the real mixing in the Gulf of Khambhat is greater than the mixing calculated with the one-dimensional criterion.

The estuary number is proportional to (see equation (4.3)):

$$E \propto \frac{\frac{1}{T}\int^{T}\int_{Ax} \frac{u_x^3}{gh_x}\, dA\, dt}{Q_f} \qquad (4.11)$$

in which u_x is the maximum flood velocity at distance x from the estuary mouth and A_x is the cross sectional area at distance x from the estuary mouth.

In a rectangular tidal flume, the estuary number decreases in longitudinal direction, hence the maximum flood velocity decreases and all the other parameters remain constant. In the Gulf of Khambhat however, the maximum flood velocity is expected to increase and the cross sectional area decreases in longitudinal direction. The variation of the estuary number in longitudinal direction is discussed in this section, as E is the integral parameter. For six different cross sectional areas the estuary numbers are determined (see Figure 4.10 and Table 4.2). At the head of the Gulf of Khambhat the current is Northeast with the flood into the Mahi Sagar River.

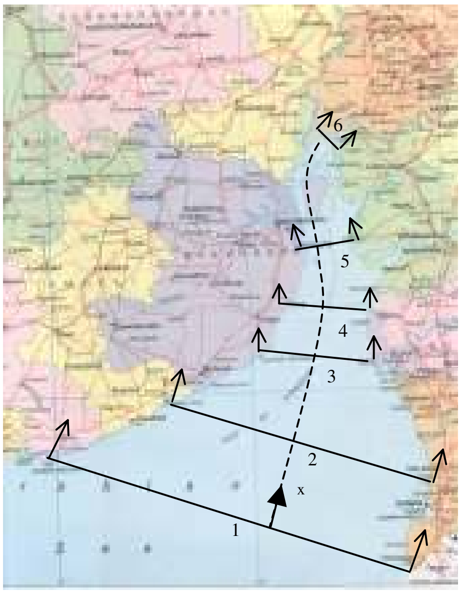


Figure 4.10 Cross sections

Table 4.2 Characteristics of the cross sections

Cross section	x co-ordinate	Width [km]
1 Diu - Umargam	0 km	200
2 Pipavav - Valsad	45 km	150
3 Methla - Hazira	90 km	66
4 Alang - Mor	120 km	50
5 Ghogha - Dahej	150 km	27
6 Mouth Mahi River	210 km	6

As pointed out in equation (4.11), the mean depth, the cross sectional area, and the maximum flood velocity alter in the longitudinal x-direction. The fresh water discharge is kept constant. In the criterion, one fresh water discharge at the end of the estuary is assumed. In the Gulf of Khambhat however, four large rivers discharge into the estuary at different locations.

The depth profiles of cross sections 2, 3, 4 and 5 are given in Figure 4.11 (Admiralty Chart 1486). All cross sections show a considerable variation in depth along their width. The mean depth changes considerably; from 26 metres relative to mean sea level in the estuary mouth to 12 metres relative to mean sea level at 150 kilometres from the mouth.

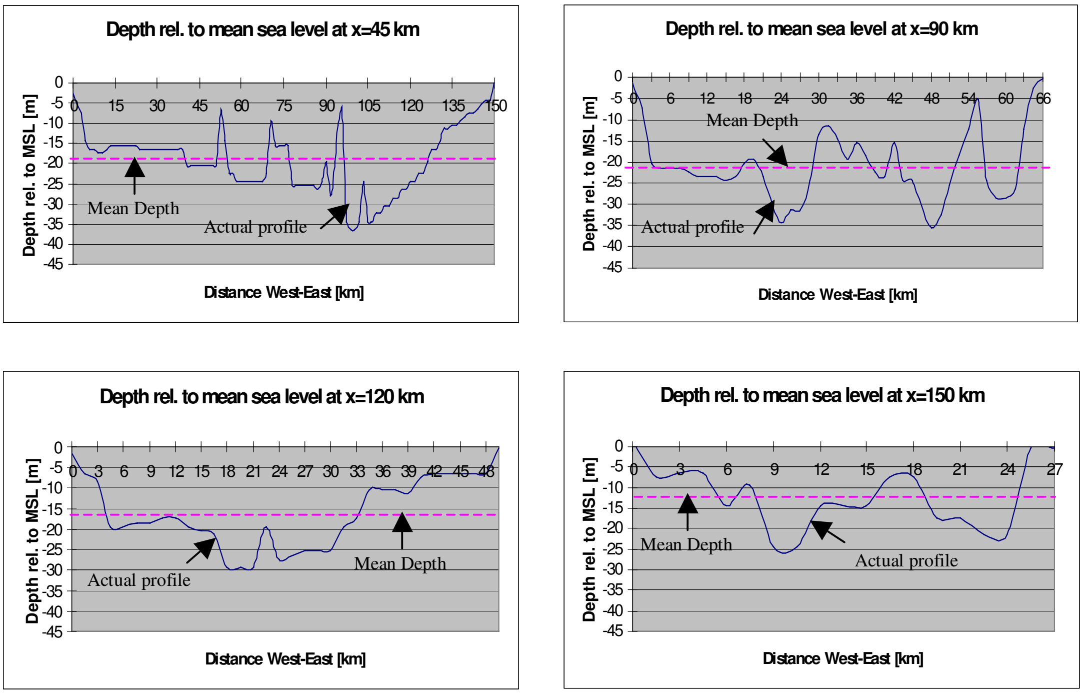


Figure 4.11 Depth profiles of cross sections 2,3,4 and 5

The mean depth of the mouth of the Mahi River is assumed to be 5 metres relative to mean sea level. This estimation is somewhat rough, due to the limited data provided by the West Coast of India Pilot and the Admiralty Chart. Beside, the charted depths are based on a very old survey and cannot be fully relied upon.

The cross sectional area is determined by multiplying the mean depth by the width of the cross section concerned. The results are shown in Table 4.3. The cross sectional area decreases quickly due to the cone shaped geometry of the estuary.

Table 4.3 Cross sectional areas

Cross section	x co-ordinate	Mean depth [m]	Width [km]	Cross sectional area [m^2]
1 Diu - Umargam	0 km	MSL-26	200	$5.20*10^6$
2 Pipavav - Valsad	45 km	MSL-19	150	$2.85*10^6$
3 Methla - Hazira	90 km	MSL-21	66	$1.26*10^6$
4 Alang - Mor	120 km	MSL-16	50	$0.80*10^6$
5 Ghogha - Dahej	150 km	MSL-12	27	$0.32*10^6$
6 Mouth Mahi River	210 km	MSL-5	6	$0.03*10^6$

With the help of the mathematical program DELFT 3D, WL | Delft Hydraulics has modelled the upper part of the Gulf of Khambhat. The flow patterns with maximum velocities during a spring and a neap tide are given in appendix G. Table 4.4 summarises the calculated maximum flood velocities for the different cross sectional areas.

Table 4.4 Maximum flood velocities

Cross section	Maximum flood velocity [m/s]
1 Diu - Umargam	0.8
2 Pipavav - Valsad	1.0
3 Methla - Hazira	1.3
4 Alang - Mor	1.4
5 Ghogha - Dahej	1.8
6 Mouth Mahi River	1.5

Now all the parameters are determined, the estuary numbers for the different cross sections can be calculated. A distinction has been made between the non-monsoon and the monsoon period. The accompanying mean river discharges are 414 m^3/s and 5533 m^3/s respectively. The estuary numbers for the different cross sections at distance x from the estuary mouth are given in Table 4.5 and Figure 4.12.

Table 4.5 Estuary numbers in longitudinal direction

Cross section	x co-ordinate	Estuary number non-monsoon	Estuary number monsoon
1 Diu - Umargam	0 km	8,0	0,6
2 Pipavav - Valsad	45 km	11,8	0,9
3 Methla - Hazira	90 km	10,3	0,8
4 Alang - Mor	120 km	10,8	0,8
5 Ghogha - Dahej	150 km	12,2	0,9
6 Mouth Mahi River	210 km	1,6	0,1

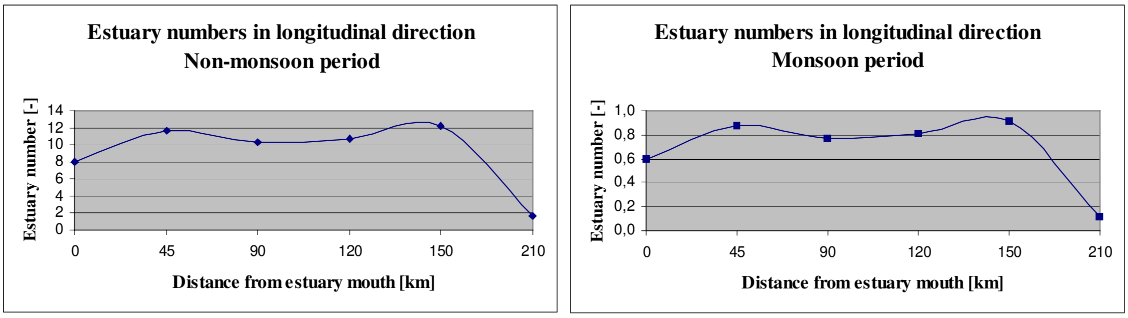


Figure 4.12 Estuary numbers in longitudinal direction

As stated before, in a rectangular tidal flume the estuary number decreases in longitudinal direction. In the Gulf of Khambhat however, the estuary number first increases then becomes relatively constant and finally decreases. The estuary number reaches its initial value at a distance of about 175 kilometres from the mouth.

From the preceding, it can be concluded that the original tidal flume with a width of 200 kilometres and a constant cross section represents the normative situation. For a fully mixed situation, the estuary number has to be greater than 0.2. The Gulf of Khambhat is fully mixed over its entire length except near the entrance of the Mahi River during the monsoon period.

5 Total suspended solids

5.1 Introduction

The siltation rate inside the planned Hazira harbour is proportional to the sediment influx per tide. This influx is determined by the sediment concentration of the water entering the basin and the water exchange volume. In this chapter, the sediment content of the penetrating water is discussed while the next chapter deals with the water exchange mechanisms.

Section 5.2 gives a description of the cyclic transport processes that play a role in environments like at Hazira. Subsequently, the data that has been collected by Fugro Geos in the vicinity of the planned port is described in section 5.3. The theory of Section 5.2 and the measurements of section 5.3 are combined with each other in section 5.4. This section focuses on tidal variations as well as seasonal variations of the observed sediment concentrations. Finally, in section 5.5 conclusions concerning the expected amount of total suspended solids are given.

5.2 Water-bed exchange

The Environment Agency, an English laboratory, has analysed water quality samples for their grain sizes in the months February, April and May 2001. The accompanying suspended sediment sieve curves are presented in appendix H. The graphs indicate that there is a relatively large amount of fine-grained sediment particles recognisable. It is difficult to draw conclusions concerning the grain size diameters from these curves, because transport to England can affect the grain size distribution. Furthermore, to obtain the single particle size diameters, deflocculation may be required.

Van Rijn (1993) states that “in a natural environment there is a continuous transport cycle of mud material, consisting of: erosion, settling, deposition, erosion and so on”. Mud is herein defined as “ a fluid-sediment mixture consisting of (salt) water, sand, silts, clay and organic materials”. See Figure 5.1.

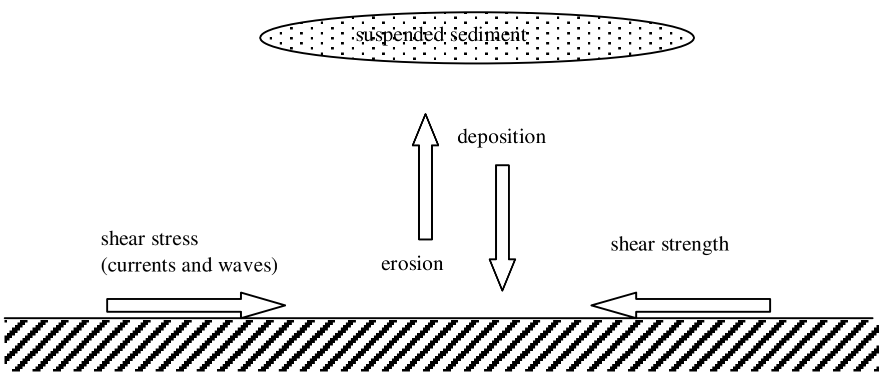


Figure 5.1 Deposition and erosion processes

5.2.1 Deposition

Krone (1962) has done laboratory experiments on deposition processes of cohesive materials. He stated that deposition is predominant when the bed shear stress (τ_b) is smaller than a critical value for deposition (τ_d).

Metha and Partheniades (1975) have described two different critical bed shear stresses for deposition, i.e. the minimum bed shear stress for full deposition ($\tau_{d,full}$) and the maximum bed shear stress for deposition ($\tau_{d,part}$).

Based on the experiments of Metha and Partheniades, the rate for deposition (D) for sediment concentrations from 0.3 to 10 kg/m^3 can be expressed as:

$$D = c\alpha w_s \qquad \text{for } \tau_b < \tau_{d,full} \qquad (5.1)$$

$$D = (c - c_{eq})\alpha w_s \qquad \text{for } \tau_{d,full} < \tau_b < \tau_{d,part} \qquad (5.2)$$

$$D = 0 \qquad \text{for } \tau_b > \tau_{d,part} \qquad (5.3)$$

where c is the sediment concentration, c_{eq} the equilibrium concentration, w_s the particle fall velocity in still water and α is a coefficient ≤ 1 (α=1 for τ_b=0 N/m^2 and α<1 for τ_b>0 N/m^2).

It generally holds that the critical shear stress for sedimentation ranges from 0.03 to 0.15 N/m^2 and from 0.15 to 1.7 N/m^2 for the full and partly deposition shear stresses respectively. Deposition is maximal around slack water.

5.2.2 Erosion

Erosion occurs when the bed shear stress exceeds a critical shear stress for erosion τ_e. This critical value depends on the characteristics of the bed material and on the deposition and consolidation history of the bed.

The critical shear stress for erosion is generally greater than the bed shear stress for full deposition. For τ_b to be larger than $\tau_{d,part}$ depends, among others, on the stage within the tidal cycle. Near slack tide, the tidal velocities decrease and the fine-grained sediment particles accumulate on larger and stronger particles that have been deposited at an earlier stage at higher velocities. This soft upper layer of the bed can be eroded rather easily with increasing velocities at the next tidal cycle.

Analysis of bed samples from flume and field experiments clearly demonstrate that the critical shear stress for erosion depends on the dry sediment density and increases with the depth of the bed layer. Flume experiments have shown that for a certain bed shear stress, an equilibrium concentration is established after some time. For such an equilibrium situation applies that τ_e = τ_b.

Patheniadis (1965) describes the erosion speed E as:

$$E = M\left(\frac{\tau_b - \tau_e}{\tau_e}\right) \qquad \text{for } \tau_b > \tau_e \tag{5.4}$$

M is a re-suspension coefficient, which depends on the material characteristics.

According to values found in literature (Van Rijn, 1993), the critical shear stress for erosion varies from about 0.15 to 1.0 N/m^2.

5.2.3 Bed shear stress

A combined influence of waves and currents occurs in the vicinity of Hazira. Waves predominantly influence the sediment concentration by stirring up the material, while the currents take care of the transport of the sediment. At shoals the wave influence dominates, while in the deep channels, like the entrance channel, the current related bed shear stress is the main factor. Bijker (1971) combined the separate transport mechanisms for currents and waves.

The bed shear stress with a current alone has been related to the velocity near the seabed and can be defined by:

$$\tau_{b,c} = \rho\kappa^2 u_t^2 \tag{5.5}$$

where:

$\tau_{b,c}$	=	current related bed shear stress [N/m^2]
ρ	=	mass density of fluid [kg/m^3]
κ	=	von Karman coefficient (0.4) [-]
u_t	=	current velocity near the seabed [m/s]

Jonsson (1966) has found that the bed shear stress for waves alone can be described in terms of the near bed velocity amplitude and the wave friction factor according to the following relationship:

$$\tau_{b,w} = \rho\kappa^2 u_{wt}^2 \tag{5.6}$$

in which:

$$u_{wt} = p\,\hat{u}_{w0}(\sin\omega t)$$

where:

$\tau_{b,w}$	=	wave related bed shear stress [N/m^2]
p	=	proportionality factor (≈0.45) [-]
u_{wt}	=	velocity due to waves near the seabed [m/s]
$\hat{u}_{w0}$	=	maximum horizontal orbital velocity component just outside the boundary layer [m/s]

In a siltation study, in which the annual deposit rate is determined, it generally holds that use is made of the average wave velocities, as extreme conditions are not representative for a whole year.

Hence, the total bed shear stress combined for waves and currents is equal to:

$$\tau_{b,cw} = \rho \kappa^2 u_r^2 \tag{5.7}$$

where:

$\tau_{b,cw}$	=	bed shear stress for currents and waves [N/m^2]
u_r	=	resulting velocity near the seabed [m/s]

Bijker added the velocities of waves and currents (see Figure 5.2). The angle between the wave crest and the current direction is given by φ.

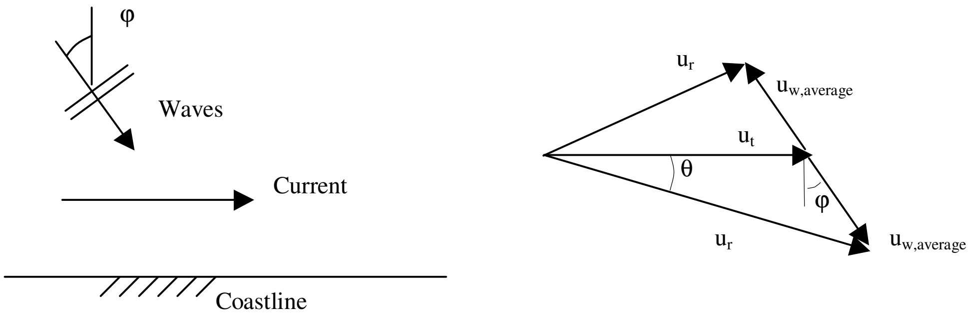


Figure 5.2 Velocity components just above the seabed

The main comment on the Bijker formula for the combination of currents and waves is the neglect of the effect of the wave boundary layer. Near the seabed, the current velocity is reduced due to wave interaction.

Regardless of the dominance of currents or waves, the exchange of particles between the water column and the seabed is proportional to:

$$Ex \propto \tau_b \propto u^2 \tag{5.8}$$

where Ex is the exchange of bed material and u is the velocity near the seabed. The greater the residual velocity, the greater the stirring up of bed particles.

5.3 Suspended sediment at Hazira

Fugro Geos has collected oceanographic and meteorological data for the Hazira harbour project from June 2000 until September 2001. Every month they have presented a report containing all the measured data. The amount of total suspended solids has been determined in two different ways:

- Relating turbidity to total suspended solids
- Analysis of water samples

The results of both measurements are discussed in section 5.3.1 and section 5.3.2 respectively. This is done using the information given in the monthly data reports provided by Fugro Geos.

5.3.1 Relating turbidity to total suspended solids

Fugro has performed fixed station measurements during the service visits of December 2000 and April 2001 by fitting turbidity sensors to current meters. Subsequently, turbidity has been related to the amount of total suspended solids using reference samples. The procedure of calibration of turbidity to total suspended solids is given in appendix I1.

Next to the fixed station measurements, which have been taken one metre below the water surface and one metre above the seabed, Fugro has also measured total suspended solids over the depth profile in April 2001. In this section the results are summarised.

Fixed station measurements

In December 2000, total suspended solids have been measured one metre below the water surface and one metre above the seabed at six different locations A to F, with a sampling interval of 10 minutes. The locations are shown in appendix B. At each location 25-hours measurements have been taken. The start and end dates of the measurements have been set a few days after neaps and springs respectively. In appendix I1, it is shown that the mean turbidity depends on the mean tidal range of the past 48 hours. In Table 5.1, the mean tidal range of the past 48 hours, for the different locations during the measuring period, is given.

Table 5.1 Mean tidal range in the past 48 hrs from 7 to 13 December for locations A to F (see appendix B)

Start Date (GMT)	Location	Mean tidal range past 48 hrs
7 December 00	A	3.3 m
8 December 00	F	4.1 m
10 December 00	C	5.3 m
11 December 00	B	5.8 m
12 December 00	D	6.2 m
13 December 00	E	6.4 m

As stated in appendix I1, the slope of the total suspended solids/turbidity curve increases as the mean tidal range of the past 48 hours increases. Given a certain turbidity, the suspended solids concentration increases as the mean tidal range in the past 48 hours increases.

According to the time series overlay plots, the suspended sediment concentration increases from December 7th to December 13th. There is no data available for the last few hours of location D near the seabed and for the whole 25 hours of location E near the seabed because water ingress caused the instruments to fail. The mean values for December 2000, for the locations A to F, of total suspended solids are about 1050 mg/l and 1150 mg/l near the surface and the seabed respectively.

In April 2001, Fugro Geos has measured the total suspended solids near the water surface and near the seabed for 7 days, beginning just before neaps and ending just before springs. Unlike the measurements in December 2000, this has been done for only one location, i.e. location T (see appendix B). A time series overlay plot of total suspended solids near the surface and near the seabed for this location is presented in appendix I2. The suspended sediment concentration is substantially lower in April compared to the measurements made in December. As stated by Fugro, the maximum and mean values of total suspended solids for April near the surface, are 975 mg/l and 427 mg/l respectively and near the seabed 1138 mg/l and 566 mg/l respectively.

A strong correlation between the suspended sediment concentration and the current profile is found. The variation of sediment concentration as a result of tidal flow in the proposed Hazira harbour area is discussed in section 5.4.

