A parametric study concerning estuary mouth dynamics and inlet closure.

"A case study of the St Lucia Estuary mouth - South Africa"



Delft University of Technology

Vincent Moerland January 2013



Delft University of Technology

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Abstract

With the development of a process-based model (Delft3D) of the St Lucia Estuary inlet, a first approach is made with regard to the estuary mouth dynamics and closure mechanisms that are observed at St Lucia inlet. The purpose of this thesis is to get a better understanding of the hydrodynamic and morphological behaviour of the St Lucia inlet with the additional effect of the Mfolozi River discharge. The focus in this thesis is on the period after 2001 till present, where the management policy is to let the St Lucia inlet function in its natural state and with the possibility to join with the Mfolozi River. A model with a schematized situation of the estuary with representative inlet geometry is forced with representative waves and tide conditions. Important factors determining the inlet stability such as tidal prism, longshore sediment transport, inlet geometry and river discharge are investigated in this thesis. The Mfolozi River mouth and St Lucia Estuary entrance are situated in a seasonal varying climatic regime with long drought periods with low riverine flows followed by wet periods and cyclonic events. A high energy wave climate in combination with a micro-tidal regime and a high rate of longshore sediment transport are the most important factors of the instability of the St Lucia inlet. According to Bruun (1978) inlets that are classified with a P/M ratio below twenty are found to be unstable and the inlet may be closed by deposition of sediment during a storm event because the tidal prism is relative small. In line with Bruun, the St Lucia inlet can be classified as an unstable inlet with a low P/M ratio of approximately two.

Three scenarios were developed with different estuary dimensions; a small, a medium and a large basin. The inlet geometry is the same in the scenarios and each scenario is modelled with five different simulations. The simulations are forced at the boundaries by a varying range of tide and wave conditions. The tide is varied from average to neap and spring tide. The wave height is varied from average to higher and extreme wave heights. Higher waves are responsible for a higher rate of longshore sediment transport and with both varying tide and wave conditions a wide range of P/M ratios are modelled. In addition the influence of a lower D_{50} was investigated, and the influence of the Mfolozi River was simulated.

The results of the simulations show that they are in line with expectations. Small P/M ratios show that inlets are unstable and different closure mechanisms are observed. Similar to what is found in nature regarding the A-P relationship, a decreasing cross-sectional area with a lowering tidal prism, is also found with the Delft3D models which suggest that the model is capable of giving a good representation of the morphodynamics.

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List of symbols

S	-	The relative density of sediment
g	m/s^2	Gravitational acceleration,
θ_{b}	deg	Wave braking angle and
K ₁	-	Empirical coefficient.
H _b	m	Significant breaking wave height
γ	-	Breaker index
ρ	kg/m ³	Density of water
ûe	m/s	Amplitude of the sinusoidal tidal motion
V	m/s	representative longshore current velocity
c_{f}	-	Friction coefficient
Т	S	Tidal period
Δh	m	Tidal range in the estuary
А	m2	Cross sectional area
D_{50}	mm	Medium diameter of sediment
M_{tot}	m ³	Total littoral drift
Р	m ³	Tidal prism
R	m	Hydraulic radius of the channel
r	-	Parameter define the stability of the inlet
V_{cr}	m/s	Critical velocity
V_m	m/s	Maximum entrance channel velocity
Х	m	Geometric parameter
Q _{max}	m3/s	Maximum river discharge
Q_{min}	m3/s	Minimum river discharge
H_{S}	m	Significant wave height
k	-	Power relation between transport and wave height
σ	s ⁻¹	Angular frequency
k	m-1	Wave number
L	m	Wave length
Т	S	Wave period
L_0	m	Deep water wave length
ML	m	Mean tidal level predicted over a 19 year cycle. (actually 18.61 and

		a third years)
НАТ	m	Highest tidal values predictable over a 19 year cycle
LAT	m	Lowest tidal values predictable over a 19 year cycle
MHWS	m	Mean high values predicted for spring tides over a 19 year period
MLWS	m	Mean low values predicted for spring tides over a 19 year period
MHWN	m	Mean high values predicted for neap tides over an 19 year period
MLWS	m	Mean low values predicted for neap tides over a 19 year period
HATOY	m	Highest values predicted for one specific year (not over the 19
		period)
LATOY	m	Lowest values predicted for one specific year (not over the 19
		period)

1. Introduction

This thesis has been performed to contribute to the understanding of the hydrodynamic and morphodynamic behaviour of the St Lucia inlet, and the Mfolozi River (Figure 1.1). The St Lucia Estuary is connected to the St Lucia lake system and its connection with the ocean is unstable and often closes. Extreme dry periods with low river inflow can result in extended periods with a closed mouth with dramatically low lake water levels and hypersaline conditions which threaten the overall biodiversity and health status of the lake system.