Profiling measurements

Next to the fixed station measurements, Fugro has also measured total suspended solids over the depth at location T. A current meter has sampled every two metres for three minutes with a time interval of one minute. These profiling measurements have taken place on 31 March 2001 and 8 April 2001 at hourly intervals, just before neaps and springs respectively. From these measurements can be concluded that the total suspended solids profile is relatively constant over the water column except at and just after slack water. See section 5.4.

5.3.2 Water samples

Monthly water samples have been collected from June 2000 until September 2001. Subsequently, the samples have been sent to England for analysis. This is done for different locations near the surface and/or near the seabed. The table presented in appendix I3 summarises the collected suspended solids details.

As a vessel could not be mobilised during SW monsoon, the water samples have been collected from Well 8 and the Essar Jetty in the Tapi River during that period (for locations see appendix B).

A wide range in the total suspended solids data can be observed throughout the year. Partly this is caused by the moments on which the water samples have been taken. They seem to have been chosen rather arbitrarily. At first sight, it is hard to compare the total suspended solids data of the different months. The variations in time are looked at in the next section.

5.4 Variations in time

The amount of suspended solids shows spatial variations as well as considerable variations in time. In this section, the variations in sediment concentration over the tidal cycle, over the neap-spring cycle and over the season are discussed.

5.4.1 Tidal cycle

As presented in the time series plots (see appendix I2), the suspended solids concentration fluctuates considerably during a tidal cycle at the proposed Hazira harbour location. At Hazira, near the surface, a decrease of total suspended solids is found at and just after slack water. However, near the seabed, the amount of total suspended solids increases just after change of the current direction. This process can be explained by the settling of sediment particles at low flow velocities.

High-suspended sediment concentrations can be found in the vicinity of the proposed harbour. Around slack water, the flow velocity decreases rapidly and it is expected that a super saturated condition arises. Winterwerp, Uittenbogaard and de Kok (2001) stated that at concentrations beyond the saturation value, the water is no longer able to keep the sediment into suspension and the vertical concentration profile can collapse. A layer of fluid mud is formed on the seabed. When the tidal flow velocity increases again, this layer is entrained rapidly.

The sediment response often lags the flow. Hence, the maximum concentration may occur a few hours after maximum flood and ebb. At Hazira, there is a time lag of about one hour between the sediment concentration and the tidal flow.

5.4.2 Neap-spring tidal cycle

As shown in appendix I2, large differences in magnitude of total suspended solids are found in a neap-spring tidal cycle. At and just after spring tide, the amount of total suspended solids is greater than at (and just after) neaps. As has been mentioned in section 5.3.1, from the profiling measurements made in April, it can be concluded that the total suspended solids profile is relatively constant over the water column except at and after slack water. Especially during neaps this effect is well recognisable as the tidal energy decreases. At springs the low current settling time is shorter than at neaps due to the faster re-mixing of the water after slack.

5.4.3 Seasonal variations

Apart from the tide, the fresh water discharge and the climate also influence the suspended matter concentrations. As presented before, the fresh water discharge varies considerably during the year. In the monsoon period, the river discharges highly increase and therefore, the concentration of fluvial material is usually greater than in the non-monsoon period. Another reason for variation of total suspended solids is the difference in climate and weather. The climate and weather of the Gulf of Khambhat is very seasonal (see section 3.3.1). Especially during the SW monsoon period there are rough weather conditions. Strong winds and high waves do occur from the Southwest. This may lead to erosion of bed material and hence, to a high supply of suspended solids to the water column.

5.5 Conclusions total suspended solids

In the preceding sections, considerable variations in time and space have been distinguished concerning the suspended sediment concentration. As stated in section 3.3.1, the four typical seasons at the West coast of India are:

- the NE monsoon period (December - March)
- the hot season (April and May)
- the SW monsoon period (June - September)
- the interim period (October and November)

In the NE monsoon period, a mean value of 1000 mg/l is estimated for the amount of total suspended solids. This value is mainly based on data measured by Fugro Geos in December 2000. As the wave action decreases, the total suspended solids are expected to diminish as well after the NE monsoon period to a substantially lower concentration in April and May. The fresh water discharge is nil and the winds are light in this period. Although tropical storms or cyclones may occur in the Arabian Sea, their effect on the total suspended solids seems to be relatively small. According to total suspended solids time series plots and water quality analysis, the expected concentration is around 500 mg/l in the hot season. The SW monsoon can be characterised by strong winds and high waves, great fresh water discharges and high water temperatures. Therefore, great amounts of suspended solids are expected. Water samples have only been collected from Well 8 and the Essar Jetty in the Tapi River, as a vessel cannot be mobilised in the SW monsoon period due to safety regulations. A suspended solids concentration of 1500 mg/l is assumed for the SW monsoon period. For the interim period (October and November), only limited data is available, as no water samples have been collected in November. The October 2000 samples describe the situation 48 hours after neap tide, when the sediment concentration reaches its minimum value. In spite of this, the total suspended solids are in the same range as in the NE monsoon period. Occasional tropical storms may occur in the Arabian Sea in this period. For the interim period, the same suspended sediment concentration as in the NE monsoon period is assumed, hence 1000 mg/l. The mean amount of total suspended solids for every season is summarised in Table 5.2.

Table 5.2 Conclusions total suspended solids

Season	Total suspended solids [mg/l]
NE monsoon	1000
Hot season	500
SW monsoon	1500
Interim period	1000

It has to be mentioned that these values are based on a relatively poor data set. For a precise determination of the total suspended solids more, and more detailed, data is required. More turbidity measurements at fixed and chosen locations are desired. Furthermore, it is recommended to collect the water samples in the future at the same moment within a tidal cycle. Nonetheless, the values given in Table 5.2 are useful for a first indication. In the remainder of this thesis, a yearly mean suspended sediment concentration of 1000 mg/l is assumed.

6 Water exchange mechanisms

6.1 Introduction

Sediment particles are transported into harbour basins by exchange of water between the harbour and the environment. Five main flow mechanisms inducing this exchange can possibly be distinguished:

1. Filling and emptying by the tide
2. Entrainment by cross flow effects
3. Entrainment by longitudinal flow effects
4. Vertical circulation currents driven by density differences
5. Additional water exchange due to the water intake

In sections 6.2 to 6.6, these possible water exchange mechanisms are discussed for the proposed Hazira harbour and expressions of the magnitude are given.

Section 6.7 summarises the results of the foregoing sections and deals with the total water exchange volumes at the proposed harbour entrance. The total suspended sediment transport can be obtained by summarising its distinct influences. In equation (6.1) the possible suspended sediment transport due to the tide, the entrainment by cross flow effects, the entrainment by longitudinal flow effects, the density currents and due to the seawater intake are given.

$$\frac{dVc}{dt} = \underset{\swarrow}{T_t} + \underset{\substack{\downarrow \\ \text{Section 6.3}}}{T_{entrainment,cross}} + \underset{\substack{\downarrow \\ \text{Section 6.4}}}{T_{staganation}} + \underset{\substack{\searrow \\ \text{Section 6.5}}}{T_d} + \underset{\substack{\searrow \\ \text{Section 6.6}}}{T_{withdrawal}} \qquad (6.1)$$

where:

V	=	total volume of water exchange [m^3]
c	=	sediment concentration [kg/m^3]
T_t	=	suspended sediment transport due to the tide [kg/s]
$T_{entrainment,cross}$	=	suspended sediment transport due to entrainment by cross flow effects [kg/s]
$T_{entrainment,\ long.}$	=	suspended sediment transport due to entrainment by longitudinal flow effects [kg/s]
T_d	=	suspended sediment transport due to density differences [kg/s]
$T_{withdrawal}$	=	suspended sediment transport due to withdrawal of water [kg/s]

6.2 Water exchange due to filling and emptying by the tide

Within a tidal period, the water level increases during flood and decreases again during the ebb tide. This results in a net water movement to and from the basin. The water exchange due to the filling and emptying of the basin by the tide can be described by the one-dimensional shallow water equations. These equations form the base for long wave phenomena and may be used when the depth is relatively small in comparison to the wavelength. They are defined by:

$$b\frac{\partial h}{\partial t}+\frac{\partial Q_t}{\partial x}=0 \tag{6.2}$$

and:

$$\frac{\partial Q_t}{\partial t}+\frac{\partial}{\partial x}\left(\frac{Q_t^2}{A}\right)+gA\frac{\partial h}{\partial x}+\frac{g}{C^2}\frac{Q_t|Q_t|}{AR}=0 \tag{6.3}$$

where:

b	=	width of the entrance [m]
h	=	water depth [m]
Q_t	=	tidal filling discharge [m^3/s]
x	=	direction parallel to the harbour axis
A	=	cross sectional area [m^2]
C	=	Chézy coefficient [$m^{1/2}/s$]
R	=	hydraulic radius [m]

At the head of the harbour basin, $Q_t = 0$ applies for all t. Equation (6.3) then simplifies to $\frac{dh}{dx}=0$ at the head of the basin. This equation may be applied to the entire basin, if the length of the harbour is smaller than about five percent of the wavelength, as is very much the case in Hazira harbour. After integrating equation (6.2) to x with as boundary condition $Q_t = 0$ at the head of the harbour (x = 0), the tidal filling discharge yields:

$$Q_t = A_b\frac{dh}{dt} \tag{6.4}$$

where A_b is the storage area of the basin.

The water exchange over a whole tidal period, the tidal prism, can then easily be defined as:

$$V_t = 2\eta A_b \tag{6.5}$$

where V_t is the water exchange due to the tide and η is tidal amplitude. As presented in Figure 6.1, the mean tidal prism is equal to the water volume between Mean Low Water (MLW) and Mean High Water (MHW) in the tidal basin.

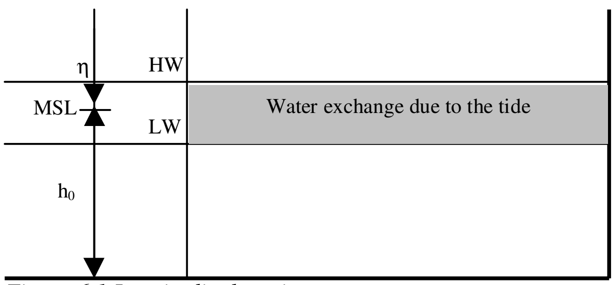


Figure 6.1 Longitudinal section

Fugro Geos has derived the tidal levels at Hazira from site investigations. As has been stated in section 3.3.2, the mean tidal range at Hazira equals 4.6 metres. It increases to the North of the Gulf.

Initially the port only accommodates the LNG berth, but handling of other berths in the future is desired. In the layout of appendix A, a future bulk terminal, a container terminal and a roro terminal are reflected. The storage area of the basin with only the LNG terminal is $1.9*10^6$ m^2. When the other berths are present as well, the storage area decreases to $1.5*10^6$ m^2, assumed that the terminals are filled (no construction on piles).

The water exchange due to filling and emptying by the tide over a tidal period, i.e. the tidal prism, can now be determined. As the storage area of the basin decreases when all the terminals are operational, the tidal prism decreases too. When the port only accommodates the LNG berth, the water exchange due to the tide equals ($4.6m*1.9*10^6m^2$=) $8.7*10^6$ m^3 per tide. For the 'full port layout', the tidal exchange amounts to $6.9*10^6$ m^3 per tidal period (=$4.6m*1.5*10^6m^2$).

6.3 Exchange due to entrainment by cross flow effects

It generally holds that currents along the harbour entrance are much larger than currents in the harbour entrance. As a result, the estuary flow separates and a mixing layer, with an asymmetric character, develops. This results in a flow pattern as schematised in Figure 6.2.

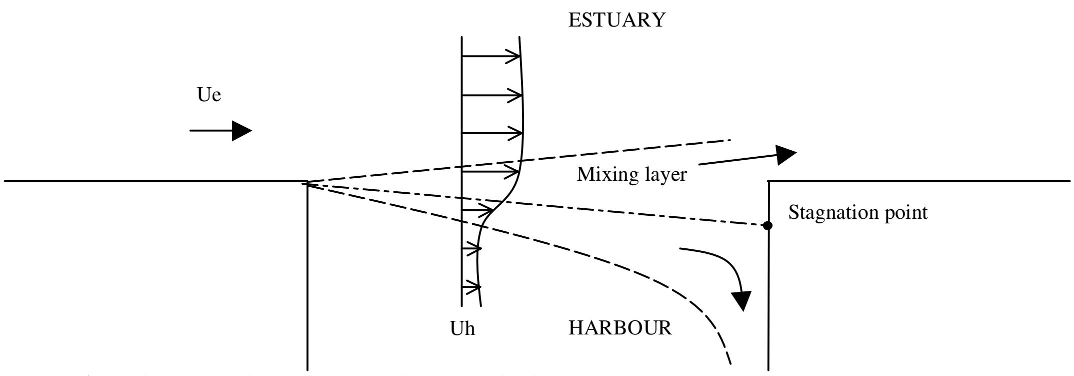


Figure 6.2 Velocity profile in the mixing layer in a harbour entrance

The centre line of the mixing layer is directed into the harbour basin. This is implied by conservation of mass when the velocity in the estuary is greater than the velocity in the mixing layer and in the basin. Water from estuary and water from the harbour basin integrate with each other in the mixing layer. An eddy in the harbour entrance is generated because water from the basin flows into the mixing layer and at the downstream side of the entrance, near the stagnation point, part of the water in the mixing layer flows back into the harbour basin.

Water exchange due to a horizontal current passing the entrance is not present at the proposed Hazira harbour. At Hazira, the entrance is not situated along the breakwater. As a result no distinct separation of the flow occurs and no mixing layer develops.

6.4 Exchange due to entrainment by longitudinal flow effects

Although no horizontal entrance eddy generated by the flow *along* the harbour entrance can be distinguished, a circulation is expected to develop in the Hazira harbour entrance. Entrainment by longitudinal flow effects causes this circulation and a turbulent water exchange process is developed. Through this mechanism, silt-laden water from outside enters the circulation current, which transports the sediments further into the basin. During flood, the direction of the circulation current is clockwise, while during ebb the direction of the eddy reverses.

Before determining the water exchange volumes due to entrainment by longitudinal flow effects at Hazira, section 6.4.1 deals with the flow pattern in the vicinity of Hazira. The water exchange by this mechanism can be conceived as a turbulent diffusion process. Therefore, the main aspects of turbulence are discussed in section 6.4.2. To determine the water exchange caused by longitudinal flow effects at the proposed Hazira harbour, a comparison is made with the existing harbour at IJmuiden in the Netherlands. Section 6.4.3 gives a description and deals with the flow pattern of the IJmuiden harbour area. Finally, section 6.4.4 calculates the water exchange caused by entrainment by longitudinal flow effects at the proposed Hazira harbour entrance.

6.4.1 Current conditions around Hazira

Svašek has carried out depth-averaged flow computations for the area surrounding Hazira by using a flow model called FINEL, which is based on the shallow water equations. An area of 35x50 km^2 of the coastal region around Hazira is divided into small triangles. The grid schematisation is given in appendix J. To obtain more accuracy in the region concerned, smaller triangles are chosen there. Within each triangle the flow variables are assumed constant. Differences in flow variables between the adjacent elements lead to an exchange of mass and momentum. FINEL calculates the corresponding flow. Bed friction, Coriolis acceleration, wind shear stress and flooding and drying of intertidal areas are included in the model. The modelling of the bathymetry of the area surrounding Hazira is based on data from the Admiralty Chart 1486, the Indian Charts of the Tapi River and from the local survey digitised by Delta Marine Consultants.

At the open sea boundaries of the model, the water levels are specified. The one remaining open boundary of the model is the Tapi River. There, the river discharge is defined. During the monsoon period, an additional wind drift of 0.30 m/s from the Southwest is applied.

The model has been calibrated and it is concluded that the water levels, the tidal velocities and the current directions are reproduced fairly well. Remaining discrepancies are the moments of slack tide related to high water and, to a less extent, the phase difference between the observed and the measured moment of low water.

Flood currents are directed North-Northwest (335°-360°) while ebb currents follow a South-Southeast (165°-180°) direction. Maximum flood occurs 2 hours before high water. The maximum flood velocity during a well-developed spring tide equals 2.9 m/s along the breakwater. Maximum ebb takes place 4 hours after high water and is equal to 1.3 m/s along the breakwater during such a well-developed spring tide.

The port entrance is orientated in Northwest direction. This orientation reduces the cross currents in front of the harbour entrance. Furthermore, incoming ships partly head into the SW (and NE) monsoon waves and winds. In this way, the ships have sufficient rudder control and their stopping distance within the port is limited.

The Sutherland Channel, just in front of the harbour, is a flood-dominated channel. The currents in this channel are directed NW during flood and SE during ebb. Although the main breakwater of the proposed harbour is aligned with the flow, the presence of the harbour causes higher velocities in the Sutherland Channel due to the contraction and bending of the streamlines around the harbour. After the flow has reached the breakwater tip, the velocity decreases again because of the increased depth of the approach channel and the diversion of the flow. See Figure 6.3.

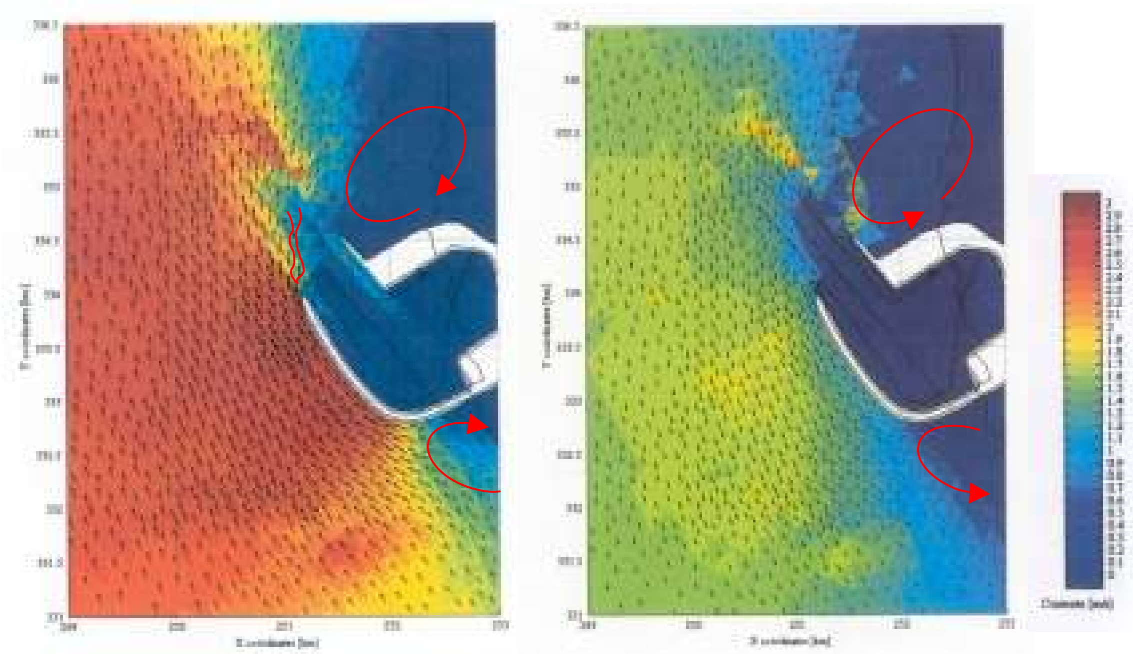


Figure 6.3 2D flow pattern with maximum flood and maximum ebb at a well-developed spring tide

As mentioned before, Svašek has carried out depth-averaged flow computations for the area surrounding Hazira. In reality however, the velocity profile is more complex and secondary flow effects are not negligible. Especially, curvature induced secondary flows may be of great influence to the current, the sediment transport and the bed topography. In the case of the proposed harbour at Hazira, secondary flow is present Northeast of the North spur and Southeast of the main breakwater. The residual currents due to the secondary induced flow are presented in Figure 6.3 by the red circles. At the transition from the estuary to the harbour basin, around the breakwater tip, an unstable free shear layer is expected to develop during flood.

6.4.2 Turbulence

Most flows occurring in nature and in engineering are turbulent. But an unambiguous and precise definition of turbulence is hard to give. Hinze (1975) defines turbulence as follows: "Turbulent fluid motion is an irregular condition of flow in which the various quantities show a random variation with time and space coordinates, so that statistically averaged values can be discerned".

Tennekes and Lumley (1972) prefer an enumeration of some of the characteristics of turbulence:

- *Irregularity* or randomness of all turbulent flows. This feature makes only a statistical approach to turbulence problems possible.

- *Diffusivity* is one of the most important characteristics of turbulence. It contributes to a great transport of momentum, heat and mass and takes care of fast spreading of velocity fluctuations. The rates of transfer and mixing are several orders of magnitude greater than the rates due to molecular diffusion.

- *Large Reynolds numbers* always arise with turbulence. The Reynolds number is defined as:

  $$\mathrm{Re} = \frac{uL}{\nu} \tag{6.6}$$

  where u is a characteristic velocity, L a characteristic length scale and ν the kinematic viscosity [m^2/s]

  Laminair flows occur at a relatively small number of Reynolds. When the Reynolds number becomes too large, the laminair flows become unstable and turbulence occurs. These instabilities are caused by the complex interaction between the non-linear inertia terms and the viscous terms in the equation of motion.

- *Three-dimensional vorticity fluctuations.* Turbulence is a three-dimensional phenomenon and is characterised by high levels of fluctuating vorticity. The random vorticity fluctuations cannot maintain themselves in two-dimensional flow.

- *Dissipation.* Turbulent flows are much more dissipative than laminair flows. A lot of kinetic energy of the turbulence is lost by the deformation work resulting from viscous shear stresses. A great supply of energy to compensate these viscous losses is needed to maintain turbulence.

- *Continuum.* The smallest length scales occurring in turbulence are usually much larger than any molecular length scale. This means that the continuum description, governed by the equations of fluid mechanics, holds for turbulence.

- *Turbulent flows are flows.* Turbulence is not a feature of fluids but of fluid flows. Turbulent flows have numerous characteristics in common. However since all flows are different each flow pattern has certain unique characteristics because the initial and boundary conditions of the non-linear equations of motion differ.

No robust mathematical methods have been realised yet to solve the problems concerning turbulence completely. Due to the randomness and the non-linearity of the equations of motion, turbulence theory heavily relies on empirical data. Analyses of the equations of motion have always lead to situations in which there are more unknown factors than equations. This is called the "closure problem of turbulence theory".

In the present thesis a dimensional analysis of the turbulent exchange processes due to entrainment by longitudinal flow effects is made. So far, research has only been carried out on entrance eddies that are generated by currents *along* the harbour entrance. At present, studies concerning eddies generated by longitudinal flow are not available.

In section 6.4.3, it is proved that the water exchange by this mechanism depends on a number of independent variables. Multiplying the variables by a certain coefficient, the solution of the water exchange due to entrainment by longitudinal flow effects is known. This coefficient is estimated by comparing the proposed harbour at Hazira with the present harbour at IJmuiden in the Netherlands. Section 6.4.3 also explains why it is justified to make this comparison. In order to make a good estimation of the water exchange by turbulence, it is important to determine a representative eddy diffusivity coefficient and an appropriate length scale. The next section deals with the eddy diffusivity.

Eddy diffusivity

To simplify the mathematics of turbulence, the diffusive nature of turbulence is treated by means of a representative eddy diffusivity. This is a somewhat dangerous approach since turbulence tends to be treated as a property of fluid instead of a property of flow. However, this approach makes it possible to determine the gross consequences of turbulence with relatively simple tools. Describing the effects of turbulence by the standard diffusion equation yields equation (6.7).

$$\frac{\partial c}{\partial t} = \frac{\partial}{\partial x_i} D_{t,i} \frac{\partial c}{\partial x_i} \tag{6.7}$$

where c is the sediment concentration, D_t the eddy diffusivity [m^2/s] and x the transport direction.

when a constant eddy diffusivity is assumed, equation (6.7) becomes:

$$\frac{\partial c}{\partial t} = D_t \frac{\partial^2 c}{\partial x_i^2} \tag{6.8}$$

Because only the gross consequences of turbulence are desired, a dimensional approach is sufficient. By doing so, equation (6.8) yields:

$$\frac{\Delta c}{T} \propto D_t \frac{\Delta c}{L^2} \tag{6.9}$$

in which L and T reflect the length and time scale respectively.

From equation (6.9) follows that the associated time scale T can described by:

$$T \propto \frac{L^2}{D_t} \tag{6.10}$$

The turbulent time scale T_t has the dimension of seconds and may therefore also be characterised by the ratio of the length scale L and a velocity scale of eddying flow u'. See equation (6.11).

$$T_t \propto \frac{L}{u'} \tag{6.11}$$

Combining equations (6.10) and (6.11) yields:

$$D_t \propto u' L \tag{6.12}$$

This expression shows that the eddy diffusivity is proportional to the product of a certain length scale and the circulation velocity of the eddy. In turbulent flows a wide range of length scales exists. In most cases only the sizes of the largest eddies are considered because they do most of the transport of momentum and sediment. Equation (6.12) does not give a numerical value. Therefore an empirical or experimentally determined coefficient is needed.

6.4.3 Case study: Present harbour at IJmuiden

To determine the water exchange caused by turbulence at the proposed Hazira harbour, a comparison is made with the IJmuiden harbour in the Netherlands. Rijkswaterstaat, which is the Dutch Directorate-General for public works and water management, has drawing up a report about the water and sediment movement in the harbour entrance at IJmuiden (Rakhorst, 1982). Measurements in and close to the harbour basin have been done during three days in September 1981. The way the water flows in and out of the basin and the way in which the sediment transport takes place are the main points of interest in this report.

A lot of similarities between the IJmuiden harbour and the Hazira harbour can be found. Like at Hazira, the entrance of the IJmuiden harbour is orientated in Northwestern direction, the flow goes North during flood and South during ebb. An overview of the IJmuiden harbour is given in Figure 6.4.

Figure 6.4 IJmuiden harbour

The distance between the Northern and Southern breakwater is 800 metres and has a direction of 60 degrees to the North. The harbour entrance has been divided into different sections such that the expected flow enters the basin perpendicular to these sections. For every section, the mean depth has been determined. By multiplying the width of each section with the corresponding mean depth, summarising these values and dividing them by the total width of all the sections, the overall mean depth can be determined. The mean depth at the entrance of IJmuiden is estimated 13 metres.

Flow pattern

The flow measurements result in different flow patterns for the different phases of the tide. Entrainment by longitudinal flow effects causes a large entrance eddy at the IJmuiden harbour. Like at Hazira, during flood it is directed to the right, while during ebb the direction of the eddy reverses. See Figure 6.5.

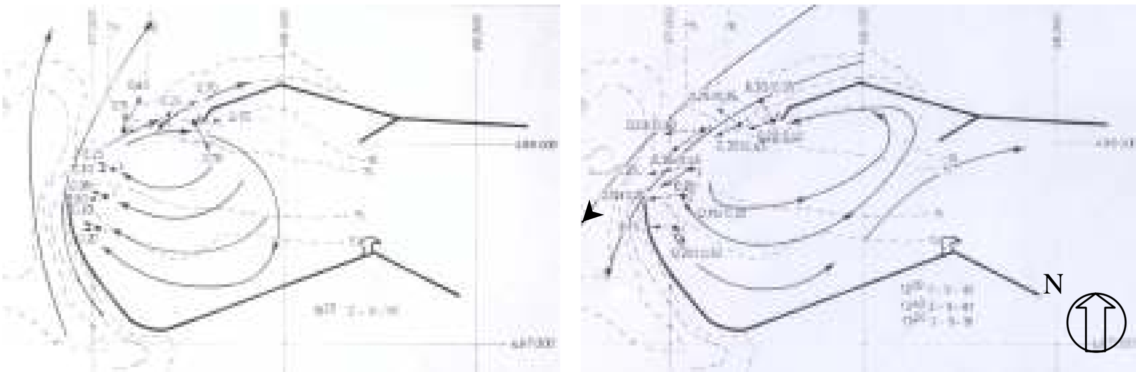


Figure 6.5 Flow patterns IJmuiden harbour (Rakhorst, 1982)

In the beginning of the flood period, water enters the basin in the centre of the entrance channel. As the flood continues, the flow enters the basin at the North of the basin entrance. The water flows out at the Southern side of the entrance during the whole flood period. In the beginning of the ebb, the water still enters the basin at the Northern side of the entrance. When ebb continues, the direction of the eddy reverses and the water flows in at the Southern side and leaves the basin at the Northern side of the entrance.

The maximum velocities in the IJmuiden harbour entrance at maximum flood are about 0.70-0.90 m/s at the Northern spur (during only one hour) and about 0.5 m/s at the Southern breakwater. The maximum ebb velocities are about half the maximum flood velocities.

Outside the harbour basin, the current velocity and direction has been measured in two reference locations. One point is situated to the North and one to the South of the harbour; location number 7 and 8 respectively. In Figure 6.6, the current velocity, with its direction, is shown for these two reference locations during the second day in September 1981. This figure presents a characteristic profile for the situation during mean tide.

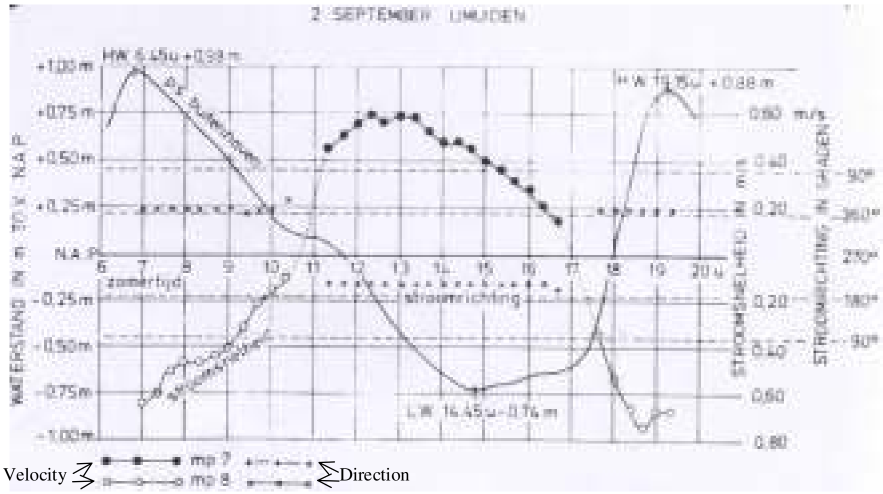


Figure 6.6 Current velocities in the reference points (outside the influence of the harbour)

The maximum current velocity at the water surface during a mean tide is assumed to be about 0.8 m/s and 0.6 m/s for flood and ebb respectively.

Discharge

The discharge through the IJmuiden harbour entrance has been measured every 20 minutes during the concerning period from 1 to 3 September 1981. The exchange rates are determined by filling and emptying by the tide, by flushing from the Northsea Channel to the Northsea and by an entrance eddy that is caused by entrainment due to longitudinal flow effects. The circulation at the entrance is responsible for the greatest part of the discharge. Table 6.1 shows the different water exchange mechanisms with their respective percentages that determine the discharge.

Table 6.1 Percentages that determine the discharge

	Ebb Percentage of outgoing discharge	Flood Percentage of incoming discharge	Ebb + Flood Percentage of total discharge
Eddy	70 - 75 %	40 - 60 %	75 - 80 %
Flushing	3 - 5 %	15 - 25 %	2 - 6 %
Tide	20 - 25 %	35 - 40 %	16 - 18 %

From the discharge measurements at the IJmuiden harbour basin entrance, it can be concluded that the sum of the incoming and outgoing discharge is about $90*10^6$ m^3 per tide. This value can be divided into $55*10^6$ m^3 and $35*10^6$ m^3 per tide for ebb and flood respectively. The flushing volume is about $5*10^6$ m^3 per tide and the tidal prism equals approximately $7*10^6$ m^3. These specific exchange rates, which hold for a mean tide, are reflected in Table 6.2.

Table 6.2 Exchange rates at IJmuiden harbour

	Ebb	Flood	Ebb + Flood
Incoming discharge [m^3]	$23 * 10^6$	$19 * 10^6$	$42 *10^6$
Outgoing discharge [m^3]	$32 * 10^6$	$16 * 10^6$	$48 *10^6$
Tide [m^3]	$7,0 * 10^6$	$7,0 * 10^6$	0
Flushing [m^3]	$1,0 * 10^6$	$4,0 * 10^6$	$5,0 *10^6$

To determine the water exchange due to entrainment by longitudinal flow effects in the IJmuiden harbour basin, the water exchange due to the tide and the additional water exchange caused by flushing are discarded. The resulting exchange is the water exchange caused by entrainment by longitudinal flow effects in one tidal cycle, reflected by the eddy.

Table 6.3 Discharge rates caused by entrainment by longitudinal flow effects in the IJmuiden harbour

	Ebb	Flood	Ebb + Flood
Incoming discharge [m^3/tide]	$23 * 10^6$	$12 * 10^6$	$35 * 10^6$
Outgoing discharge [m^3/tide]	$23 * 10^6$	$12 * 10^6$	$35 * 10^6$

As expected, the incoming and outgoing discharges are the same. The water due to entrainment by longitudinal flow effects amounts to $35*10^6$ m^3 per tide at the present harbour at IJmuiden. This means that the eddy is responsible for $\frac{(35+35)*10^6}{90*10^6}*100\% = 78\%$ of the total discharge. This is in line with the percentages presented in Table 6.1.

The sediment influx caused by entrainment by longitudinal flow effects is given by:

$$T_{entrainment,long.} = \frac{d(V_{entrainment,long.} * c)}{dt} \tag{6.13}$$

where $T_{entrainment,long.}$ is the suspended sediment transport due to entrainment by longitudinal flow effects [kg/s], $V_{entrainment,long}$ is the volume of water exchange caused by this effect [m^3] and c is the mean sediment concentration [kg/m^3].

The suspended sediment transport caused by this mechanism $T_{entrainment,long.}$ can be expressed by the eddy diffusivity D_t, as described in section 6.4.2 (see equation (6.12)), the cross sectional area of the harbour entrance A, the sediment concentration c and the transport direction x.

Equation (6.14) reflects this relationship:

$$T_{entrainment,long.} = D_t \frac{dAc}{dx} \tag{6.14}$$

Equation (6.14) may dimensionally be interpreted as:

$$T_{entrainment,long.} \propto D_t \frac{\Delta(bhc)}{L} \tag{6.15}$$

where b is the width of the entrance, h is the entrance depth and L is a characteristic length scale.

Dimensional interpretation of equation (6.13) and combining it with equation (6.15) yields:

$$\frac{\Delta(V_{entrainment,long.} * c)}{\Delta t} \propto D_t \frac{\Delta(bhc)}{L} \tag{6.16}$$

Section 6.4.2 has already shown that the eddy diffusivity is proportional to the product of the circulation velocity of the eddy and an appropriate length scale. In the case of Hazira, the specific length and velocity scales represent the largest eddy sizes because these are responsible for the largest part of the transport of momentum and sediment particles. The largest eddies are supposed to be as large as the width of the incoming flow.

The following relation is assumed:

$$L \approx b \tag{6.17}$$

Substituting the preceding into formula (6.16), assuming a constant cross sectional area over the width of the entrance and dividing the result by Δc yields:

$$Q_{entrainment,long.} \propto u'bh \tag{6.18}$$

where $Q_{entrainment,long}$ reflects the discharge due to entrainment by longitudinal flow effects [m^3/s].

The discharge due to entrainment by longitudinal flow effects is proportional to the circulation velocity of the eddy u' and the cross sectional area of the entrance (bh). If all the variables are known, the solution is known except for a numerical coefficient. As a lot of similarities between the IJmuiden harbour and the proposed Hazira harbour can be found, this coefficient k can be obtained by scale relation, relating the proposed Hazira harbour to the existing harbour at IJmuiden. The circulation velocity is replaced by a characteristic velocity outside the basin. Furthermore, the mean depth h_0 and a characteristic width b_{char} of the basin entrance are used. This is included in the coefficient k. See equation (6.19).

$$Q_{entrainment,long.} = k * u_{char} b_{char} h_0 \tag{6.19}$$

As stated earlier, the distance between the Southern and Northern breakwater of the port of IJmuiden, i.e. the width of the entrance, is 800 metres and its mean depth amounts to about 13 metres. The water exchange caused by entrainment by longitudinal flow effects is $35*10^6$ m^3 per tide.

Substitution of these values into equation (6.19) yields for the numerical coefficient k:

$$k = \frac{Q_{entrainment,long.}}{\left(u_{car} b_{char} h_0\right)_{IJmuiden}} = \frac{35*10^6}{u_{char} * 800 * 13} = \frac{3.4*10^3}{u_{char,IJmuiden}} \tag{6.20}$$

The numerical coefficient k is known, except for the characteristic velocity at IJmuiden. The value of the characteristic velocity is largely dependent on which location is looked at. The next section deals with this parameter.

6.4.4 Water exchange due to entrainment by longitudinal flow effects at Hazira

This section focuses on the water exchange due to entrainment by longitudinal flow effects at the proposed Hazira harbour. As has been stated before, a large eddy is expected to develop in the harbour entrance induced by longitudinal flow effects. Figure 6.7 gives an overview of the situation during ebb.

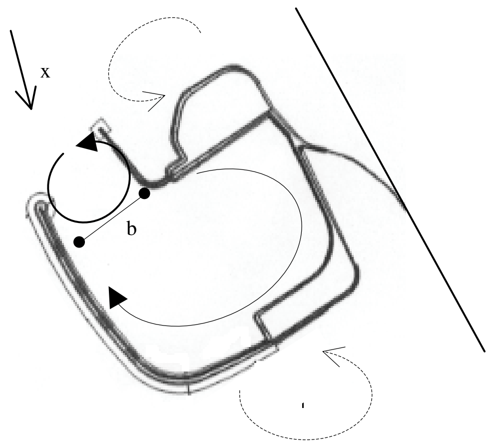


Figure 6.7 Situation at the Hazira harbour entrance during ebb

The water exchange per tide caused by entrainment by longitudinal flow effects at Hazira, $V_{entrainment,long.,Hazira}$ [m^3/tide], can now be defined by substitution of equation (6.20) into equation (6.19):

$$V_{entrainment,long.,Hazira} = 3.4 * 10^3 \frac{u_{char,Hazira}}{u_{char,IJmuiden}} \left(b_{char} h_0\right)_{\text{Hazira}} \tag{6.21}$$

The width of the Hazira harbour entrance is approximately 465 metres. As the orientation of the IJmuiden harbour and Hazira harbour is approximately the same, the widths of the harbour entrances can be compared directly. The mean depth depends on which situation is looked at. When only the LNG terminal is fully operational, the cross sectional area of the entrance is about 7000 m^2 relative to mean sea level. The accompanying mean depth equals the quotient of the cross sectional area and the characteristic width and amounts to around 15 metres. However, when all the terminals are fully operational, the cross sectional area equals about 8600 m^2 with a mean depth of approximately 18 metres.

The only unknown factor of equation (6.21) that remains is the ratio of a characteristic velocity at Hazira to a characteristic velocity at IJmuiden. This factor is discussed below.

Velocity ratio

The desired ratio is given by $\left(\frac{u_{char,Hazira}}{u_{char,IJmuiden}}\right)$. The question arises which location needs to be chosen to optimise comparison between the current velocities at Hazira and at IJmuiden. In first instance, the undisturbed flow velocities are looked at, thus the velocities outside the influence of the harbours concerned.

At IJmuiden, the undisturbed flow velocity has been measured in two reference points. On the basis of Figure 6.6 (location 8), the maximum flood velocity at the water surface, during a mean tide is assumed to amount to 0.8 m/s.

In the vicinity of Hazira, considerable tidal currents occur due to the large vertical tidal ranges. The maximum current velocity is directly proportional to the tidal range. Measurements around the proposed harbour location have been done at location C3 (see appendix B) in the months July and November 2000 and April 2001. The relations between the current velocity at the water surface and the tidal range are shown in appendix K. With a mean tidal range of 4.6 metres at Hazira, the maximum current speed at the water surface during flood is about 1.6 m/s for the months July and November and about 1.5 m/s for April.

For the velocity ratio, due to comparison of the undisturbed flood velocities, the following holds:

$$\left(\frac{u_{Hazira}}{u_{IJmuiden}}\right)_{undisturbed} \approx 2 \qquad (6.22)$$

At Hazira, the maximum flood velocities during a mean tide are approximately twice as large as at IJmuiden.

However, the bathymetry of the proposed Hazira harbour location differs from the bathymetry of the IJmuiden harbour area. At Hazira, the adjacent coastal region is characterised by a shallow and partly drying foreshore, while around the IJmuiden harbour relatively large depths can be found. Just to the North of the proposed Hazira harbour location, a sandbank, which is in between 3 and 4 metres above Chart Datum, is present. The shoals are expected to influence the velocities in front of the Hazira harbour. As a result of this difference in bathymetry, the velocities in front of the relevant harbours are compared with each other as well.

Figure 6.8 shows the mean measured current pattern for the area around IJmuiden and the modelled current pattern for the area around Hazira at maximum flood. In the figure, the tidal stream data is presented in knots for the harbour at IJmuiden and in m/s for the proposed Hazira harbour.

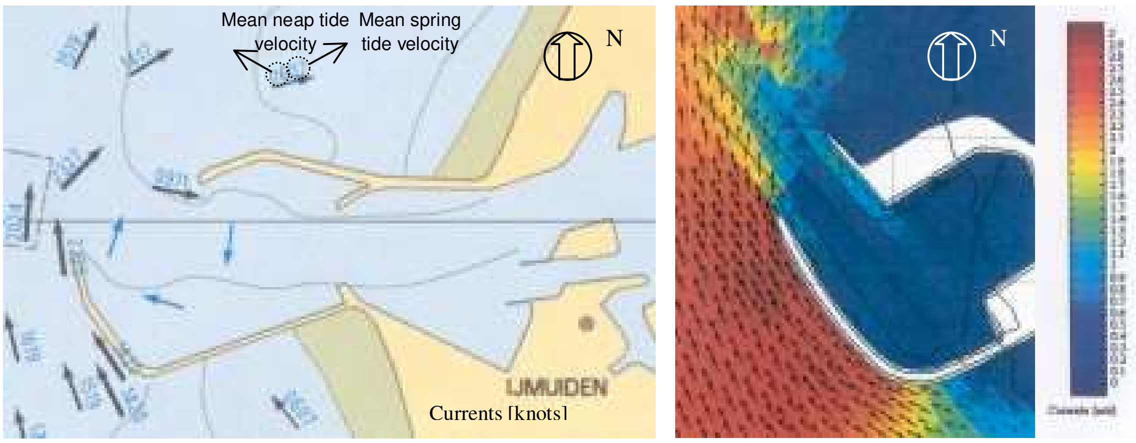


Figure 6.8 Maximum flood velocities at the IJmuiden and the Hazira harbour entrance

In front of the IJmuiden harbour entrance, outside the influence of the circulation current, the maximum flood velocity, referring to the upper water layer, amounts to 2.3 knots (1.2 m/s) at mean neap tide and to about 2.7 knots (1.4 m/s) at mean spring tide. The velocity pattern during maximum flood around the Hazira harbour entrance is computed for a well-developed spring tide. Comparison of the flow velocities, outside the circulation, yields another velocity ratio that includes the anticipated impact of the proposed Hazira harbour. In this case, the maximum flood velocities in front of the IJmuiden and the Hazira harbour entrances are about the same. See equation (6.23).

$$\left(\frac{u_{Hazira}}{u_{IJmuiden}}\right)_{disturbed} \approx 1 \tag{6.23}$$

From the preceding, it follows that the ratio of the velocity at Hazira to the velocity at IJmuiden is largely dependent on the location evaluated. The water exchange due to entrainment by longitudinal flow effects is thought to be proportional to this velocity ratio (see equation (6.21)). Therefore, great differences in water exchange volumes due to this mechanism are found. The water exchanges for the disturbed and the undisturbed conditions are shown in Table 6.4.