The Mfolozi River, south of the St Lucia inlet, used to be connected to the St Lucia Bay, which used to functioning as a connection to the ocean for both the Mfolozi River and the St Lucia Estuary. This was the natural state of the system. After 1950 the Mfolozi mouth was artificially separated from the St Lucia inlet, to address the perceived threat of siltation of the St Lucia lake system from the silt-laden Mfolozi waters. Since then the Mfolozi mouth has been kept continuously separate from the St Lucia inlet.



Figure 1.1: Aerial view of the St Lucia inlet and Mfolozi River mouth (Google Earth, 2012)

1.1 Objectives and research methodology

The purpose of this thesis is to understand the way the St Lucia inlet works in terms of hydrodynamics and morphodynamics. The research focuses on the St Lucia inlet and the Mfolozi River discharge. The working of the St Lucia inlet is investigated, and scenarios have been setup to get more knowledge about the conditions that result in closure of the inlet. Important factors determining the inlet stability such as tidal prism, longshore sediment transport, inlet geometry and river discharge are investigated in this thesis. By using Delft3D different scenarios have been modelled and simulated.

The following research questions are central in this study:

Research question:

• What is the main cause of the closure of the St Lucia inlet and which characteristic processes influences the morphological behaviour of the inlet?

Sub questions:

- What are the governing characteristics of the hydrodynamic processes such as tides, waves and currents?
- How do the hydrodynamic processes influence the morphological behaviour of the St Lucia estuary inlet?
- What is the influence of the discharge of the Mfolozi River?
- What is the influence of waves on sediment transport under specific representative wave conditions?
- What are the timescales for closure and what is the relationship to longshore transport rates?

In order to get insight in the working of the St Lucia inlet system and the influence of the Mfolozi River, two scenarios have been modelled using Delft3D.

The defined scenarios are as following:

1. Schematised configuration of the St Lucia inlet, without the lake system being connected, forced by representative wave and tide conditions. This scenario applies to three different tidal basin sizes and each basin size is simulated with 5 different wave and tide conditions;

 Schematised configuration of St Lucia inlet, without the lake system being connected. In combination with a specific discharge flowing into the basin, representing the Mfolozi River. This scenario applies to those simulations of scenario 1 that where unstable and closed.

1.2 Thesis structure

In chapter 2 an overview of the literature is given. First a description and the origin of tidal inlets are given, following with an explanation of the morphological units of a tidal inlet. In this part all the features of a tidal inlet are summarised. Subsequently the hydrodynamic classification in which the St Lucia inlet can be placed is described and an overview of the hydraulic boundary conditions and the geometric elements is given.

Following with the stability of tidal inlets, in this part the mechanisms that determine the location stability is described. Processes such as migrating inlets, bar bypassing and tidal currents that transport material into inlets are explained. Three main mechanisms of the way how sediments bypass along inlets are summarised. Then empirical relationships concerning cross-sectional stability are described. At the end of the chapter an overview of the literature concerning wave-related processes is given.

Chapter 3 gives a description of the study area. It gives background information and a historical view of the St Lucia lake system and the Mfolozi River mouth. It describes the way St Lucia lake mouth used to be and how human interventions changed the working of the system. Climate conditions such as the wave climate, tidal data and tidal prism observations with flows rates are presented in this chapter. Observations of longshore sediment transport rates are summarised as well. The Mfolozi River catchment is described and St Lucia hydrodynamics and offshore bathymetry is presented.

In chapter 4 the working of the process-based program Delft3D is explained. The used modules of the suite will be summarised. Subsequently the setup of the model which represents a schematised configuration of the St Lucia inlet is described. The three different scenarios and the choices made regarding these scenarios will me made clear. And an overview is given of the simulations that are conducted.

In chapter 5 the results of the three scenarios, in total 15 simulations, are analysed and elaborated. Additional to these simulations, an extra set of sensitivity simulations were conducted with different input parameters to get a better understanding of the influence and sensitivity of the input parameters. And the last section of the chapter covers the results of two simulations selected from the main scenarios whereby a river discharge is added in the model. This is done to investigate the influence of the Mfolozi River discharge.

Chapter 6 addresses conclusions and recommendations for further research.

2. **Overview of literature**

2.1 Tidal inlets

According to Davis and Fitzgerald (2004), "a tidal inlet is defined as an opening in the shoreline through which water penetrates the land, thereby providing a connection between the ocean and bays, lagoons, and marsh and tidal creek systems".

Tidal currents maintain the main channel of the tidal inlet. Even when a tidal inlet coincide with a mouth of a river, the inlet dimensions and the sediment transport patterns are still governed, to a large extent, by the volume of water exchanged at the inlet mouth and the reversing tidal currents. An important parameter is determined by the volume entering or leaving the inlet. The water entering during a flooding tide or leaving during the ebbing cycle is referred to as the tidal prism (see also chapter 2.3.2 Escoffier's model). So the tidal prism is a function of the open surface area of the basin inside the inlet and the tidal range.