Table 6.4 Water exchange volumes caused by entrainment by longitudinal flow effects for the different velocity ratios

	$\left(\frac{u_{Hazira}}{u_{IJmuiden}}\right)_{disturbed} \approx 1$	$\left(\frac{u_{Hazira}}{u_{IJmuiden}}\right)_{undisturbed} \approx 2$
Only LNG terminal fully operational [m3/tide]	$V_{entrainment,long.,Hazira} = 2*10^7$	$V_{entrainment,long.,Hazira} = 5*10^7$
All terminals fully operational [m3/tide]	$V_{entrainment,long.,Hazira} = 3*10^7$	$V_{entrainment,long.,Hazira} = 6*10^7$

As results from Table 6.4, the water exchange due to entrainment by longitudinal flow effects is expected to vary in between $2*10^7$ and $6*10^7$ m^3 per tide. It depends largely on the adopted velocity ratio. The 'disturbed' and the 'undisturbed' velocity ratios respectively provide a lower and an upper limit of the water exchange caused by this mechanism. Furthermore, due to the increased depth, the water exchange due to this mechanism is somewhat larger when all terminals are fully operational.

6.5 Density driven exchange flow

The density induced water exchange may be of great importance for harbour siltation. The exchange flow is driven by density differences between the water in and outside the harbour basin.

If the density of the water outside the harbour is larger than the density of the water in the basin, water enters the basin along the seabed, while water from the basin leaves the estuary near the surface. This process is reversed when the density of the water outside the basin is lower. The density induced water exchange affects the whole harbour basin, while most water exchange mechanisms are restricted to the area near the entrance. Tidal filling or emptying of the basin reduces the density driven exchange flow. The discharge rate due to the tide equals $2u_th/2$, where u_t is the tidal velocity. Figure 6.9 shows a schematisation of the density induced water exchange under various circumstances.

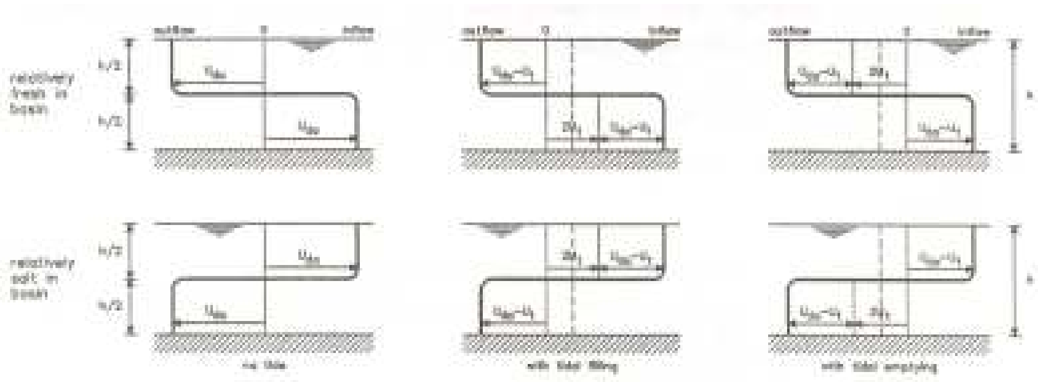


Figure 6.9 Schematised profile of density induced water exchange

In reality, the toe of the salt wedge is held back slightly by friction.

The current caused by density differences can be compared to a lock with fresh water on one side and salt water on the other side (see Coastal Engineering Volume I, 2000). See Figure 6.10.

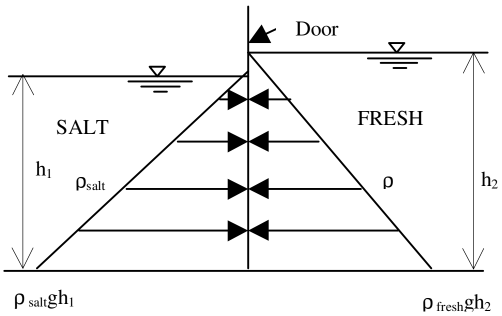


Figure 6.10 Comparison between density currents and hydrostatic pressures on a lock, at equilibrium

At equilibrium holds:

$$\frac{1}{2}\rho_{salt} g h_1^2 = \frac{1}{2}\rho_{fresh} g h_2^2 \tag{6.24}$$

However, this condition is not stable. The resulting moment is not zero and a current develops. The velocity of this current equals the velocity of the density current.

The density current speed can be estimated by:

$$u_{do} = f * \sqrt{\frac{\Delta\rho}{\rho} gh} \tag{6.25}$$

where u_{do} is the exchange flow velocity without influence of tidal in- and outflow , $\Delta\rho$ is the characteristic density difference and f is a coefficient resulting from friction (≈0.45).

When density differences between the water in and outside the harbour basin are present, the rate of density induced exchange flow can be described by:

$$Q_d = (u_{do} - u_t) * \frac{1}{2} hb \tag{6.26}$$

where Q_d is the exchange rate due to density currents [m^3/s] and u_t is the tidal velocity [m/s].

The density induced exchange in a small harbour is expected to be less than the exchange in a large harbour because the water in the small basin follows the density fluctuation in front of the harbour more easily. This results in a reduction of the characteristic density difference inducing the density currents.

The relationships between a number of relevant parameters are given in Figure 6.11.

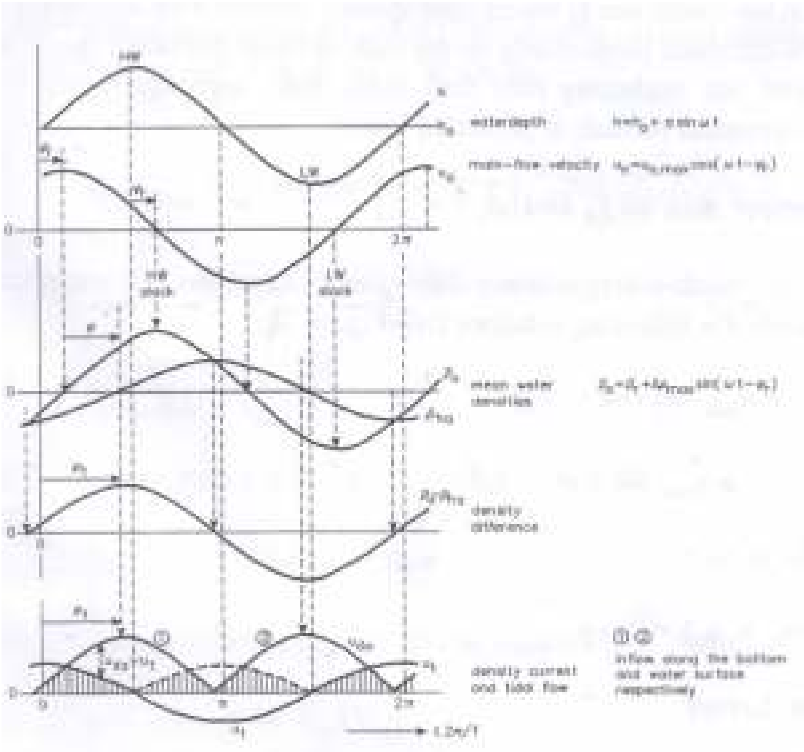


Figure 6.11 Relationships between the main parameters concerning the density induced exchange

The density differences between the water outside the harbour and inside the basin can be induced by differences in salinity, in sediment concentration, and in temperature, or a combination of these factors. Temperature variation between the harbour basin and the estuary is not anticipated and is not dealt with in this report. It generally holds that the difference in salinity is the most important factor for density induced exchange flow.

However, as has been mentioned in section 3.3.8, no salinity differences over a tidal period can be distinguished in the vicinity of Hazira. As a result, no water exchange due to salinity induced density currents is expected in the proposed Hazira harbour basin. The well-mixed condition in the Gulf of Khambhat is also proved by the determination of the estuary number in chapter 4.

6.6 Additional exchange due to withdrawal of water

Withdrawal or discharge of water from and to the basin affects the water exchange between the harbour basin and the estuary. Mainly, the flow velocities in the harbour entrance due to tidal filling and emptying of the basin are influenced. Through this it may also influence the other water exchange mechanisms.

If water is withdrawn from the harbour or if water with the same density is released into it, no extra density currents occur. If water with a different density is discharged into the harbour basin, the situation becomes more complex. The flushing water partly mixes with water from the harbour basin and flow along the water surface (or seabed) towards the harbour entrance where it flows out of the basin as an additional density flow.

Since there is a water intake in the basin at the proposed Hazira harbour, no additional density differences occur and are therefore not discussed further. The seawater outfall is situated outside the harbour basin (see appendix A). The discharges of the intake and the outfall depend on the LNG throughput. For every $1*10^9$ kg/year of LNG, a pump with a capacity of 1 m^3/s is required. The LNG terminal is originally dimensioned to handle $2.5*10^9$ kg/year expanding to a throughput of $5*10^9$ kg/year pa. On the longer term, when also the bulk and container terminal are fully operational, a throughput of $10*10^9$ kg/year is required. A constant withdrawal of water during the whole year and during all the tides is assumed. In the situation with only an operational LNG terminal, a withdrawal discharge of 5 m^3/s is needed. When all the terminals are operational, the required suction discharge is 10 m^3/s.

The tidal flow velocity in the harbour entrance, when the seawater intake is present, equals:

$$u_{tf} = \hat{u}_t \cos(\omega t) - u_f \tag{6.27}$$

where u_{tf} is the tidal flow velocity in the entrance with flushing or withdrawal, u_t is the tidal flow velocity in the entrance without flushing or withdrawal and u_f is the flow velocity in the entrance due to flushing or withdrawal.

Three different situations can be distinguished:

1. $$\frac{u_f}{\hat{u}_t} \geq 1 \tag{6.28}$$
2. $$\frac{u_f}{\hat{u}_t} \leq -1 \tag{6.29}$$
3. $$-1 \leq \frac{u_f}{\hat{u}_t} \leq 1 \tag{6.30}$$

As withdrawal of water is defined as negative flushing, the value of $u_f/\hat{u}_t$ is always negative. Therefore, for the Hazira harbour project, situation two or three holds.

Assume the following relations:

$$u_f = \frac{Q_f}{A_0} \tag{6.31}$$

$$\hat{u}_t = \frac{\pi * V_t}{A_0 * T} \tag{6.32}$$

where Q_f is the suction discharge (defined as negative), V_t the tidal prism and A_0 the area of harbour entrance below Mean Sea Level.

Substitution of these equations in the relation $\frac{u_f}{\hat{u}_t}$ yields:

$$\frac{u_f}{\hat{u}_t} = \frac{\frac{Q_f}{A_o}}{\frac{\pi V_t}{A_o T}} = \frac{Q_f T}{\pi V_t} \tag{6.33}$$

With only an operational LNG terminal, the relation between the amplitude of the tidal flow velocity and the withdrawal velocity in the entrance is $-7.5*10^{-3}$. This ratio amounts to $-18*10^{-3}$ for the ‘full port layout'.

Using equation (6.30), the contribution of the withdrawal discharge only occurs during that part of the tide when u_{tf} is positive. The water exchange per tide due to the combined contribution of the tidal prism and the withdrawal of water V_{tf} can be approximated by the following integral:

$$V_{tf} = 2A_0 \int_0^t (u_t \cos(\omega t) - u_f)dt \tag{6.34}$$

with:

$$t = \frac{T}{2\pi} \arccos\left(\frac{u_f}{\hat{u}_t}\right) \tag{6.35}$$

Integration gives:

$$V_{tf} = V_t \sin(\omega t) - 2Q_f t \tag{6.36}$$

In Table 6.5, the results of the equations given above are summarised. This table shows that the water exchange due to the combined influence of the tide and the intake increases with only a few percent in comparison to the situation without an intake.

Table 6.5 Combined influence of tidal prism and withdrawal of water

	Only LNG terminal fully operational	All terminals fully operational
u_f	$-2.4*10^{-6}$ m/s	$-5.7*10^{-6}$ m/s
$\hat{u}_t$	$3.2*10^{-4}$ m/s	$3.2*10^{-4}$ m/s
t_1	$1.1*10^{4}$ s	$1.1*10^{4}$ s
Q_f	-5 m^3/s	-10 m^3/s
V_t	$8.7*10^{6}$ m^3/tide	$6.9*10^{6}$ m^3/tide
V_{tf}	$8.9*10^{6}$ m^3/tide	$7.1*10^{6}$ m^3/tide

Hence, it is assumed that the influence of the intake on the total water exchange volumes present in the proposed Hazira harbour can be neglected.

6.7 Conclusions

In this chapter, the mechanisms that cause sediment transport to the proposed Hazira harbour basin have been considered. The exchange between the harbour and the Gulf is induced by the water motion in the entrance. At the Hazira harbour entrance, two main exchange mechanisms can be distinguished: (1) water exchange due to filling and emptying by the tide, and (2) turbulent water exchange processes caused by entrainment due to longitudinal flow effects.

The last mechanism, which has been described in section 6.4, is responsible for the greatest part of the total exchange volume. A large eddy, which is generated by entrainment due to longitudinal flow effects, is expected to develop in the Hazira harbour entrance. After relating the proposed Hazira harbour to the existing harbour at IJmuiden in the Netherlands, the water exchange due to entrainment by longitudinal flow effects at the proposed Hazira harbour ($V_{entrainment,long.}$) has been determined. At present, studies concerning eddies generated entrainment due to longitudinal flow effects are not available. Usually, entrance eddies are generated by currents *along* the harbour entrance. This mechanism, which is discussed in section 6.3, is not present at the proposed Hazira harbour entrance. At Hazira, no distinct separation of the flow occurs and no mixing layer develops.

The water exchange volume due to filling and emptying by the tide V_t, has been discussed in section 6.2. It depends on the variations in tidal level, as well as on the surface area of the harbour.

In section 6.5, it has been concluded that no water exchange due to salinity-density currents V_d is expected in the proposed Hazira harbour basin. Subsequently, section 6.6 states that the additional water exchange due to the seawater intake $V_{withdrawal}$ is of minor influence on the total water exchange volumes and can be neglected.

Table 6.6 gives an overview of the various water exchange mechanisms. The total water exchange volume at the proposed harbour entrance can be obtained by summarising its distinct influences

Table 6.6 Total water exchange volumes

Water exchange mechanism	Only LNG terminal fully operational	All terminals fully operational
V_t	$8.7*10^6$ m^3/tide	$6.9*10^6$ m^3/tide
$V_{entrainment,cross}$	-	-
$V_{entrainment,long.}$	$2\text{-}5*10^7$ m^3/tide	$3\text{-}6*10^7$ m^3/tide
V_d	-	-
$V_{withdrawal}$	Negligible	Negligible
V_{total}	$3\text{-}6*10^7$ m^3/tide	$4\text{-}7*10^7$ m^3/tide

In chapter 8, a sensitivity analysis is carried out to get an impression of the main factors that determine the annual deposit rate of the proposed Hazira harbour basin. So far, it can be concluded that the water exchange due to entrainment by longitudinal flow effects is largely dependent on which location is chosen for comparison between the Hazira and the IJmuiden current velocities.

7 Sediment trapping processes

7.1 Introduction

This chapter discusses the behaviour of the suspended sediment that enters the Hazira harbour basin. The suspended sediment influx per tide has been discussed in the preceding. This sediment influx consists of the tidal mean sediment concentration and the total volume of water exchange per tide. When the settling factor of the incoming suspended sediment is known, the sedimentation of the suspended sediment in the Hazira harbour basin can be determined.

This settling factor depends on several aspects and processes. Section 7.2 describes the fall velocity of a single sediment particle. Section 7.3 deals with sediment-flow interactions and section 7.4 describes processes concerning secondary eddies and secondary flow. Finally, the settling factor of the Hazira harbour basin is discussed in section 7.5 and the conclusions are given in section 7.6.

7.2 Particle fall velocity

The sediment, which is transported around the Hazira harbour, can be divided into two types. The non-cohesive bed material, i.e. sand, dominates the morphological behaviour of areas under the influence of waves and currents. The cohesive and partly organic fine material, i.e. clay and silt, is dominant in the more sheltered areas like the harbour basin. Fine materials can only settle in quiet areas because their settling rate is relatively small. Most sand does not reach the harbour basin because most grains have already settled. The relative sand content of the deposits is estimated based on experience with existing harbour basins. In most cases, 5% is a realistic estimate (Eysink, 1989). This factor is also used for the proposed Hazira harbour basin.

From the graphs of the suspended sediment sieve curves (see appendix H), it follows that there is a relatively large amount of fine sediment particles recognisable. Based on experience and values found in literature, the fall velocity w_s of a single fine particle (floc) is assumed to be $1*10^{-4}$-$5*10^{-4}$ m/s.

Richardson and Zaki (1954) have investigated the influence of other particles on the fall velocity of a single particle. Experiments have shown that the fall velocity of a single particle is strongly reduced when the sediment concentration is large. This effect is called hindered settling and is largely caused by the upward flow of the fluid induced by displacement of the particles.

Richardson and Zaki (1954) have stated that the fall velocity in a fluid-sediment suspension can be determined by the following formula:

$$w_{s,m} = w_s \left(1 - \frac{c}{c_{gel}}\right)^{\beta} \qquad (7.1)$$

It generally holds that the exponent β is about 5 for fine-grained sediment. c_{gel} represents the "gelling concentration". This is the concentration at which a space-filling network is formed due to flocculation. A value of 60 g/l is adopted as gelling concentration. Equation (7.1) now becomes:

$$w_{s,m} = w_s \left(1 - \frac{c}{60}\right)^5 \tag{7.2}$$

where $w_{s,m}$ is the particle fall velocity in a suspension and c is the concentration.

Filling in the yearly mean sediment concentration of 1000 mg/l yields a reduction of the particle fall velocity in a still and clear fluid by 8 percent. The particle fall velocity in a suspension remains $1*10^{-4}$ to $5*10^{-4}$ m/s.

7.3 Sediment-flow interactions

Large amounts of suspended solids are observed in the area around Hazira Winterwerp, Uittenbogaard and de Kok (2001) have analysed conditions at which rapid siltation from saturated mud suspensions in navigation channels and harbour basins takes place. The maximum suspended sediment concentration that can be carried by turbulent flow is called the saturation concentration. Teisson has been the first to write about the existence of a saturation concentration for fine cohesive sediment. When the sediment concentration exceeds the saturation value, a complete collapse of the turbulence and the concentration profile occurs. A high-concentrated fluid mud layer is formed near the seabed with $c_{max} = c_{gel}$. According to Winterwerp et al. (2001), the depth-averaged saturation concentration C_s is defined by:

$$C_s = K_s \frac{\rho}{\Delta g} \frac{u_*^3}{h w_{s,m}} \tag{7.3}$$

where K_s is a proportionality coefficient, Δ is the relative sediment density and u_* the shear velocity.

As described in chapter 5, the present sediment concentration profile at Hazira is relatively constant over the entire water column. Only during or just after slack water, the suspended sediment concentration increases near the bed and decreases near the estuary's surface due to the settlement of sediment particles. It is hypothesised that a thin layer with a high concentration is formed at the bed of the Gulf of Khambhat. This layer is entrained rapidly when the tidal velocity increases again.

When the water enters the Hazira harbour basin, the flow velocity decreases rapidly. It is expected that the water within the basin is no longer able to keep the sediment into suspension and a super-saturated condition arises. A layer of high-concentrated fluid mud is formed near the seabed. As long as no turbulence can be generated at the water mud interface, the sediment does not come into suspension again.

The collapse of the concentration profile takes place at the same time scale as the time required for total settling, $T_s = \frac{h_0}{w_s}$. However in some circumstances, the water column is cleansed faster than is expected on the basis of this settling time. It is anticipated that the collapse of the concentration profile with the formation of the high-concentrated fluid mud layer can cause a sediment driven density current. That density current is responsible for the rapid cleaning of the water column.

As described in section 6.5, the proportional density current velocity can be given by:

$$u_{do} = 0.45\sqrt{\frac{\Delta\rho}{\rho} gh} \tag{7.4}$$

where u_{do} is the velocity of the density current without tidal influence.

Looking at the Hazira harbour entrance, a maximum density difference of about 1000 mg/l is assumed. This results in a density current of 0.1-0.2 m/s. In Figure 7.1 the situation at the harbour basin entrance is schematised at two different times.

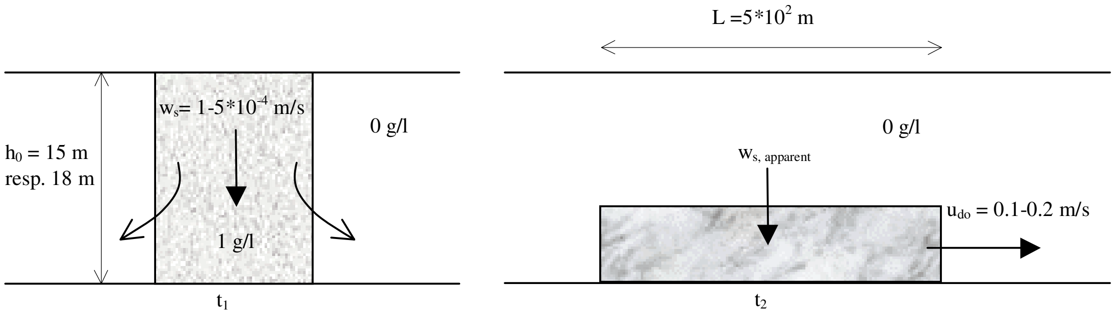


Figure 7.1 Collapse of concentration profile

At time t_1, the water column enters the Hazira harbour basin. There, the flow velocity is much smaller than outside and a collapse of the concentration profile is expected. This collapse has taken place at time t_2. A layer of fluid mud has been formed and a density current has arisen. Due to the density current, the cleansing of the water column goes much faster than is expected on the basis of the settling time. As a consequence, an apparent settling velocity appears. This apparent settling velocity is greater than the settling velocity of a single fine-grained floc particle.

In Figure 7.1, it is assumed that the collapse of the concentration profile has a length scale of about the width of the entrance, i.e. $5*10^2$ metres. All the other parameters are obtained from the Hazira harbour project. When there is no exchange between the water column and the seabed, the mass balance per unit metre width yields:

$$w_{s,apparent} L = u_{d0} h_0 \tag{7.5}$$

When only the LNG terminal is fully operational the mean depth of the Hazira harbour entrance equals 15 metres. The apparent fall velocity becomes:

$$w_{s,apparent} * 5 * 10^2 = 0.1 \text{ à } 0.2 * 15$$
$$w_{s,apparent} = 0.003 - 0.006 m/s \qquad (7.6)$$

When all the terminals are operational:

$$w_{s,apparent} * 5 * 10^2 = 0.1 \text{ à } 0.2 * 18$$
$$w_{s,apparent} = 0.004 - 0.007 m/s \qquad (7.7)$$

For both situations hold that the apparent fall velocity is much greater than the particle fall velocity, which is around $1*10^{-4}$ to $5*10^{-4}$ m/s. In formula:

$$w_{s,apparent} >> w_s \qquad (7.8)$$

As discussed later on this report, the fall velocity as obtained by Svašek by trial and error confirms the great apparent fall velocities.

Thus, the main conclusion of this section can be drawn:

It is hypothesised that the concentration profile collapses when the water enters the Hazira harbour basin and a layer of fluid mud is formed in the basin. This may lead to the development of a sediment driven density current. The interaction between the collapse of the concentration profile and the density current that is generated by this collapse leads to rapid siltation in the Hazira harbour basin.

7.4 Secondary effects

More than one eddy can be present in a harbour basin. The velocity pattern and the number of eddies in a harbour basin depend strongly on the geometry of the basin. Langendoen (1992) has summarised the results of the experiments done by Dursthoff and Booij. They have concluded that more than one eddy occurs in a basin with a length to width ratio (L/B) greater than two. With a length to width ratio in between one half and two, only one eddy is present in the basin. Furthermore, Booij has observed that the length of the primary eddy does not become much larger than 1.5 times the width of the basin for harbours with L/B>2. Measurements show that the flow velocity of the eddy has its maximum near the downstream sidewall of the harbour basin and decreases in the flow direction of the eddy. This is presented in Figure 7.2.

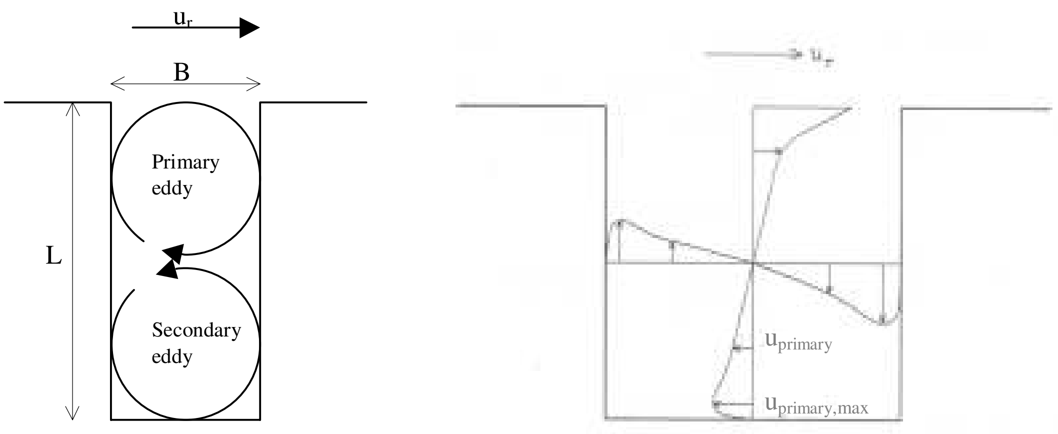


Figure 7.2 Eddies in a harbour basin and the velocity profile of the primary eddy

For square harbours with an entrance width similar to the basin width holds that the average maximum velocity of the primary eddy $\bar{u}_{primary,\max}$ is about 33 percent of the velocity of the river u_r. The average velocity of the primary eddy $\bar{u}_{primary}$ is about 22 percent of the river velocity. For harbours with a smaller entrance width than the basin width, the velocities in the eddy are proportionally smaller.

Sometimes, it is claimed that the transfer of momentum from the primary to the secondary eddy is the same as the transfer of momentum from the river (or estuary) to the primary eddy. If this is true then the following formulas can be established:

$$\bar{u}_{\text{secondary, max}} = 0.1\, u_r$$

$$\bar{u}_{\text{secondary}} = 0.04\, u_r \qquad (7.9)$$

There is a secondary flow recognisable in the eddy. Near the surface, the flow in the eddy is directed towards the edge of the eddy. This is due to the centrifugal forces. At the bed the flow is in opposite direction, i.e. towards the centre of the eddy. This phenomenon is comparable with the situation that arises while stirring in a cup of tea. In Figure 7.3 these torridoial movements are sketched.

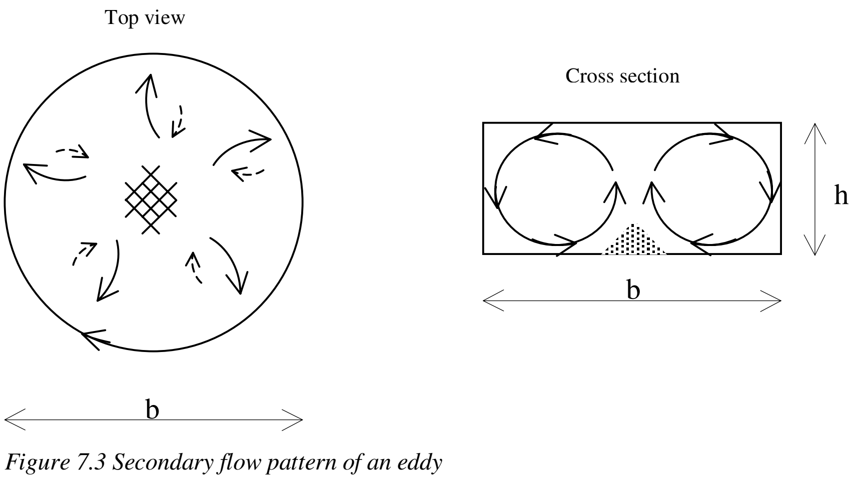


Figure 7.3 Secondary flow pattern of an eddy

The secondary flow is important for the sediment transport in a harbour basin. Sediment particles are transported by the curved flow towards the centre of the eddy. Here, the flow is rather small and often the sediment particles cannot be kept in suspension and settle. For this reason, most of the tea leaves when stirring in a cup of tea are collected in the centre of the glass.

As stated in section 6.4.3 a lot of similarities can be found between the IJmuiden harbour and the Hazira harbour. Their entrance orientation and their velocity pattern show great resemblance. However, there are some differences in size and geometry between both harbours. At Hazira, the harbour entrance is much smaller than the basin width. As mentioned before, an eddy is anticipated at this entrance. The circulation of this entrance eddy is expected to drive an additional secondary eddy in the Hazira basin. There is also an eddy recognisable in the entrance of the IJmuiden harbour. The length of the IJmuiden harbour is large in comparison to its width. As L/B>2, more than one eddy is expected to occur in the harbour basin of IJmuiden.

Secondary eddies can be of great influence on the sediment transport. The several eddies transport the sediment particles further into the harbour basin where the particles finally settle. As a result of these secondary effects, the sediment particles are probably transported further into the IJmuiden harbour basin than into the Hazira harbour basin. A piori it is hard to say how many eddies develop in the IJmuiden harbour basin. A possible flow pattern is given in Figure 7.4.

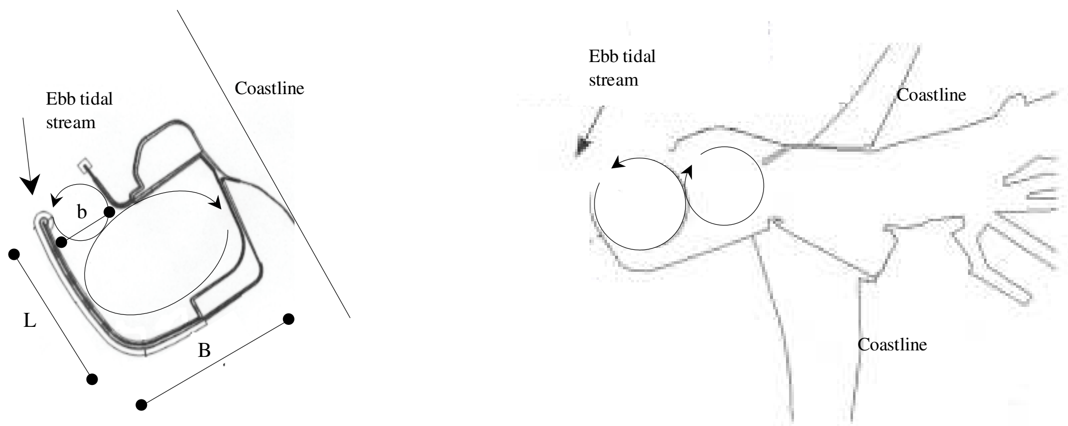


Figure 7.4 Flow pattern Hazira harbour and possible flow pattern IJmuiden harbour

In the next section, the trapping coefficient, or the settling factor, of the Hazira harbour basin is determined. A comparison is made with the harbour at IJmuiden and additional siltation caused by secondary eddies is anticipated. The rapid siltation that is expected in the basin at Hazira results from the interaction between the concentration profile and the flow. This is revealed by means of the apparent velocity, which is much greater than the single particle fall velocity.

7.5 Settling factor

The part of penetrating suspended sediment that settles in the harbour basin depends on the ratio of the time required for total settling (the settling time T_s) and the time available, the retention time T_r. Eysink (1989) gives an expression for the sedimentation factor p, see equation (7.10):

$$p = 1 - \exp\left(-\frac{1}{T_s} \left(1 - \frac{u^2}{u_c^2} \right)_{basin} T_r \right) \qquad (7.10)$$

in which u is the flow velocity inside the harbour basin. The index c reflects the critical value below which sedimentation occurs. It is obvious that p can never exceed one.

The factor $(1-u^2/u^2_c)$ in equation (7.10) has been introduced because the fall velocity of a particle may be reduced by mixing in the harbour basin. The resulting velocity is called the effective fall velocity (Krone, 1962). Reduction of the fall velocity leads to a reduction of the settling factor. Hence, the settling factor reaches its maximum value when there is no reduction. Equation (7.10) then becomes:

$$p = 1 - \exp\left(-\frac{T_r}{T_s}\right) \tag{7.11}$$

7.5.1 The retention time

Headland (1994) has stated that the retention time can be reflected by the time required to replace the average harbour volume Vol_{basin0} at the tidally averaged exchange rate. See Table 7.1.

Table 7.1 Determination of retention time, primary effects

	Only LNG terminal fully operational	All terminals fully operational
b_{car}	465	465 m
h_0	15 m	18 m
A_{basin}	$190*10^4$ m^2	$150*10^4$ m^2
Vol_{basin0}	$29*10^6$ m^3	$28*10^6$ m^3
V_{total}	3-6$*10^7$ m^3/tide	4-7$*10^7$ m^3/tide
T_r	$Vol_{basin0}/V_{total} \approx$ 6-11 hr	$Vol_{basin0}/V_{total} \approx$ 5-10 hr

From Table 7.1 follows a primary retention time of 6 to 11 hours for the LNG port and 5 to 10 hours for the LNG, bulk and container port.

The actual retention time of the water in the Hazira harbour basin is probably greater than is assumed in Table 7.1. The values as shown in Table 7.1 form the lower boundary of the retention time, because the effects of the expected secondary eddy are not included. This secondary eddy may transport sediment particles further into the basin, which leads to a longer retention time.

7.5.2 The settling time

As stated earlier, the time required for total settling can be described by:

$$T_s = \frac{h_0}{w_s} \tag{7.12}$$

where h_0 represents the mean depth and w_s the effective fall velocity of the fine-grained sediment particles.

When the water enters the Hazira harbour basin, a collapse of the concentration profile is expected and a layer of fluid mud is formed near the seabed. The interaction of the collapse of concentration profile and the development of a sediment driven density currents lead to an apparent fall velocity that is much greater than the fall velocity of a single fine-grained sediment particle. The apparent fall velocity and the fall velocity of a single particle determine the lower and the upper limit of the settling time respectively. When the settling time is calculated on the basis of the particle fall velocity, a multitude of ten hours is required for total settling. However, when applying the apparent fall velocity, the time required for total settling decreases to about one hour.

7.5.3 Results concerning the settling factor

In section 7.2, it is assumed that five percent of the inflowing suspended sediment consists of sand particles. It is expected that all sand settles in the basin, hence $p_{sand} = 1$. The part of the incoming fine suspended sediment that is trapped in the basin each tide is determined by using equation (7.11):

$$p_{fines} = 1 - \exp\left(-\frac{T_r}{T_s}\right) \qquad (7.13)$$

Applying a settling time on the basis of the apparent fall velocity and a retention time as a result of secondary effects, a settling factor of one is obtained. All incoming fine-grained sediment settles in the basin. Hence:

$$p_{fines,upperboundary} = 1 \qquad (7.14)$$

The lower boundary of the settling factor for fine-grained sediments is obtained by neglecting secondary effects and applying the single particle fall velocity. Great differences in this lower boundary of the settling factor is found due to the uncertainties in the single particle fall velocity, which ranges in between 1 and $5*10^{-4}$ m/s. With an adopted fall velocity of $1*10^{-4}$ m/s, the settling factor is minimal and equals 0.1. Additional research on the single fine-grained particle diameter is strongly recommended in order to get a more detailed insight in the particle fall velocity and hence, a more accurate assumption of the lower boundary of the settling factor.

Probably, a realistic estimation for the total settling factor in the Hazira harbour basin is somewhere in between the upper and the lower boundary of the settling factor for fine-grained materials. The influence of sand on the total settling factor is expected to be small because it is assumed that only five percent of the inflowing suspended sediment consists of sand. In this thesis, a total settling factor that varies from 30 to 50 percent is considered. Literature about trapping efficiencies in other harbours confirms that these values are a reasonable assumption. Hence,

$$0.3 < p_{total} < 0.5 \qquad (7.15)$$

7.5.4 Siltation of the IJmuiden harbour basin

In this section, the siltation in the IJmuiden harbour basin is discussed. Besides water exchange measurements, the sediment concentration has been measured at the IJmuiden harbour entrance. Most sediment enters the basin during maximum flood and to less extent during maximum ebb. A decrease of sediment concentration is recognisable during slack tide. The observed suspended solids concentrations at IJmuiden are much lower than at Hazira. During ebb the concentrations are about 60 mg/l and 40 mg/l for water that enters and leaves the basin respectively. With flood, concentrations of about 100 mg/l for incoming water and 35 mg/l for outgoing water can be found.

Rijkswaterstaat (the Dutch Directorate-General for public works and water management) has calculated the siltation in the IJmuiden harbour basin by multiplying the measured sediment concentration with the measured water exchange per tide through the entrance. The measurements have been carried out during the first three days of September 1981. On the first measuring day, the siltation of the IJmuiden harbour basin equals $6.7*10^2$ ton. The incoming sediment is around 3000 ton per tide that day. This results in a settling factor of about 22 percent. Unfortunately, the siltation rate of the other two days is not available because sediment has been dumped just outside the basin, which disturbs the natural sedimentation rates.

The measured siltation rate is compared with the dredging rate per year. In the IJmuiden harbour basin, a yearly amount of $3.0*10^6$-$3.5*10^6$ m^3 of sediment is dredged. This corresponds with an amount of 4000-5000 m^3 of sediment each tide. Rijkswaterstaat assumes a dry bulk density of 200 kg/m^3, which results in a siltation of 800-1000 ton per tide.

As discussed in section 7.4, it is expected that the secondary effects are greater at the IJmuiden harbour basin than at the Hazira harbour basin. On the basis of secondary effects, the trapping coefficient at IJmuiden is expected to be greater than at Hazira. This does not result from the calculated settling factor at IJmuiden of 22 percent. The depth of the IJmuiden harbour entrance is smaller than at Hazira and the retention time of the basin is probably greater due to the larger harbour basin volume and more secondary effects. These aspects result in a greater settling factor (see equation (7.11)).

A reason for the relatively low settling factor in the IJmuiden harbour basin may be a smaller particle fall velocity than at Hazira. Probably no collapse of the concentration profile takes place at IJmuiden. The relatively small difference of the observed suspended solids concentrations for water that enters and leaves the IJmuiden harbour basin confirms this assumption. Unfortunately, the suspended sediment particle diameters are not given in the report (Rakhorst, 1982) presented by Rijkswaterstaat. As long as this particle diameter is not known, a reliable comparison between the settling factor at IJmuiden and the settling factor of the suspended sediment at Hazira is not possible.

7.6 Conclusions

In the preceding the behaviour of suspended sediment that enters the Hazira harbour basin has been analysed. Water with very high concentrations of suspended sediment enters the basin. The greatest part of the inflowing suspended sediment consists of fine-grained material.

When the water has entered the Hazira harbour basin, the flow velocity decreases rapidly and it is expected that the water within the basin is no longer able to keep the sediment into suspension. As a result a collapse of concentration profile takes place and a layer of fluid mud is formed in the basin. The collapse of the concentration profile is expected to generate a sediment driven density current. This interaction can lead to rapid siltation in the harbour basin at Hazira.

An eddy, which is generated by entrainment due to longitudinal flow effects, is anticipated in the Hazira harbour entrance. The circulation of this entrance eddy probably drives a secondary eddy in the basin. Secondary eddies can be of great influence to the sediment transport. They can transport suspended sediment particles further into the basin where the particles may settle.

The settling factor depends on the ratio of the time required for total settling and the time available for settling, the retention time. The processes mentioned above are expected to lead to a greater settling factor in the Hazira harbour basin than expected on the basis of the single particle fall velocity, which determines the settling time, and the retention time due to primary effects.

It is expected that all incoming sand settles in the basin, hence the settling factor of sand equals one. The settling factor of fine-grained materials shows a relatively wide range. Especially, the lower boundary of that settling factor has not been clearly defined yet. In this thesis, a total settling factor in between 30 and 50 percent is considered. Further research on the sediment particle diameters is strongly recommended to obtain a more accurate estimate of the settling factor that is expected at the Hazira harbour basin.

Another aspect is the accumulation of sediment particles in the centre of the eddy. The flow in an eddy is highly curved. Small particles spiral towards the centre where they settle due to the small velocity near the seabed.

8 Siltation in the Hazira harbour basin

8.1 Introduction

In this chapter, the various processes that have been dealt with in the preceding are synthesised. Combination of the expected mean suspended sediment concentration (chapter 5), the water exchange mechanisms (chapter 6) and the sedimentation factor (chapter 7) results in the yearly sedimentation of suspended sediment (in kilograms) in the basin. Finally, the total volume of suspended sediment that settles in the basin is determined.

From the beginning of this research, a mathematical program called SILTHAR, used for the computation of siltation in harbour basins, has served as a guideline. It is discussed in section 8.2. The annual siltation rate of the planned Hazira harbour basin is dealt with in section 8.3. Section 8.4 discusses the estimated maintenance dredging volumes and describes measures that can be taken to combat and prevent high siltation rates. Finally, in section 8.5, a sensitivity analysis is carried out to get an impression of the main factors that determine the annual deposit rate.

8.2 The siltation model SILTHAR

The mathematical model SILTHAR predicts the siltation in a semi-enclosed harbour basin. It is a relatively simple semi-empirical model and its name is an acronym of SILTation of HARbour basins. SILTHAR has been developed by Delft Hydraulics to predict the effect of measures, like harbour extensions, on the siltation rate. The siltation rate is an important figure for the economic feasibility of a harbour.

SILTHAR determines the siltation rate in a semi-enclosed harbour basin through three quantities: the sediment influx per tide T_s, the sedimentation factor p and the dry bulk density of the deposit ρ_d. This relationship is given in equation (8.1)

$$\Delta T_s = \frac{p * T_s}{\rho_d} \qquad (8.1)$$

where

ΔT_s	=	sedimentation of suspended sediment in the basin per tide [m^3/tide]
T_s	=	sediment influx per tide [kg/tide]
p	=	part of the suspended sediment that settles in the basin [-]
ρ_d	=	dry bulk density of the deposits [kg/ m^3]

The overall structure of the SILTHAR model is given in Figure 8.1.