According to Bruun (1978) tidal inlets are mainly distinguished by three main groups of inlets: those with a geological origin, those with a hydrological origin and those with a littoral drift origin. Inlets with a geological background have typically rocky formations of gorges. They do not follow the laws for tidal inlets in alluvial material like sand.

Inlets where rivers enter the ocean belong to the hydrological origin. Such inlets are exposed to tidal currents, which penetrate into the river mouth and contribute to changes in the geometry. In cases of density currents complex situations in the channel may arise. The inlets with a littoral drift origin are controlled by the littoral sediment transport. Here the governing process determines the stability and equilibrium of the inlet.

Most inlets on sea coasts have a littoral drift origin. The predominant factors for the creation of such situations are:

- breakthrough caused by rise of sea level and formation of spit barriers;
- breakthrough caused by consolidation of soil and sea level rise;
- by formation of a barrier;
- by formation of a barrier across the inlet or the bay.

Most inlets situated on littoral drift coasts did not survive over the years because tidal flows were insufficient and the littoral drift deposits dominant.

2.2 Tidal inlet morphology

A tidal inlet system consists of three major morphological units:

- The ebb tidal delta, a sand body formed seaward of the entrance channel;
- The tidal gorge, the narrow deep channel at the inlet entrance;
- The flood tidal delta, a shield of sand which develops in the tidal basin landward of the tidal gorge.



Figure 2.1: A) Morphological features of a tidal inlet on a sandy coast, B) Cross section profile from x to y through the tidal gorge and over both flood and ebb tidal deltas (Smith, 1987)

The most common morphological features of an ebb tidal delta are the main ebb channel, channel margin levees, swash platforms with swash bars, marginal shoals, marginal flood channels and the

delta terminal lobe. Ebb tidal flows are the main cause of the formation of the ebb delta. The flows that occur from the narrow tidal gorge as a fully turbulent diverging jet over the falling tide maintain the ebb channel. Scouring effects occur in the channel during this process whereby sediment is transported seawards onto the ebb channel.

On the sides of the main ebb channel, channel margin levees can develop. These submarine bars are formed where the sediment transporting capacity of the ebb currents decreases along the edge of the laterally diverging ebb tidal jet. Swash platforms are formed by near shore wave action. They have often migrating swash bars developed on it.

Marginal shoals mainly occur at the outer edge of the ebb tidal delta. These shoals are formed, build up and continuously flattened by waves and tidal currents. Mostly the ebb delta marginal shoals are shallow channels. On the outer edges of the ebb delta a few larger ebb dominated channels can be present, while along the coast nearer to the shore, channels are predominantly flood dominated. Also known as "marginal flood channels", they carry flood currents towards the inlet during the initial phase of a flooding tide.

The most narrow and usually deepest section of a tidal inlet is the tidal gorge. In this section both the flood currents and ebb currents are concentrated. The geometry depends on a number of factors:

- The geological origin of the inlet;
- The supply of sand from longshore drift;
- The geological substrate through which it passes;
- The tidal prism.

The flood tidal delta is formed on the landward side of the tidal gorge. The main driving mechanism that builds this delta is sediment transported into the inlet by flood currents. Usually on the sides and near to the coastal barrier ebb dominated channels are found. Features that contain the flood tidal delta are the flood ramp and flood channels, the ebb shield, ebb spits and spillover lobes. The flood ramp starts at the end of the tidal gorge. It is a steadily shallowing section of the flood delta, and typically divides into a number of shallow diverging channels. Currents passing these sections of the flood tidal delta are nearly always flood dominated. The ebb shield becomes visible when the water level lowers. The landward side has a steep slope and is connected to the ebb dominated channels. During the ebb tide most of the water is flowing out of the tidal basin over and around the ebb

shield shoal. The ebb shield is occasionally breached by small shallow channels. Ebb spits are formed by accumulated sediments that become especially well developed at the margins of the flood delta and result in a separation of the flood delta ramp from the ebb flow dominated channels.

The most important factors determining the overall morphology of the tidal deltas, especially the ebb-tidal delta, are the combination of waves and tides. Wave action is the main driving force to move sediments onshore and limits the area over which the ebb-tidal delta can spread out.

2.3 Hydrodynamic classification

In the inlet of the estuary wave and tidal influences are combined. Outside the inlet the tidal range depends primarily on the ocean tides and their interaction with the continental shelf. Micro-tidal, meso-tidal and macro-tidal ranges can be distinguished see Table 2.1.

Classification	Mean spring tidal range
Micro-tidal regime	< 2 m
Meso-tidal regime	2 – 4 m
Macro-tidal regime	> 4 m

Table 2.1: Tidal classification of coast and tidal inlets

Wave conditions are generated seaward and thus independently of the inlet. Wave energy can be classified as low, medium and high (Table 2.2).

Table 2.2: Wave	e climate	classification
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Wave energy class	Mean significant wave height [m]
Low wave energy	< 0.6
Medium wave energy	0.6 – 1.5
High wave energy	> 1.5

Because the parameters of the tidal range and the wave energy are independent of the inlet system configuration, Hayes (1979) and Davis and Hayes (1984) used this information to distinguish different classes in terms of wave domination and tide domination see Figure 2.2. Each class develops its own specific morphologic feature. For example, the relative size of the flood- and ebb-tidal deltas and the mechanism for sand-bypassing of entrances are dependent on the relative

MEDIUM LOW HIGH WAVE ENERGY high 6 macrotidal tide-dominated (nigh) 5 low mean spring tidal range (m) tominated (10m) macrotidal 4 nived energy (tide dominant) tide 3 high mixed energy St Lucia mesotidal 2 1.84 low mesotidal wave-dominant 1 microtidal 0 1.5 1.58 0 0.5 2 2.5 1 mean wave height (m)

wave/tide dominance. Every class covers a spectrum of tidal ranges and wave heights; the relative effects of waves and tides rather than the absolute tidal ranges and wave height are important.

Figure 2.2: Hydrodynamic classification (Hayes, 1979)

2.4 Hydraulic boundary conditions and geometric elements

Crucial for determining the morphology of tidal basins and inlets are the hydraulic boundary conditions. Not only is the relative dominance mentioned above of tides or waves are important, but there are several other hydraulic conditions that also control the morphodynamic behaviour (Bosboom and Stive, 2011).

The most important hydraulic and geometric elements are:

- The surface area of the basin in combination with tidal range, determine in principle the tidal prism. Several empirical relations regarding the tidal prism are found to be the equilibrium or minimal stable cross-sectional channel area of the inlet, the sand volume stored in the ebb delta and the channel volume.
- Phase speed difference of the propagating wave (tide) caused by different morphological geometries during low and high tide. This results in strengthening or weakening of the magnitude of the maximum flood flow compared to the maximum ebb flow, and thereby shorten or lengthen the flood duration compared to the ebb duration; this consequently leads to a net import or export of sediment respectively and hence steers the morphological development of the basin in time. Important factors for this phenomenon are the surface areas at low and high water and the mean water depth at high and low water, both of which are determined by a combination of tidal range, channel depth and intertidal storage areas or flats.
- Tidal waves in basins may be either progressive or standing, or a mixture of the two, depending on the length of the basin. In case of short tidal basins the tidal wave is reflected and it has a standing character. The tidal range is of similar magnitude as on the open sea. In longer basins resonance can occur when the length of the basin is a quarter of the tidal wave length, or a multiple of that, in that case the tidal amplitude will increase in the basin. The longer the basin the more tidal wave is damped by friction. In case of no reflecting tidal wave, the tidal wave tends to have a stronger propagating character.
- In very short basins, relative to the tidal resonance length, the combination of tidal range, channel depth and intertidal storage areas or flats results in a different type of asymmetry than asymmetry between ebb and flood duration. In short basins, the duration of the flow change is different during HW and at LW slack. This can influence the net transport of fine sediments in the basin.

2.5 Inlet stability

2.5.1 Location stability

According to Bruun (1978), looking to the history of tidal inlets a continuously changing geometry is observed. The important parts that vary the most are the length of the inlet channel and its configuration as well as the cross-sectional area of the gorge. Tidal inlets situated at littoral drift coasts, are in most cases about to deteriorate because it's not possible to deposit sand in the inlet or in its basin and ocean shoals forever. Some inlets experience natural by-passing of sand, the material is carried from one side of the inlet to the other. Bypassing of sediments can take place partly or wholly on an offshore bar, this depends on the depth of the bar, which must be shallow enough to utilize the transport mechanism of the wave and current action.

Tidal currents may also bypass material. Flood currents usually deposit material in the inlet channel, after which ebb currents flush the material back in the ocean again. In some cases the material may be jetted so far out in the ocean that it is lost forever for the shore. Or longshore currents may transport it to the beach on the downdrift side, so that the leeside erosion may be decreased or even eliminated.

The most inlets which are situated on littoral drift coasts do have a migrating character in the direction of the prevailing littoral drift. In some cases the inlet may move in the opposite direction. The predominant factor therefore is a special combination of tidal flow, and wave action favouring deposits on the downdrift side and erosion of the updrift side. See Figure 2.3.



Figure 2.3: Model of a migrating inlet (Davis and Fitzgerald, 2004)

The rate of movement of migrating inlets on sandy coasts depends on the magnitude of littoral drift, the velocity of tidal and other currents, and the phase difference between any longshore tidal currents and the tidal currents in the inlet. As a result of sand deposits, greater on one side than the other, the channel is often forced against the downdrift side of the inlet, causing continued erosion.

2.5.2 Inlet sediment bypassing

The mechanism of sand that moves past tidal inlets and is transferred to the downdrift shoreline is called inlet sediment bypassing. Davis and Fitzgerald (2004) summarized the most common ways in which inlets bypass sand: 1) inlet migration and spit breaching; 2) stable inlet processes; 3) ebb-tidal delta breaching, see Figure 2.4.