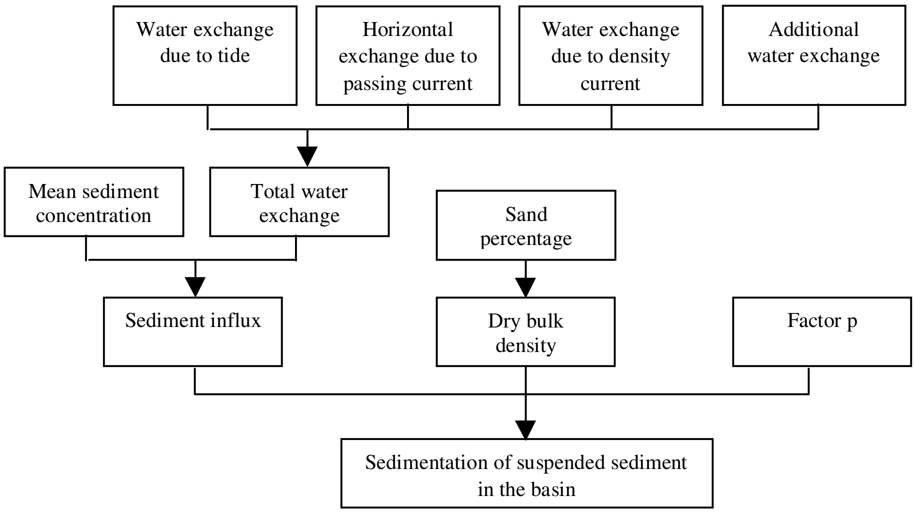


Figure 8.1 The structure of the SILTHAR model

The sediment influx per tide is the product of the tidal mean sediment concentration of the penetrating water and the total volume of water exchange per tide. In SILTHAR, this total water exchange volume is the sum of the tidal prism, the exchange due to a horizontal current passing the harbour entrance, the exchange by density currents and the additional exchange due to flushing or withdrawal. See equations below:

$$T_s = cV_{total} \tag{8.2}$$

with:

$$V_{total} = V_t + V_h + V_d + V_{add} \tag{8.3}$$

where:

c	=	mean sediment concentration of the penetrating water [kg/m^3]
V_{total}	=	total volume of water exchange [m^3/tide]
V_t	=	water exchange due to the tide [m^3/tide]
V_h	=	water exchange due to the horizontal flow along the entrance [m^3/tide]
V_d	=	water exchange due to density currents [m^3/tide]
V_{add}	=	additional water exchange due to flushing/withdrawal [m^3/tide]

All the mentioned water exchange mechanisms have been described in chapter 6. SILTHAR does not deal with the water exchange due to entrainment by longitudinal flow effects.

8.3 Siltation in the Hazira harbour basin

As stated in section 2.5, the main objective of this study is to provide a reliable estimate of the annual siltation rate in the proposed Hazira harbour basin. In this section, the various parameters concerning the siltation of the Hazira harbour basin are summarised and combined to fulfil this objective.

For determination of the annual deposit rate in the Hazira harbour basin, the same approach as used in the SILTHAR program is followed. Initially, the planned port only accommodates the

LNG berth. However in the future, apart from the LNG berth in the North, the harbour includes general cargo and bulk berths as well as a container terminal. In the present section, the siltation in the Hazira harbour basin is discussed for a layout in which only the LNG terminal is operational, as well as for the ‘full port layout’ in which all terminals are fully operational.

8.3.1 Sediment influx

Multiplication of the yearly mean suspended sediment concentration and the total volume of water exchange per tide reflects the average sediment influx per tide T_s (see equation (8.2)).

The assumed annual mean suspended sediment concentration in the Gulf of Khambhat is based on measurements performed by Fugro Geos. In December 2000 and April 2001, Fugro has related turbidity to the amount of suspended solids by using reference samples. Furthermore, water samples have been collected each month from June 2000 until September 2001. These samples have been sent to England for analysis. Unfortunately, they only give a rough indication of the mean sediment concentration because the outcome is largely dependent on when the water samples are taken. Great differences in magnitude of the amount of suspended solids during neap and spring tide and during high and low water can be found.

From the data obtained by Fugro Geos, it follows that the sediment concentration in the Gulf of Khambhat is very large. Especially during the monsoon period high concentrations are expected. Concentrations up to about 2300 mg/l have been observed. As stated in section 5.5, a yearly mean sediment concentration of 1000 mg/l is assumed for the Hazira harbour basin.

In the present study, the total water exchange volume is the sum of the tidal prism and the exchange volumes due to entrainment by longitudinal flow effects, which generate an entrance circulation.

Water exchange due to entrainment by cross flow effects is not present at the proposed Hazira harbour. At Hazira, the entrance is not situated along the breakwater. As a result no distinct separation of the flow occurs and no mixing layer develops.

No density induced water exchange is expected. There have been some problems with the salinity data as derived from the CT recorder. In the monthly data reports, fresh water incursions in the CT data are found at about 0.5 metres above seabed. However, further research by Fugro has provided evidence that the apparent reductions in salinity are not real. Probably, the high sediment content and insufficient flushing have contaminated the recorder.

As stated in section 6.6, the influence of the additional exchange caused by the water intake on the total water exchange volume can be neglected.

The tidal prism has been determined in section 6.2. The mean tidal prism can be defined by the water volume between Mean Low Water (MLW) and Mean High Water (MHW) in the harbour basin. The tidal prism equals $8.7*10^6$ m^3 per tide for the harbour layout with only a LNG terminal and $6.9*10^6$ m^3 per tide for the ’full port layout’. The wet surface area of ‘the full port layout’ is smaller when all berths are present.

The water exchange due to entrainment by longitudinal flow effects takes care of the greatest part of the total water exchange volume at Hazira. For a quantitative interpretation of this exchange mechanism, a comparison is made with the present harbour at IJmuiden in the Netherlands. A lot of similarities between the planned Hazira harbour and the existing harbour at IJmuiden can be found. In section 6.4 is found that the water exchange caused by entrainment due to longitudinal flow effects is proportional to a characteristic velocity ratio of the proposed Hazira harbour to the existing harbour of IJmuiden, the mean entrance depth and a characteristic width of the entrance.

The water exchange due to entrainment by longitudinal flow effects is largely influenced by the location that is chosen for comparison of the velocities at IJmuiden and at Hazira. If one looks at the anticipated impact of the harbour basin concerned, by comparison of the flood velocities just in front of the basin, the velocity ratio equals about one. However, when the 'undisturbed' maximum flood velocities during a mean tide are compared with each other, the velocity ratio of the proposed Hazira harbour to the existing harbour at IJmuiden is about two.

The foregoing results in a water exchange caused by entrainment due to longitudinal flow effects that is expected to vary from $2*10^7$ and $6*10^7$ m^3 per tide for, respectively, a layout in which only the LNG terminal is operational with a velocity ratio that equals one and for the 'full port layout' with a velocity ratio of about two.

Table 6.4 summarises the sediment influx per tide, T_s, for the different conditions. This sediment influx is equal to the product of the mean suspended sediment concentration and the total volume of water exchange.

Table 8.1 Sediment influx for the different velocity ratios

	$\left(\frac{u_{Hazira}}{u_{IJmuiden}}\right)_{disturbed} \approx 1$	$\left(\frac{u_{Hazira}}{u_{IJmuiden}}\right)_{undisturbed} \approx 2$
Only LNG terminal fully operational	$T_s = 3*10^7$ kg/tide	$T_s = 6*10^7$ kg/tide
All terminals fully operational	$T_s = 4*10^7$ kg/tide	$T_s = 7*10^7$ kg/tide

As results from Table 6.4, the sediment influx per tide is expected to vary in between $3*10^7$ and $7*10^7$ kg/tide and is largely dependent on the adopted velocity ratio of the proposed Hazira harbour and the existing harbour at IJmuiden.

8.3.2 Settling factor

The part of the penetrating sediment that settles in the basin has been discussed in chapter 7. It depends on the ratio of the time required for total settling, i.e. the settling time, and the time available, i.e. the retention time. Most of the incoming suspended sediment consists of fine-grained particles.

In the proposed Hazira harbour basin, it is assumed that the settling factor is greater than expected on the basis of the single particle fall velocity, which determines the settling time, and the retention time due to primary effects. Two reasons can be mentioned for this expected increase in the settling factor. Firstly, a collapse of the concentration profile is hypothesised when the water enters the Hazira harbour basin and a fluid mud layer is formed near the seabed. This collapse may generate a sediment-driven density current. The interaction between the collapse of the concentration profile and the density current can lead to rapid siltation in the

Hazira harbour basin. Secondly, the expected secondary eddy in the basin can cause a greater trapping efficiency as well. Secondary eddies may transport suspended sediment particles further into the basin where the particles finally settle.

As stated in chapter 7, the total settling factor is assumed to vary in between 30 and 50 percent. Research on trapping efficiencies in other harbours confirms that these values are plausible.

8.3.3 Dry bulk density

The dry bulk density of the deposits depends on several factors, such as the type of sediment (sand or silt), sand and/or organic matter content of a mud deposit and the consolidation time after deposition (SILTHAR manual, 1995).

In the SILTHAR manual (1995), the dry bulk density is determined based on data of dredged materials of harbour basins of the Port of Rotterdam in the Netherlands. The following empirical relationship is used:

$$\rho_d = 350 + 1250 * p_s^2 \qquad (8.4)$$

in which p_s is the relative sand content of the deposit. See also Figure 8.2.

By assuming a relative sand content of 5 percent (see chapter 7), the dry bulk density of the deposit is about 350 kg/m^3. Assuming a seawater density ρ of 1025 kg/m^3 and a sediment density ρ_s of 2650 kg/m^3, this results in a wet density of about 1240 kg/m^3. See equation (8.5) (Van Rijn, 1993):

$$\rho_{wet} = \rho + \left(\frac{\rho_s - \rho}{\rho_s} \right) \rho_{dry} \qquad (8.5)$$

The wet density is the weight of the water and sediment mixture per unit volume. It is an important feature for research on the consolidation rate in the harbour.

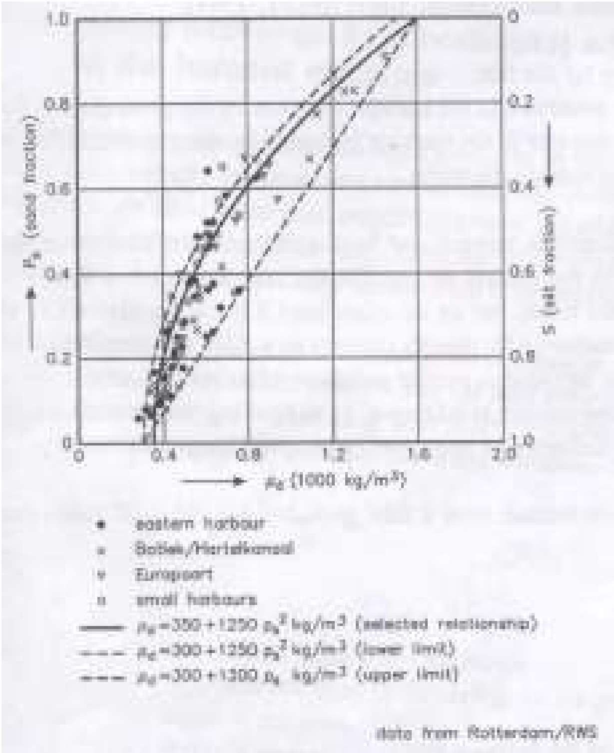


Figure 8.2 Dry bulk density of the deposits in relation to its relative sand content

8.3.4 Conclusions annual deposit rate

The preceding sections have dealt with the three quantities that determine the siltation of suspended sediment (in volume) in the basin per tide. The tide in the Gulf of Khambhat is semi-diurnal with a tidal period that equals 12 hours and 25 minutes. This results in about 706 tides a year. The annual deposit rate can be defined by:

$$\Delta T_s = n\left(\frac{p * T_s}{\rho_d}\right) \tag{8.6}$$

where ΔT_s is the sedimentation of suspended sediment (in volume) in the basin per year and n is the number of tides a year.

In Table 8.2, the expected annual siltation volumes for the proposed Hazira harbour basin are summarised by applying the results of the preceding sections. In this table, distinction is made between the settling factor and the velocity ratio for both layouts.

Table 8.2 Initial siltation volumes (m^3/year)

	Trapping efficiency	$\left(\frac{u_{Hazira}}{u_{IJmuiden}}\right)_{disturbed} \approx 1$	$\left(\frac{u_{Hazira}}{u_{IJmuiden}}\right)_{undisturbed} \approx 2$
Only LNG terminal fully operational	p = 0.30 p = 0.50	$\Delta T_s = 20*10^6$ m³/year $\Delta T_s = 30*10^6$ m³/year	$\Delta T_s = 35*10^6$ m³/year $\Delta T_s = 55*10^6$ m³/year
All terminals fully operational	p = 0.30 p = 0.50	$\Delta T_s = 20*10^6$ m³/year $\Delta T_s = 35*10^6$ m³/year	$\Delta T_s = 40*10^6$ m³/year $\Delta T_s = 65*10^6$ m³/year

From Table 8.2, it follows that the expected amount of sedimentation varies from 20 to 65 m^3/year.

The estimated maintenance dredging volumes apply for the case that the design depth of the basin is reduced with a maximum of approximately 10 to 15 percent as the deposit rate decreases when the cross sectional area of the entrance reduces.

The storage area of the Hazira harbour basin amounts to $1.9*10^6$ m^2 when only the LNG terminal is present. When also the other berths are present the wet surface area decreases to $1.5*10^6$ m^2. Table 8.3 shows the initial deposit rates in metres when sediment particles are assumed to settle equally over the entire basin and when the dry bulk density is assumed to be equal to 350 kg/m^3.

Table 8.3 Initial siltation rate in metres per year, with ρ_d= 350 kg/m^3

	Trapping efficiency	$\left(\frac{u_{Hazira}}{u_{IJmuiden}}\right)_{disturbed} \approx 1$	$\left(\frac{u_{Hazira}}{u_{IJmuiden}}\right)_{undisturbed} \approx 2$
Only LNG terminal fully operational	p = 0.30	10 m/year	18 m/year
	p = 0.50	17 m/year	29 m/year
All terminals fully operational	p = 0.30	14 m/year	26 m/year
	p = 0.50	24 m/year	43 m/year

The annual siltation largely depends on the location chosen for comparison of the Hazira and IJmuiden current velocities. It is hypothesised that a velocity ratio of one reflects the lower boundary of the annual deposit rate, whereas a velocity ratio of two can be seen as the upper boundary. The sensitivity of the different parameters is discussed in section 8.5. This sensitivity analysis is carried out to get an impression of the main factors determining the annual deposit rate.

The assumed deposit rates in the ‘full port layout’ are somewhat larger than the rates that are expected when only the LNG terminal is present. This difference is caused by an increase in the mean entrance depth from 15 to 18 metres, which results in an increase in water exchange by entrainment due to longitudinal flow effects for the ‘full port layout’.

Sediment particles do not settle equally over the bed of the harbour basin. Due to secondary effects, the water exchange resulting from the eddy may affect the whole basin, while the tidal water exchange is restricted to the area near the entrance. Steep bed slopes may develop at the entrance of the Hazira harbour basin. Next to sediment induced density currents, this may lead to a natural flow when the slope exceeds its maximum value. As a result, the sediment particles spread out on the seabed.

8.4 Maintenance dredging of the basin

In order to maintain the required depth in the harbour basin by dredging, the quantity of mainly fine-grained materials taken out must be equal to the quantity deposited into it. As stated in the preceding section, the assumed sedimentation of mainly fine-grained materials in the Hazira harbour basin is very large. A multitude of ten million cubic metres of mainly fine-grained sediment is expected to settle on the bed of the basin each year.

Based on the present siltation study, rather high siltation volumes are expected in the proposed Hazira harbour. It is recommended to review the options changing the harbour layout. By changing the orientation of the harbour, great progress with regard to the accretion problems can be obtained.

When an alternative layout is not desired, options for improvement of the present situation, which follows from the present layout, are considered. A number of mitigating measures is mentioned below.

Construction of a lock

There may be thought of construction of a lock in the Southern part of the main breakwater. By opening the lock during rising tide, a strong flood current, which is directed to the Northwest, is present inside the harbour basin. This current can bring settled sediments into suspension again and transport them, together with the rest of the suspended sediment, towards the area outside the harbour basin. There, it is expected that the suspended sediments are swept further to the North due to the strong flood currents, which are present in the Gulf. Research on safe manoeuvrability of the ships in the basin has to be carried out.

Water injection

Even with revision of the harbour layout or construction of a lock in the main breakwater, maintenance dredging of the basin is required. The most effective dredging method may be the use of the water injection technique. A water injection dredger uses water pressure to fluidise the bed material that has to be removed. By means of centrifugal pump induced water jets, a dense fluid slurry is created. This slurry is transported from the harbour basin area in the form of a density current. The density induced flow is generated by the difference in density between the slurry and the water. See Figure 8.3.

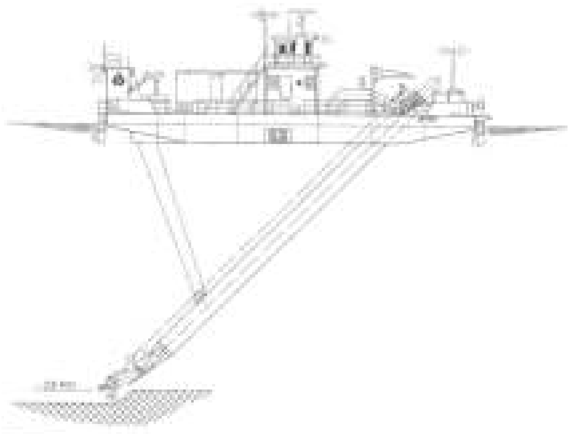


Figure 8.3 Water injection dredger

Outside the Hazira harbour basin, these fine-grained sediments can be swept further to the North of the Gulf of Khambhat by the relatively strong flood currents outside the basin. This relatively low-cost dredging technique is limited to silts, unconsolidated clays and fine sands.

Silt trap

Construction of a silt trap outside the harbour entrance may reduce the amount of incoming suspended sediment in the proposed Hazira harbour basin. The suspended sediment particles are expected to settle in the pit.

Nautical bottom in fluid mud areas

The top layer of the mud deposits on the seabed at Hazira probably has a low density and a weak shear strength. The question arises if, in this situation, save navigation is possible with restricted or even negative keel clearance with respect to the water-mud interface. Roovers (1988) defines the nautical bottom as "the horizontal layer, situated between the top of the mud layer and the hard bottom, above which a vessel can navigate and manoeuvre safely".

In 1983, a study on navigation in mud areas has been carried out by a workgroup, set up by the Permanent Technical Committee of PIANC. This study has resulted in a nautical bottom that corresponds with a density of 1200 kg/m^3 for the Europoort area in the Netherlands. Following equation (8.5), the accompanying dry sediment density yields 285 kg/m^3.

Kerckaert et al. (1988) have done research on determination of the nautical bottom at the port of Zeebrugge in Belgium. Application of three different techniques has resulted in a nautical bottom with a corresponding density value of at least 1150 kg/m^3.

At the same time, they have carried out full-scale navigation and manoeuvring tests with a trailing suction hopper dredger. The most important conclusion that can be drawn from these tests is that as soon as the keel penetrates the fluid mud, the vessel's resistance increases considerably. However, there is no obstruction for a vessel to pass through the top of the fluid mud layer. Tests have been carried out with a density of 1140 kg/m^3.

An important feature for the density of the nautical bottom is the consolidation rate of the mud deposits. Van Rijn (1993) describes consolidation as "a process of floc compaction under the influence of gravity forces with a simultaneous expulsion of pore water and gain in strength of the bed material". It is strongly affected by the initial thickness, the initial concentration and the permeability of the fluid mud layer. It generally holds that the consolidation process proceeds relatively slower in a thick layer than in a thin layer, because the pore water has to travel over a longer distance.

It is expected that the density of the fluid mud layer increases with its depth and with the time. However, the rate of which that process takes place, determines the thickness and density of the bed layer at a certain time and hence, the amount of maintenance dredging that has to take place.

Various consolidation stages can be distinguished in estuarine conditions. Table 8.4 reflects these stages with the accompanying values of the wet and dry sediment densities in estuarine conditions (Van Rijn, 1993).

Table 8.4 Density ranges of consolidated mud

Consolidation stage	Rheological behaviour	Wet sediment density [kg/m^3]	Dry sediment density [kg/m^3]
Freshly consolidated (~ 1 day)	Dilute fluid mud	1000-1050	0-100
Weakly consolidated (~ 1 week)	Fluid mud	1050-1150	100-250
Medium consolidated (~ 1 month)	Dense fluid mud	1150-1250	250-400
Highly consolidated (~ 1 year)	Fluid-solid	1250-1350	400-550
Stiff mud (~ 10 years)	Solid	1350-1400	550-650
Hard mud (~ 100 years)	Solid	>1400	>650

At Hazira, a dry bulk density of the deposits of about 350 kg/m^3 is assumed (section 8.3.3). From the table above, it follows that the settled material is expected to be dredged within a month, when this approach is applied.

8.5 Sensitivity analysis

In this section, a sensitivity analysis is carried out to get an impression of the main factors that determine the annual deposit rate of the proposed Hazira harbour basin. The starting point is the annual sedimentation rate as calculated in section 8.3. The two different port layouts, each with different scenarios, are evaluated. For both layouts the sedimentation rate is calculated for settling factors of 0.3 and 0.5 and velocity ratios of one and two.

Around the computed mean sedimentation parameters, a certain range is possible. This deviation may for example be the result of inaccuracies in the measuring methods or change of circumstances in the environment. Little changes of the mean value of a certain parameter may have great effects on the yearly sedimentation rate.

The influence of a certain parameter on the yearly sedimentation rate can be expressed by the sensitivity coefficient. This coefficient reflects the relative change in sedimentation rate by a relative change in one of the sedimentation parameters. See equation (8.7).

$$Sensitivity\ coefficient = \frac{\Delta\text{sedimentation rate} \big/ \text{computed sedimentation rate}}{\Delta\text{sed.parameter} \big/ \text{computed sed.parameter}} \qquad (8.7)$$

With insight in the sensitivity coefficients of the different parameters, more efficient research on the decisive variables can be done. The results of the sensitivity analysis are interpreted graphically in the next sections. The influence of the velocity ratio (section 8.5.1), the mean settling factor (section 8.5.2), the mean sediment concentration (section 8.5.3) and the dry bulk density on the annual deposit rate (section 8.5.4) is discussed.

A complete overview of the main parameters that influence the annual deposit rate is given in equation (8.8).

$$Annual\ deposit\ rate = n * \left(\frac{pcV_{total}}{\rho_d} \right) = n * \left(\frac{pc(V_t + V_{entrainment,long.}}{\rho_d} \right)$$

$$Annual\ deposit\ rate = n * \left(\frac{pc(\overset{\text{Section 6.2}}{\overset{\downarrow}{2\eta A_b}} + \overbrace{3.4 * 10^3 * \frac{u_{Hazira}}{u_{IJmuiden}} bh_0}^{\text{Section 6.4}}}{\rho_d} \right) \tag{8.8}$$

8.5.1 Sensitivity to the velocity ratio

As stated in section 6.4.3, the water exchange due to entrainment by longitudinal flow effects is proportional to the ratio of a characteristic velocity at Hazira to that of IJmuiden. This velocity ratio depends on the location that is chosen for comparison of the Hazira and IJmuiden current velocities.

In the undisturbed situation, when there is no impact of the harbours concerned, the relation of the maximum flood velocity during a mean tide at Hazira to the maximum flood velocity during a mean tide at IJmuiden equals about two. However, when the modelled flow velocities in the vicinity of the proposed harbour are compared with the measured flow velocities at IJmuiden, a velocity ratio of approximately one is obtained. In Figure 8.4, the sensitivity of the annual deposit rate to the adopted velocity ratio is shown.

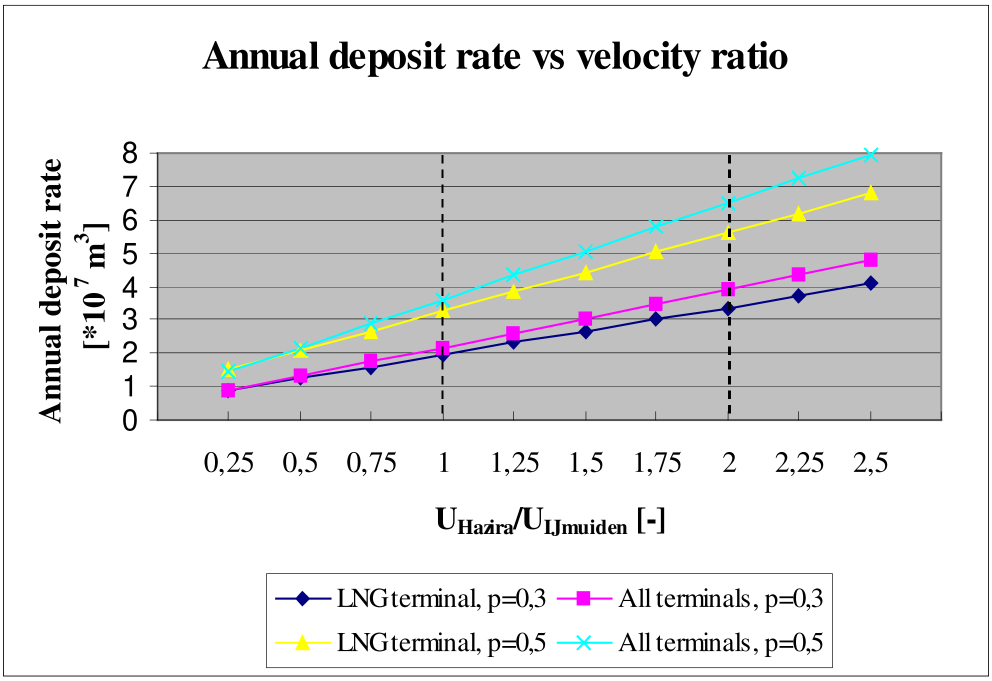


Figure 8.4 Sensitivity of the annual deposit rate to the velocity ratio

The accompanying sensitivity coefficient equals 0.8. Hence, a relative change of one in the velocity ratio equals a relative change in the annual deposit of 0.8.

8.5.2 Sensitivity to the settling factor

The settling factor is expected to vary in between 30 and 50 percent. However, additional research on the particle fall velocity is required in order to get a more accurate assumption of the settling factor, as a wide range around the anticipated settling factors can be found in the present thesis.

Figure 8.5 shows the sensitivity of the annual deposit rate to the settling factor. For both layouts the sedimentation rate is calculated for velocity ratios of one and two.

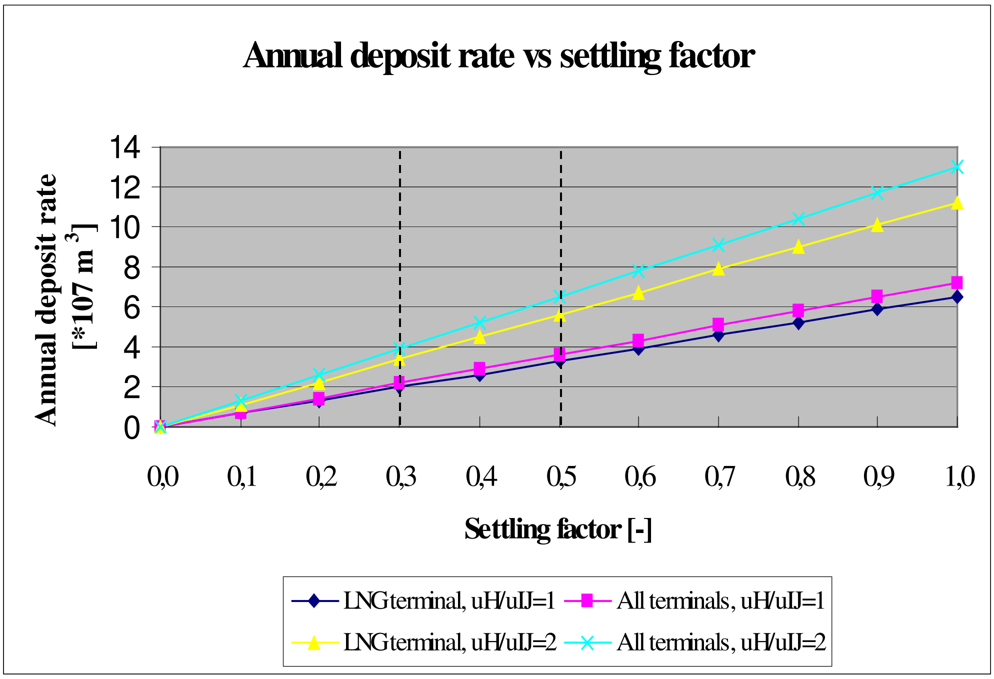


Figure 8.5 Sensitivity of the annual deposit rate to the settling factor

From equation (8.8) follows that annual deposit rate and the settling factor are proportionally related. This means that the sensitivity coefficient is one. A relative change of the settling factor provides the same relative change in the annual deposit rate.

8.5.3 Sensitivity to the sediment concentration

The yearly mean suspended sediment concentration at Hazira is estimated at 1 kg/m^3. The annual deposit rate and the mean sediment concentration are proportionally related (see equation (8.8)). As a result, the sensitivity coefficient equals one. The sensitivity of the annual deposit rate to the mean sediment concentration is reflected in Figure 8.6.

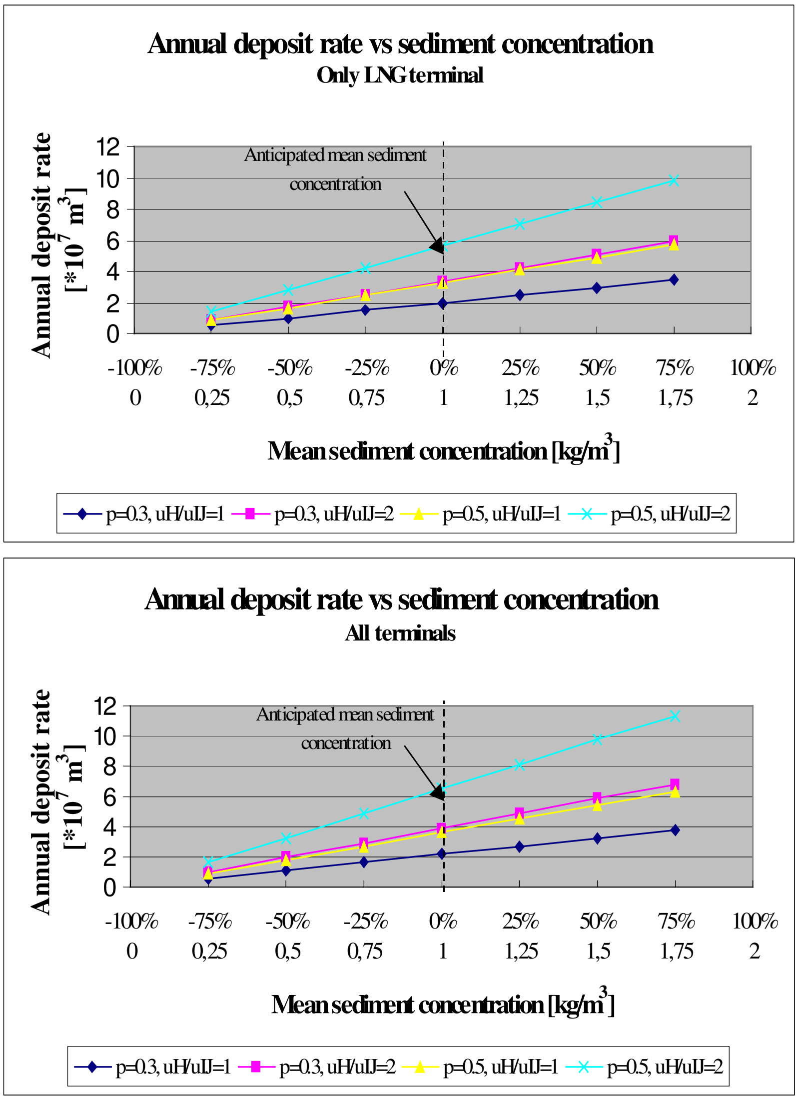
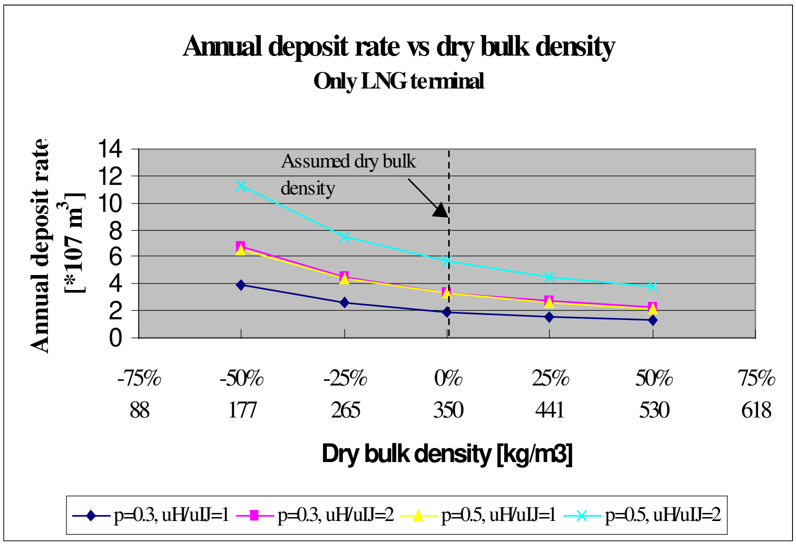


Figure 8.6 Sensitivity of the annual deposit rate to the mean sediment concentration

For both layouts the sedimentation rate is calculated for settling factors of 0.3 and 0.5 and velocity ratios of one and two. A relative change of the sediment concentration provides the same relative change in the annual deposit rate. As expected, the mean sediment concentration appears to be an important sediment parameter for determining the sedimentation in the proposed Hazira harbour basin. Unfortunately, the available data concerning the amount of suspended solids is poor. More suspended sediment data is required to provide a more accurate estimate of the mean suspended sediment concentration.

8.5.4 Sensitivity to the dry bulk density

Unlike the previously mentioned parameters, the dry bulk density of the deposits is not linearly related to the annual deposit rate. As a result, the sensitivity coefficient cannot be assumed constant. A decrease in dry bulk density of 50 percent to 177 kg/m^3, yields an increase in the annual deposit rate of 200 percent in relation to the calculated rate. On the other hand, if the dry bulk density increases with 50 percent to 530 kg/m^3, the annual deposit rate decreases with ‘only’ 70 percent. Figure 8.7 shows the negative and variable relation between the annual deposit rate and the dry bulk density of the deposits.



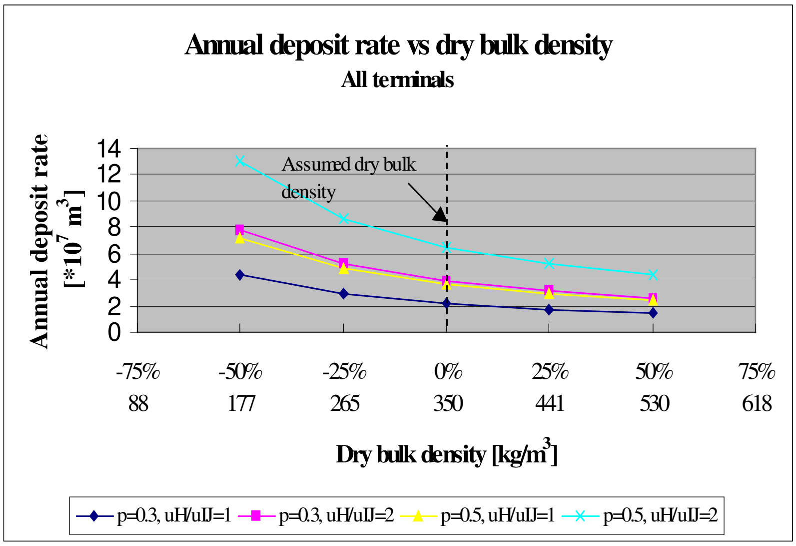


Figure 8.7 Sensitivity of the annual deposit rate to the dry bulk density

As follows from Figure 8.7, a certain decrease in the dry bulk density has larger consequences on the siltation rate in the basin than a similar increase in the dry bulk density. See also Table 8.5.

Table 8.5 Sensitivity dry bulk density

Variation	Sensitivity coefficient
+50%	-0.7
+25%	-0.8
-25%	-1.3
-50%	-2.0

8.5.5 Conclusions sensitivity analysis

The influence of the various sedimentation parameters can be evaluated by means of comparison of their sensitivity coefficients. The larger the absolute value of the sensitivity coefficient, the larger the influence of the parameter concerned. In Table 8.6, the sensitivity coefficients of the velocity ratio, the settling factor, the sediment concentration and the dry bulk density are summarised.

Table 8.6 Sensitivity coefficients

Sensitivity to	Sensitivity coefficient
Velocity ratio	0.8
Settling factor	1.0
Sediment concentration	1.0
Dry bulk density	See Table 8.5

From Table 8.6, it follows that the annual deposit rate is rather sensitive to all of the parameters that have been dealt with in this section. However, next to the sensitivity, the reliability of the calculated parameter needs to be discussed as well. It is expected that the velocity ratio is the most uncertain parameter. Nevertheless, the other parameters are also rather uncertain. Uncertainties in the calculated parameters may lead to large deviations of the calculated annual deposit rate as well. To decrease the range around the discussed sedimentation parameters, more research is required.

9 Discussion

9.1 Comparison with Svašek results

Svašek has carried out several studies concerning the siltation of the basin of the proposed Hazira harbour. On the basis of flow studies, the port layout has been optimised a couple of times. Two siltation studies have been carried out for a port layout that is fairly similar to the layout presented in appendix A. In the following sections, both studies are discussed.

Section 9.1.1 gives a description of the siltation study performed by Svašek in December 2000. A good comparison with the siltation results of the present study is possible because the structure of both sedimentation studies is almost the same. Section 9.1.2 deals with the latest siltation study as performed by Svašek in August 2001. Again, the basis for the siltation study is the current model FINEL, but this time a mathematical model is used to compute the sediment flux to the seabed.

9.1.1 Siltation study Svašek December 2000

In the siltation study performed by Svašek in December 2000, the yearly deposit rate has been calculated based on estimates of the water exchange volumes, the mean suspended sediment concentration, a siltation ratio and the dry bulk density.

Like in the present study, the total volume of water exchange is expected to result from the tidal prism and from the circulation at the entrance. Both water exchange mechanisms are discussed. Very little data on sediment transport and morphology was actually available from the area surrounding Hazira in December 2000. In this section, the several parameters are discussed, compared with the present study and reviewed.

Water exchange due to tidal prism

Svašek assumes a water exchange caused by the tide of $7.5*10^6$ m^3 per tide. The tidal prism in the present thesis equals $8.7*10^6$ m^3 per tide, when only the LNG terminal is fully operational, and 6.9 *106 m3 per tide for the ‘full port layout’ (see section 6.2).

The difference in tidal prism between both studies is easy to explain. Svašek assumes a wet surface area of the basin of $1.5*10^6$ m^2. This assumed storage area is based on a situation in which the LNG terminal is fully operational and the bulk terminal platform has already been constructed. In the present study, two situations are compared: (1) only the LNG terminal is present ($A_{wet} = 1.9*10^6$ m^2) and, (2) all the terminals are present ($A_{wet} = 1.5*10^6$ m^2).

Furthermore, it holds that the tidal range has been optimised by the metocean studies performed by Fugro. In its siltation study in December 2000, Svašek assumes a mean tidal range of 5 metres instead of the more detailed range of 4.6 metres.

Water exchange due to the entrance circulation

Svašek has based the water exchange caused by the entrance eddy on the calculated exchange volumes at the harbour entrance during the tide. In Figure 9.1, the exchange volumes are given. The tidal prism volumes are also reflected in this figure.

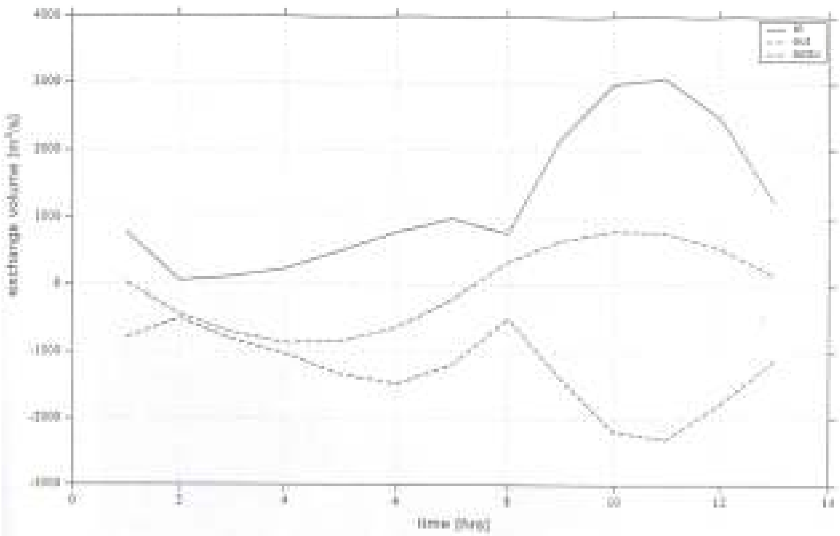


Figure 9.1 Exchange volumes harbour entrance

From Figure 9.1, it can be concluded that the computed circulation discharges are much larger than the tidal discharges. During flood the eddy volume is approximately four times the tidal volume, while during ebb there is still an entering volume, which is approximately equal to the outgoing tidal prism. Figure 9.1 is based on the water exchange volumes for an old harbour layout. Without further research on the impact of the revised layout on the exchange volumes, Svašek states that the water exchange volume caused by entrance circulation equals about four times the tidal prism. This corresponds with an exchange of $3*10^7$ m^3 per tide (with a channel depth of CD-12.5 m).

In the present study, it has been concluded that the water exchange due to entrainment by longitudinal flow effects is proportional to a characteristic velocity ratio, the width and the mean depth of the entrance. If all these variables are known, the water exchange due to this mechanism can be estimated, except for a numerical coefficient. A scale relation with the IJmuiden harbour entrance has been made to obtain this factor. This has resulted in a water exchange volume due to the eddy from 2 to $5*10^7$ m^3/tide for an entrance depth of CD-12.5 m.

The, in the present thesis, expected water exchange volume caused by entrainment by longitudinal flow effects varies around the volume that is computed by Svašek. Both studies contain several uncertainties. Svašek does not look at the expected water exchange volumes for the improved layout. It assumes a rather arbitrary eddy exchange volume, based on an old layout, which is equal to four times the tidal prism. Svašek uses the flow model FINEL in two-dimensional mode while the circulation at the entrance is highly three-dimensional. This may lead to water exchange volumes that are not completely in line with reality.

At present, the empirical scale relation that is used in present thesis, is the only available tool. However, it has to be considered with care as well. It is based on data at the IJmuiden harbour entrance of only three days. Furthermore, the result is sensitive to the location that is chosen for comparison of the current velocities at Hazira and at IJmuiden. The use of a three-dimensional flow model at the entrance of the proposed Hazira harbour with proper formulation of the turbulent horizontal exchange processes is highly recommended.

Mean sediment concentration

In December 2000, at the time Svašek has carried out the siltation study, very little data regarding the amount of total suspended solids was available. For this reason, Svašek has assumed a yearly mean sediment concentration of 500 mg/l.

During the December 2000 and April 2001 service visits, Fugro has fitted turbidity sensors to current meters. Subsequently, turbidity has been related to the suspended sediment concentration using reference samples that have been analysed in a laboratory. Although the data is still relatively poor, the observations show that an annual mean suspended solids concentration of 500 mg/l is exceeded. As stated in chapter 5, a yearly mean sediment concentration of 1000 mg/l is assumed in the present study.