1) Inlet migration and spit breaching

In this situation an enormous supply of sand and a dominant longshore transport direction cause spit building at the end of the barrier. The sand accumulates at the end of the barrier and causes the inlet to migrate by eroding the downdrift barrier. The inlet is moved in the downdrift direction of the shoreline, resulting in a lengthening of the inlet and a retardation of the exchange of water between the ocean and the backbarrier. A large water level difference between the ocean and the bay is the result, which makes the coastal barrier highly vulnerable to breaching, especially during storms. Ultimately the barrier spit will breach and a new inlet develops in a more favourable position and the tidal prism is diverted to the new inlet resulting in a closure of the old inlet.

2) Stable inlet processes;

This mechanism of sediment bypassing occurs at inlets that do not migrate and whose main ebb channels remain more or less in the same position. Sand enters by three different mechanisms; wave action along the beach, flood-tidal and wave-generated currents through the marginal flood channel and waves breaking across the channel margin linear bars. Ebb currents transport the sand deposited in the main channel seaward to the terminal lobe. At low tides waves breaking on the terminal lobe transport sand along the side of the delta toward the landward beaches. At high tides the waves break over the terminal lobe and create swash bars on both sides of the main ebb channel. The swash bars

migrate onshore due to the dominance of landward flow across the swash platform. At the final stage of migrating they attach to the channel margin linear bars forming large bar complexes.

3) Ebb-tidal delta breaching;

This process is about sediment bypassing that occurs at inlets with a stable throat position, but whose main ebb channels migrate through their ebb-tidal deltas. Sand enters in the same manner as described in the first process above, but now the delivery of sediments by longshore transport results in sand accumulation updrift of the ebb-tidal delta. The sedimentation on the updrift side causes the main ebb channel to deflect towards to the downdrift shoreline, until it reaches almost parallel to the coastline. This new configuration leads to inefficient tidal flows through the inlet, and eventually leads to breaching of a new channel through the ebb-tidal delta. Most of the time this process takes place during spring tides, or periods of storm surge when the tidal prism is very large.



Figure 2.4: Conceptual model of: Inlet migration and spit breaching, Stable inlet processes and Ebb-tidal delta breaching. By (Fitzgerald, Kraus and Hands, 2001)

2.5.3 Mechanisms of inlet closure

Ranasinghe, Pattiaratchi and Masselink (1999) performed a study with a morphodynamic model which simulates seasonal closure of tidal inlets. They based their two conceptual models on earlier studies done about inlet closure mechanisms (Bruun and Gerritsen, 1960; Fitzgerald, 1988; Gordon, 1990; Hayes, 1991; Ghosh et al., 1991; Largier et al., 1992; Murtagh and Nelson, 1993; Treloar et al., 1993; Cooper, 1994).

Mechanism 1:

In this mechanism the interaction between the inlet currents and the longshore currents controls inlet closure. The longshore sediment transport is interrupted by the tidal inlet; as a result a shoal will form updrift of the inlet. On the downdrift side of the channel a smaller shoal will develop, this shoal results from the deposition of sand due to the retardation of the ebb current as it diverts by the longshore current and is shifted away from the inlet entrance in downdrift direction. Depending on the supply of longshore sediment transport, sand depositing on the updrift shoal will result in the growth of the shoal and eventually form a spit across the inlet channel. The currents in the inlet channel and the involved capacity of removing the sediments that is being deposited in the channel mouth will determine the progradation of the spit. If the current is not strong enough the spit will continue to grow and prograde until the inlet is completely blocked.

Mechanism 2:

This mechanism is possible when the currents in the inlet are smaller than 1 m/s. Therefore this occurs only in mircro- or mesotidal environments where the tidal prisms are small. In this mechanism the following characteristics are present: weak inlet currents, onshore sediment transport due to swell waves in combination with small longshore currents and hence longshore sediment transport rates small as well. Seasonal variations result in the winter when storm conditions dominate in the formation of an offshore bar. When long period swell waves start to dominate the sand will be transported back to the beach. The inlet is only able to stay open in this period due to strong ebb flows which occur due to large tidal ranges or high river discharge. In the summer the river discharge reduces and while the ebb current weakens onshore sand transport due to the swell waves will eventually close the inlet.



Figure 2.5: Schematic depiction of inlet closure by longshore processes and cross-shore processes (Ranasinghe, Pattiaratchi and Masselink, 1999).

2.5.4 Escoffier's model

Escoffier (1940) was the first who studied the stability of the cross-sectional area of the inlet appropriately. He did his research on the governing factors: tidal currents, storms, the tidal prism and the littoral sediment transport. Because of the littoral drift leaving and entering the inlet with the tide, there can be a considerable variation of the cross-sectional area. Escoffier's predominantly study led to an expression for the maximum cross-sectionally averaged entrance channel velocity u_e for a given estuary inlet.

The maximum cross-sectional velocity is the maximum velocity during the tidal cycle. To understand its behaviour as a function of the cross-sectional area, it can be approximated as the amplitude \hat{u}_e of a sinusoidal tidal motion u. In that case the maximum cross-sectionally averaged entrance velocity $u_e = \hat{u}_e$ is related to the tidal prism P. The tidal prism P is equal to the time integral of the inflow during flood or to the outflow during ebb. P is defined as following:

$$P = \int_{0}^{\frac{1}{2}T} A_e u dt = \int_{0}^{\frac{1}{2}T} A_e \hat{u}_e \sin\left(\frac{2\pi}{T}t\right) dt = \frac{TA_e}{2\pi} \hat{u}_e$$

From this it follows that:

$$\hat{u}_e = \frac{\pi P}{A_e T}$$

Escoffier (1940) related u_e to the hydraulic radius of the channel R, its cross-sectional area A_e and the tidal range in the estuary Δh . Escoffier assumed that the other variables such as the channel bed roughness, the length of the channel, the surface area of the estuary and the tidal range at sea and in the basin are constants, since the calculation was made for a given estuary. He combined the variables into a single parameter "x", such that a larger entrance cross-section results in a larger value of "x". Qualitatively, he found that u_e varied as a function of "x" more or less as shown in Figure 2.6.



Figure 2.6: Channel velocity geometry relationship

In this Figure 2.6 the horizontal line V_{cr} has been drawn and the intersections of this line with the V_m curve are points which represent inlets whose channels are stationary in size. When solving the two equations equal to each other, the two intersections point B and D are their roots. The first one is the unstable root and the second one is the stable one. So when the channel dimensions place it between A-B, the channel will close by natural processes because the friction is too high and the channel too small. The part above V_{cr} , between B-D is the part where erosion takes place until the stable point D is reached, and then channel is restored to its initial condition. When the inlet dimensions places itself

below D between D-E, then the channel becomes smaller but the velocity increases. So here sedimentation occurs until point D is reached.

In this section an overview is given of the empirical relationships of tidal inlets.

2.5.5 Inlet cross-sectional stability

As stated by Davis and Fitzgerald (2004), tidal inlets occur all over the world and they exhibit consistent relationships which were used to formulate predictive models. The models are based on field data collected at different tidal inlet locations. This data was analysed, and two important relationships have been discovered: first the inlet throat cross-sectional area is closely related to tidal prism, and second the ebb tidal delta volume is a function of the tidal prism.

2.5.5.1 Minimum equilibrium cross-sectional area and tidal prism

A very important part concerning Escoffier's curve is the area between B and D above u_{eq} . O'Brien (1931, 1969) and Jarrett (1976) did both relevant studies in finding an empirical relationship between the inlet cross-sectional area and the tidal prism.

The general form of the empirical relationship for the equilibrium cross-section based on the tidal prism is as follows:

$$A_{eq} = CP^{q}$$

in which:

- A_{eq} is the minimum equilibrium cross-sectional area of the entrance channel (throat) measured below mean sea level in m²,
- P is the tidal prism, often the spring tidal prism in m³,
- C and q are the coefficients.

O'Brien (1969) showed that for 28 US entrances $C = 4.69 \times 10^{-4}$ and q = 0.85, when the data was limited to only 8 non-jettied entrances he derived $C = 1.08 \times 10^{-4}$ and q = 1. Looking at the assumption of Escoffier that the equilibrium velocity u_{eq} is approximately 0,9 m/s, this implies that the associated values of q and C are respectively 1 and 7.8 x 10⁻⁵.

2.5.5.2 Sediment bypassing at an inlet entrance

Bruun and Gerritsen (1960) and Bruun (1978) proposed a parameter r to define the stability of the inlet based on the sediment by-passing capacity:

$$r = \frac{P}{M}$$

in which P is the tidal prism in m³ and M is the total littoral drift in m³/year.

P/M	Channel stability
> 150	The channel is reasonable stable. A little forming of a bar and good flushing
	properties. These inlets have typical flow-bypassing (tide-dominated)
50-150	Inlets with well-developed ebb delta and one or more channels. Bar and flow
	bypassing. (mixed-energy)
20-50	The channel is highly variable in location and area, with multiple channels
	possible. The inlet can have many bars. To keep the inlet stable dredging and
	jetties are typically required to maintain navigable depths. These types have
	usually bar-bypassing. (typical wave-dominated)
< 20	This is an unstable inlet. The inlet may be closed by deposition of sediment
	during a storm event because the tidal prism is relatively small.

Table 2.3: Channel stability according to the P/M ratio

2.6 Wave-related processes

Waves are the prime movers for the littoral processes at the shoreline. For the most part, they are generated by the action of the wind over water. Waves transport the energy imparted to them over enormous distances, dissipative effects such as viscosity play only a small role. Waves generated at the ocean's surface area do mostly break in the surf zone where the energy is dissipated. Energy of a wave is related to the square of its height and dissipation in the surf zone can be quite large. The energy of a wave is most of the time measured in terms of energy per unit water surface area $E = \frac{1}{8}\rho g H_{rms}^2$ in which H_{rms} the root mean square wave height is.