Retention time and settling factor

Svašek classifies the average retention time according to the water exchange mechanism in question. The retention time of the tidal prism equals half a tide, which corresponds to approximately 6 hours. An accompanying siltation ratio of 75 percent is assumed. Svašek assumes that the average retention time of the water in the entrance circulation is much less than the retention time of the tidal prism: 0.5-1 hours instead of 6 hours. For this reason the settling factor of the entrance circulation is assumed small, about 20 percent.

To make comparison with the present study possible, a combined settling factor for both water exchange mechanisms is determined. This is realised according to their water exchange volumes, hence:

$$p_{combined} = \frac{p_{tide} + 4p_{eddy}}{5} = 0.3 \qquad (9.1)$$

Svašek expects 30 percent of the incoming suspended sediment to settle in the basin. In this thesis, the settling factor ranges from 30 to 50 percent. Two main reasons for the higher settling factor can be mentioned.

In this study, Svašek does not take into account the influence of the secondary eddy that is expected in the Hazira harbour basin. Secondary eddies can transport sediment particles further into the basin where the particles finally settle. Secondary effects are expected to lead to a higher sediment trapping efficiency.

Secondly, it is hypothesised that the suspended sediment concentration profile collapses when the (high-concentrated) water enters the Hazira harbour basin. A density current may be triggered by this collapse, which leads to rapid siltation in the Hazira harbour basin.

Dry bulk density

In the siltation study of December 2000, a dry bulk density of 500 kg/m^3 is adopted. Assuming a seawater density ρ of 1025 kg/m^3 and a sediment density ρ_s of 2650 kg/m^3, this results in a wet density of about 1330 kg/m^3. The wet density is an important parameter for research on the consolidation rate in the harbour.

In the present study, the dry bulk density is based on data of dredged materials of harbour basins of the Port of Rotterdam. As a result, a dry bulk density of about 350 kg/m^3 is assumed. The accompanying wet density of the deposits amounts to about 1240 kg/m^3.

Annual sedimentation rate

Resulting from the exchange due to the tidal prism and the entrance circulation, Svašek assesses the total siltation of fine-grained sediments at $8*10^6$ m^3 per year. It holds that the LNG terminal is fully operational and the bulk terminal platform has already been constructed. In the present study, the deposit rate for the situation in which only the LNG terminal is fully operational is expected to vary from $20*10^6$ m^3 per year.

Although a great difference in the annual sedimentation rate is found between the two studies, the discrepancies can be explained fairly well. To conclude this section, the main causes for the difference in the expected sedimentation rates are summarised:

- Observations have shown that a mean suspended solids concentration of 500 mg/l, as assumed by Svašek, is too low. In the present study, a yearly mean sediment concentration of 1000 mg/l is expected. This leads to an annual siltation, which is twice as much as the one adopted by Svašek.
- Svašek does not take into account the secondary effects that may transport sediment particles further into the basin. Furthermore, a collapse of the suspended sediment concentration profile is hypothesised in the present thesis. A density current may be generated by this collapse, which leads to rapid siltation in the Hazira harbour basin. Both processes are expected to lead to higher trapping efficiencies.
- The difference in water exchange due to the tide is partly caused by analysis of different situations. Svašek assumes a situation in which also the bulk terminal platform is present. This leads to a smaller wet surface area and hence, a smaller tidal prism is obtained. On the other hand, it holds that the tidal range has been optimised. Svašek assumes a mean tidal range of 5 metres instead of a range of 4.6 metres, which results from site investigations by Fugro Geos. A larger tidal range, as assumed by Svašek, leads to a larger tidal prism.

Svašek has confirmed that the level of detail of the finite element grid of the coastal area surrounding Hazira, used in this study, is not adequate for accurate eddy modelling and that the results of this study have to be interpreted in a qualitative way.

9.1.2 Siltation study Svašek August 2001

In August 2001, Svašek has investigated the siltation of the ‘full port layout’ at Hazira. Apart from the LNG berths, the harbour includes general cargo and bulk berths, as well as the container terminal. For Svašek's study in August 2001, the morphological computer model

MORFIN has been used. As the sediment transport can be divided into two types of sediment, MORFIN contains different models for sand and fine-grained material. The latter type of sediment is dominant in the more sheltered areas like the harbour basin. This section deals with some interesting aspects of the siltation study as performed by Svašek in April 2001.

Mathematical simulation package MORFIN

The principle of the model for fine-grained material is that siltation takes place when the shear stress is smaller than the critical value assumed for sedimentation (provided that there is fine-grained sediment available in the water). The model updates bed levels but there is no feedback to the flow. The sedimentation in the basin is calculated using Krone (1962). Krone states that the sedimentation speed for fine sediments can be expressed by:

$$\begin{aligned} &\phi_{s,fines} = w_s c(1-\tau/\tau_s) \quad \text{for } \tau < \tau_s, \\ &\text{else } \phi_{s,fines} = 0 \end{aligned} \tag{9.2}$$

where $\phi_{s,fines}$ reflects the fine sediment flux to the seabed due to sedimentation in kg/m^{2}s, w_s the particle fall velocity in m/s, c the suspended sediment concentration in kg/m^3, τ the bed shear stress and τ_s the critical shear stress for sedimentation in Pa. The shear stress is calculated directly from the flow model FINEL by:

$$\tau = \rho g u^2 / C^2 \tag{9.3}$$

where u is the current velocity and C is the Chézy coefficient in m$^{1/2}$/s

Fall velocity

In April 2001, sediment concentrations, water levels and flow velocities are available for the same period. For this reason, the fall velocity of a fine sediment particle has been calculated based on the data measured in April 2001. For the conditions around slack tide, a fall velocity of approximately 0.003 m/s is assumed.

This relatively large fall velocity of 0.003 m/s is in line with the findings of section 7.3. In this section, it is hypothesised that the concentration profile collapses when the flow velocity decreases rapidly. The sediment can no longer be kept in suspension and a high-concentrated fluid mud layer is formed just above the seabed. The hypothesised collapse of the concentration profile leads to apparent fall velocities that are much greater than fall velocities of single sediment particles in 'normal' conditions, which are around $1*10^{-4}$ to $5*10^{-4}$ m/s.

Mean sediment concentration

Svašek confirms that the suspended sediment concentration of 500 mg/l, as supposed in the study of December 2000, is too low. In this second study a mean sediment concentration of 800 mg/l is assumed. This value is based on the April 2001 data when the observed amount of total suspended solids is substantially lower than the measured data of December 2000. Therefore, in the present thesis a mean suspended sediment concentration of 1000 mg/l is assumed.

Critical shear stress

After calibration, a value of 2 Pa is assessed for the critical shear stress for sedimentation. In comparison with values found in literature (Van Rijn, 1993), a critical shear stress of 2 Pa is relatively high. Generally it ranges from 0.03-1.7 Pa. Svašek states that the siltation results are very sensitive to this critical shear stress for sedimentation.

Annual sedimentation rate

Svašek computes the yearly siltation volumes of the proposed Hazira harbour by multiplying the values of the relative change in depth, which are based on a period of 14 days, by 26. In theory, this coincides with maintenance dredging every 14 days. Svašek concludes that the total dredging volume in the harbour basin and entrance channel, being a combination of sand and silt, is around 8.1×10^6 m^3 per year.

However, this volume reflects the relative siltation rate and not on the absolute change in depth due to the presence of the harbour. This leads to some discrepancies with the method used in the present thesis.

The relative siltation rate reflects the change in depth that should be attributed to the presence of the harbour. It is obtained by subtracting the change in depth for the reference situation without harbour and with the 'full port layout' from each other. This way, possible modelling inaccuracies that may occur both in the reference situation and in the situation with the harbour, reduce to zero. Svašek states that this makes the relative siltation rate the best indication for research on the morphological impact concerning the sediment transports in the area surrounding Hazira. Hence,

$$relative\ change\ depth = change\ depth\ with\ harbour - change\ depth\ without\ harbour$$

where erosion is considered positive.

The relative siltation rate does not reflect the actual volume of sediment that has to be dredged in the area surrounding Hazira when the harbour is present. The morphological one-year computation for sand shows that a considerable amount of sand has to be dredged from the harbour basin. This is not in line with reality because most sand has settled before the particles enter the basin. The sedimentation of sand particles in the basin results from the erosion of sand in the reference situation after one year. The actual dredging volumes of fine-grained materials are expected to be greater than assumed by Svašek.

Furthermore, Svašek has based the siltation of the ‘full port layout’ on a minimal depth of the entrance of 12.5 metres below Chart Datum, i.e. MSL-16.9 m. See Figure 9.2. However, large bulk carriers have to be able to enter the basin at low water. This requires a minimal depth of 17.5 metres below Chart Datum, i.e. MSL-21.9 m.

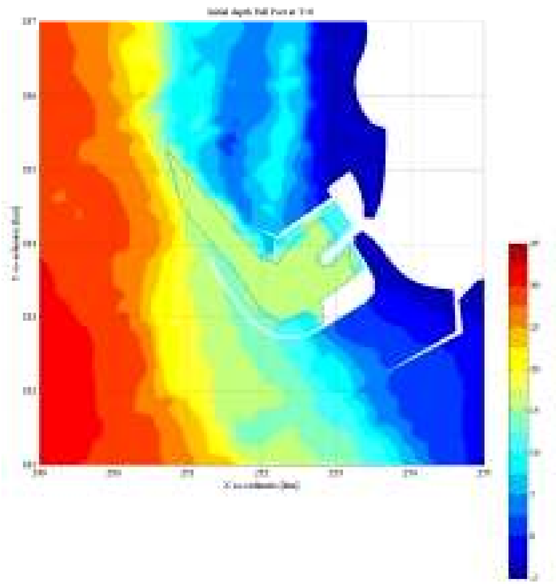


Figure 9.2 Depth profile with harbour relative to MSL at T=0

A deeper entrance leads to more incoming sediment. As a result, more siltation is expected than concluded by Svašek in the siltation study of August 2001.

Svašek assumes a dry bulk density of 450 kg/m^3. This corresponds with a wet density of 1300 kg/m^3. Based on the situation with maintenance dredging every 14 days, the adopted dry bulk density of 450 kg/m^3 is too high as indicated by Table 8.4.

As discussed earlier in this section, an interesting aspect comprises the large apparent fall velocity, found by Svašek looking at the April 2001 data. This large fall velocity corresponds with the assumed interaction between sediment and flow, as stated in section 7.3.

9.2 Accuracy of data

9.2.1 Shape of harbour entrance

In the present thesis, the harbour entrance is schematised to a rectangular cross section with an average entrance depth h_0, which is computed as $h_0=A_0/b$. In this equation, A_0 reflects the cross sectional area of the entrance relative to mean sea level and b is the entrance width at mean sea level.

Since the anticipated harbour entrance at Hazira has a relatively wide trapezoidal cross section, schematisation to a rectangular cross section generally is a good approximation. A reliable theoretical computation of the water exchange in a harbour of arbitrary shape is time consuming and shape factors have to be introduced.

9.2.2 Mean suspended sediment concentration

As a result of the highly dynamic character of the Gulf of Khambhat, the water carries a high amount of suspended sediment. Water quality analysis and turbidity measurements have shown suspended sediment concentrations ranging up to 2300 mg/litre depending on the location and the phase of tide.

Unfortunately, the expected suspended solids concentrations have been based on relatively poor data. Fugro Geos has only measured turbidity during the service visits of December 2000 and April 2001. Furthermore, the water samples have been collected on an arbitrary moment within the tidal cycle. As the annual deposit rate and the mean sediment concentration are proportionally related, the sediment concentration in other port studies in the Gulf of Khambhat is considered in this section.

Dholera port

2DH morphological studies have been carried out to assess the impact of the proposed Dholera port in the Northwestern part of the Gulf of Khambhat (Kant, 2000). The anticipated port is located about 25 kilometres North of Bhavnagar (see appendix L1) The layout of the port, which is exposed, is given in appendix L2.

The highly dynamic character of the Gulf is clearly visible in the shift of coastline over the last ten years. To the North of the project site, the coastline has eroded with an average rate of about 10 m/year while the erosion rate of the Southern side even amounts to 70 m/year. At the same time the Bavaliari Creek moves to the South and the Gundala Creek moves to the North with rates of respectively 13 and 60 metres per annum. The coastline at the project site has to be fixated in order to protect Dholera port.

As a result of the soil characteristics and the large tidal range and currents, the amount of suspended sediment in the water is high. Measurements in the Dholera Channel have shown sediment concentrations varying from 0.25 to 14 g/l. Especially during spring tide, high amounts of suspended sediments can be found. These measurements have been performed in March and April (1999). In the monsoon period, even higher amounts of suspended sediment may be expected. As a result, high dredging rates are hypothesised. For the initial design, it has been assumed that about $35\text{-}240 * 10^6$ m^3/year of sediment needs to be dredged. As the port is not feasible in this way, the port layout has been optimised by reducing the cross sectional area of the channel, which has resulted in a maximum deposition rate of about $10 * 10^6$ m^3/year.

Bhavnagar new port

The high dredging rates in the Bhavnagar new port also confirm the high suspended sediment concentrations in the water. Bhavnagar new port is situated about 10 kilometres to the East of Bhavnagar City. In this port, locks are present which are only opened during high water. During just one single tidal period, sediment is accumulated in front of the locks and sedimentation in the basin occurs due to density currents. Continuous and heavy dredging is necessary in this port.

Based on a very old survey on the entrance channels at Bhavnagar ports by WL|Delft Hydraulics (1952), the silt content of the water in the vicinity of Bhavnagar New Port increases to about 20 g/l during spring tide. Although this amount has to be treated with care, it confirms the high amount of suspended sediment in the Gulf of Khambhat.

Results

It is concluded that the suspended sediment concentrations at the planned Dholera port and at Bhavnagar new port are greater than the suspended sediment concentrations in the vicinity of Hazira. The larger tidal range to the Northern part of the Gulf can easily explain this. As the tidal current velocity is expected to be proportionally related to the transport capacity of the water and due to flood dominance of the Gulf of Khambhat, the sediment concentration of the water generally increases to the North of the Gulf.

Precise comparison of the Hazira data with the data of the formerly mentioned ports is not possible. However, from the foregoing follows that the assumed yearly mean suspended sediment concentration of 1000 mg/l at the proposed Hazira harbour location is not an excessively high value. Probably, the estimated amount of suspended sediment of 1000 mg/l is a reasonable estimate for the yearly mean suspended sediment concentration in the vicinity of Hazira.

10 Conclusions and recommendations

10.1 Conclusions

The present study has provided insight in the expected siltation rate in the last development phase of the proposed Hazira harbour. The annual siltation rate inside this semi-enclosed harbour basin is calculated through three different quantities: the sediment influx per tide, the sedimentation factor and the dry bulk density of the deposits. The sediment influx is determined from the tidal mean sediment concentration of the water entering the basin and the total water exchange volumes.

The anticipated sedimentation in the basin consists mainly of fine-grained sediments and is very large. A multitude of ten million cubic meters is expected to settle in the proposed Hazira harbour basin each year.

The initial siltation volumes per year, for different velocity ratios, are reflected in Table 10.1. They apply for the case that the design depth of the basin is reduced with a maximum of approximately 10 to 15 percent.

Table 10.1 Initial siltation volumes

	Trapping efficiency	$\left(\frac{u_{Hazira}}{u_{IJmuiden}}\right)_{disturbed} \approx 1$	$\left(\frac{u_{Hazira}}{u_{IJmuiden}}\right)_{undisturbed} \approx 2$
Only LNG terminal fully operational	p = 0.30 p = 0.50	$\Delta T_s = 20*10^6$ m^3/year $\Delta T_s = 30*10^6$ m^3/year	$\Delta T_s = 35*10^6$ m^3/year $\Delta T_s = 55*10^6$ m^3/year
All terminals fully operational	p = 0.30 p = 0.50	$\Delta T_s = 20*10^6$ m^3/year $\Delta T_s = 35*10^6$ m^3/year	$\Delta T_s = 40*10^6$ m^3/year $\Delta T_s = 65*10^6$ m^3/year

Furthermore, from the present research on the siltation of the proposed Hazira harbour, it can be concluded that:

- As a result of the highly dynamic character of the Gulf of Khambhat, the water carries a large amount of suspended sediment. The total suspended sediment pattern shows considerable variations in time. During a tidal cycle, near the surface, the sediment concentration decreases at and just after slack water. These low velocities lead to settling of the sediment particles. As a result, near the seabed, an increase in the amount of total suspended solids is found after the change of current direction. It generally holds that the sediment concentration at springs is greater than at neaps because of the increased flow velocities. Apart from the tide, the suspended sediment concentrations are also expected to be influenced by the four typical seasons, which can be distinguished at the West coast of India.

- The water exchange due to entrainment by longitudinal flow effects, which is expected to generate a horizontal eddy in the entrance of the proposed Hazira harbour, takes care of the greatest part of the total exchange volume. An analogy with the eddy that occurs in the existing harbour at IJmuiden in the Netherlands, can be established. It is found that the water exchange caused by the eddy is proportional to the ratio of a characteristic velocity at Hazira to a characteristic velocity at IJmuiden, the mean entrance depth and a characteristic width of the entrance. The water exchange caused by the eddy appears to be sensitive to the location that is chosen for comparison of the characteristic current velocities at Hazira and at IJmuiden.

- No density driven exchange flow induced by salinity differences over the tide, is expected in the planned harbour. This fully mixed condition is also found for the whole Gulf of Khambhat by means of the estuary number. The estuary number classifies estuaries according to their degree of mixing.

- Due to the decreased flow velocities, the water within the proposed Hazira harbour basin is not expected to be able to carry the high amount of available sediment in suspension any longer. A super-saturated condition arises and it is hypothesised that the sediment concentration profile collapses when the water enters the harbour basin. Probably, this collapsed suspension leads to the development of a sediment-driven density current, which causes rapid siltation in the harbour basin. A collapse of the concentration profile leads to apparent sedimentation velocities that are much greater than the fall velocity of a single sediment particle.

- The primary eddy, which is anticipated in the Hazira harbour entrance, is expected to drive a secondary eddy in the basin. Secondary eddies can be of great influence on the sediment transport and on the trapping efficiency of the basin. Secondary eddies can transport the sediment particles further into the harbour basin where the particles finally settle. This process may lead to a large settling factor of the proposed Hazira harbour.

10.2 Recommendations

Based on the findings of the present study, specific recommendations for the Hazira harbour project, as well as more general recommendations can be made.

Recommendations with regard to the Hazira harbour project

- Further research on the consolidation rate of the fine-grained materials, which are expected to settle on the seabed of the proposed Hazira harbour, is recommended. In the present study, the dry sediment density of the deposits is calculated based on data of the Port of Rotterdam in the Netherlands. Probably, the top layer of the fluid mud, deposited on the Hazira seabed, has a low density and weak shear strength. It is expected that the density of the mud layer increases with its depth and with time. The rate at which this process takes place, the consolidation rate, determines the density and the thickness of the mud layer after a certain period.

  Based on research on the Europoort areas in the Netherlands, it has been found that a vessel can navigate and manoeuvre safely in a fluid mud layer up to a bulk density of 1200 kg/m^3. The consolidation rate of the mud influences the maintenance dredging volumes: a higher consolidation rate yields a larger dry sediment density at a certain time, which results in lower maintenance dredging volumes. However, the density of the mud layer influences the maintenance dredging techniques.

- In the present study, the horizontal water exchange, caused by entrainment by longitudinal flow effects in the harbour entrance, is calculated by relating the proposed Hazira harbour to the existing harbour at IJmuiden. However, the bathymetry of the area in the vicinity of Hazira differs significantly from the bathymetry of the IJmuiden harbour area. The adjacent coastal region of the proposed harbour location is characterised by a shallow and partly drying foreshore, while around the IJmuiden harbour relatively large depths can be found. The accuracy of the assumed relationship is unknown and more research is required. The use of a three-dimensional flow model at the proposed Hazira harbour entrance with proper formulation of the turbulent horizontal exchange processes can provide more detailed information concerning the water exchange induced by longitudinal flow effects.

- Additional survey on the sediment concentration in the vicinity of Hazira is required. The annual deposit rate is sensitive to the mean suspended sediment concentration. The yearly mean suspended sediment concentration of 1000 mg/l, as assumed in this thesis, is based on relatively poor data. Turbidity has only been related to the amount of total suspended solids during the service visits of December 2000 and April 2001. Furthermore, water samples have been collected every month. The moments on which these samples have been taken seem to have been chosen rather arbitrarily, which causes a wide range in total suspended solids data throughout the year.

  The assumed suspended sediment concentration at Hazira has been compared with the measured concentrations at the planned Dholera port and the existing Bhavnagar new port, situated in the Northern part of the Gulf of Khambhat. The sediment concentrations at these harbours are much larger than 1000 mg/litre. This suggests that the adopted sediment concentration in the vicinity of Hazira is a reasonable estimate. However, more total suspended solids data nearby the project location is desirable.

- As high maintenance dredging volumes are expected in the proposed Hazira harbour basin, it is recommended to consider possible options of improvement of the present layout to minimise siltation.

General recommendation

- At Hazira, the tidal flow is directed parallel to the entrance channel. As a result, ideal circumstances for stowage of the water are created. Entrainment by longitudinal flow effects generates an eddy at the entrance of the proposed harbour and large water exchange volumes can be distinguished.

  So far, research has only been carried out on eddies induced by entrainment due to cross flow effects. At present, studies concerning eddies generated by entrainment by longitudinal flow effects are not available. Therefore, it is strongly recommended to carry out laboratory experiments on this mechanism. This can lead to an improved understanding of the water exchange due to this mechanism.

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