Waves transform in the surf zone because the waves are affected by the seabed through processes such as refraction, shoaling, bottom friction and wave-breaking. The point where the waves are affected by the bottom is approximately when the water depth becomes less than about half the wavelength (see 2.6.1). In the surf zone the frequency of a wave remains the same but the propagation speed c slows down which results in a decrease of the wave length L.

The waves entering from deeper water are still propagating with a higher speed, and tend to catch up with the waves in shallower water. This results in a concentration of wave energy and an increase in wave height. The process of increasing waves is called shoaling. Besides the decrease of propagation speed of the waves in the cross-shore direction, it is also possible to have water depths varying in the longshore direction which forces the propagation speed in the longshore direction to change. This forces an obliquely incident wave to bend toward normal incidence direction. This process is called refraction. Another phenomena is diffraction, this is the transfer of wave energy along the wave crest due to sheltering by obstructions like islands or breakwaters. This results in lower wave heights at the lee side of an obstruction.

2.6.1 Dispersion relationship

The wave motion can be described by the continuity equations and the Navier-Stokes equations of motion. However there are some complications when solving these equations. The needed solution; the surface elevation, is also the surface boundary condition for solving the equations. To overcome the non-linear processes, a simplification is made by linearizing the surface boundary conditions. In combination with the assumption of a horizontal bottom a solution of the equations is found in the form of a simple Fourier component which reads:

 $\eta = a \sin \omega t - kx$
in which η is the surface elevation and $k = \frac{2\pi}{L}$ the wave number and $\omega = 2\pi f = \frac{2\pi}{T}$ the angular frequency. The surface elevation propagates with the phase velocity $c = \frac{L}{T} = \frac{\omega}{k}$. Neglecting non-linearities can be seen as a good approximation for not too steep waves at deep

$$\omega = \sqrt{gk \tanh kh}$$

water. The wave dispersion relationship reads:

it can be seen as a relation between the angular velocity and the wave number. It is a function of the local water depth h and the gravitational force g. The phase velocity $c = \frac{\omega}{k}$ is then:

$$c = \frac{gT}{2\pi} \tanh kh = c_0 \tanh kh = c_0 \tanh 2\pi h/L$$

This velocity is the rate at which any phase of the wave propagates in space. If we look at Figure 2.7, a transition can be seen between shallow, intermediate and deep water. Looking at the function tanh kh, if kh \gg 1 than the function tanh kh is 1 which results in the deep water phase celerity of $c_0 = 1.56T$ as a function of T. This is called the deep water or "short wave" approximation and holds for kh > π or h/L > 0.5. It is found that for wind waves generated in oceanic waters, for which the "short wave" approximation holds, the phase celerity is linear dependent on the wave period. So this means that longer waves propagate faster than shorter waves. Waves travelling at different speeds can occur for example in a wind wave field; the separation of those waves is called frequency dispersion. And ocean wave are highly dispersive.





Looking at section between 0 and 0.5 in Figure 2.7 the part tanh kh in the dispersion relation equals kh for kh $\gg 0$. The celerity of the wave in this part reduces to $c = \sqrt{gh}$. This means that if the wave is long enough, according to kh < 0.31 or h/L < 0.05, the wave celerity is only dependent on the local water depth. And since the wave is not dependent on the wave period, the waves in this region are called non-dispersive.

2.6.2 Shoaling

Shoaling is the effect of waves entering shallow water increase in wave height. According to the dispersion relationship when the water depth decreases the propagation speed and wave length also decreases. This is a result of the waves being affected by the bottom and this process starts about when the water depth becomes half of about the wave length (see 2.6.1).

Since the propagation speed of the individual wave crests slows down and reaches the same speed as the group velocity, and remembering that the energy of the waves travel with the group velocity. A compensation of conservation of energy results in a higher wave height. Theoretically the amplitude of the waves go to infinity, since this is not possible in reality, the waves will lose their energy due to dissipation by wave breaking.

2.6.3 Refraction

Considering an obliquely incident wave approaching from deep water to the shore, the wave is longcrested and the bottom contours are straight and parallel to the coast. The wave is in the shoaling region and outside the breaker zone.

Where the waves approaching the underwater contours at an angle, the part were the crest travels in deeper water travels at a higher speed than the part of the wave in shallower water since in this zone the wave celerity if dependent on the local water depth: $c = \sqrt{gh}$. This results eventually in the turning of the wave crest towards shallower depth contours.

2.6.4 Diffraction

If waves interact with obstructions such as breakwaters, offshore islands or abrupt changes in the bottom contours, a large variation of wave energy along a wave crest occurs which leads to transfer of energy along the wave crests. This effect is called diffraction. The waves in the lee side of a obstruction will bend and propagate further in the "shadow zone". Due to the lateral wave energy

transfer, the wave energy is lower in the diffraction zone and therefore lower wave heights are observed in this area.

2.6.5 Wave breaking

Described in the effect of shoaling the waves can go theoretically to infinity near the coast, this only holds in the absence of a physical limit to the steepness of waves. It is found that wave crests become unstable when the particle velocity exceeds the velocity of the wave crest, the result is that the waves start to break. This breaking condition corresponds to a crest angle of about 120°.

In 1944 Miche expressed the limiting wave steepness based on the Stokes wave theory, which is a non-linear expression of the linear Airy theory that better describes steeper waves. The general formulation reads:

$$\left[\frac{\mathrm{H}}{\mathrm{L}}\right]_{\mathrm{max}} = 0.142 \mathrm{tanh}(\mathrm{kh})$$

In deep water this equation reduces to

$$\left[\frac{H_0}{L_0}\right]_{max} = 0.142 \approx \frac{1}{7}$$

And in shallow water becomes:

$$\left[\frac{H}{L}\right]_{max} = 0.142 \frac{2\pi h}{L} \approx 0.88 \frac{h}{L}$$

Which is equivalent to the breaker index:

$$\gamma = \left[\frac{H}{h}\right]_{max} = \frac{H_b}{h_b} \approx 0.88$$

The breaker index shows that in the shallow near-shore zone wave-breaking of individual waves starts when the wave height becomes greater than a certain fraction of the water depth. This is called the depth-induced breaking since the limiting wave height is governed by a water depth limitation.

2.6.6 Effect of bed slope on breaking process

An important parameter concerning breaking process due to bed slope is the Iribarren parameter which reads:

$$\xi = \frac{\tan \alpha}{\sqrt{\frac{H_0}{L_0}}}$$

where $\tan \alpha$ the steepness of the beach represents, L_0 the wave length in deep water and H_0 the deep water wave height is.

Four types of beaches are considered in the range of four different Iribarren numbers. A distinction is made between surging, plunging, collapsing and spilling beaches.



Figure 2.8: Breaker types based on Iribarren numbers

2.6.7 Radiation stress

Longuet-Higgins and Stewart (1964) defined "radiation stress" as the excess momentum flux due to the presence of waves. It is proportional to the mean energy density of the waves; E. Waves can change the momentum through net inflow or outflow of momentum, either by net inflow or outflow of momentum with the particle velocity or via a net wave-induced pressure force.

Changes in radiation stresses can cause the following processes:

- lowering the mean water level in the shoaling zone,
- raising the mean water level in the surf zone,
- driving a longshore current in case of obliquely approaching waves to the shore.

The radiation stresses consist of two components:

- the transfer of momentum ρu through a plane with the particle velocity normal to that plane,
- the wave-induced pressure force acting on the plane due to the wave induced pressure
 pwave

When considering waves approaching with an angle with the x-axis (x is the direction normal to the coast), the induced particle velocity has two components, one in the x and one in the y-direction. These particles transport both x-and y-momentum respectively ρu_x and ρu_y . The radiation stress is obtained by integrating the transport of the momentum through the entire plain over the depth. Averaged in time yields for the total wave-averaged transport of x-momentum in x-direction or radiation stress S_{xx} :

$$S_{xx} = \int_{-h}^{\eta} (\rho u_x) u_x dz + \int_{-h}^{\eta} p_{wave} dz$$

Similar for the x-momentum in the y-direction, also known as the shear component, S_{xy} :

$$S_{xy} = \int_{-h}^{\eta} (\rho u_x) u_y dz$$

This only is the case for waves approaching under an angle, for normally incident waves in the xdirection u_y is zero and hence S_{xy} is zero.

Looking at a plane in the y-direction, along the coast, the radiation stress S_{yy} reads:

$$S_{yy} = \int_{-h}^{\eta} (\rho u_y) u_y dz + \int_{-h}^{\eta} p_{wave} dz$$

And the shear component:

$$S_{yx} = \int_{-h}^{\eta} (\rho u_y) \, u_x dz$$

For waves approaching normal to the coast, the velocity component u_y is zero. S_{yy} reduces to the pressure part, and the shear component S_{yx} is also zero.

Combining the linear wave theory for expressing the radiation stresses, in which the pressure part is equal to $\left(n - \frac{1}{2}\right)E$ and the advective part, the part due to the transport of momentum by the particle velocity, is nE.

The radiation stresses can be expressed as:

$$S_{xx} = \left(n - \frac{1}{2} + n\cos^2\theta\right)E$$
$$S_{yy} = \left(n - \frac{1}{2} + n\sin^2\theta\right)E$$

$$S_{xy} = S_{yx} = (n \cos \theta \sin \theta)E$$

2.6.8 Wave-induced forces

The radiation stresses acting in a vertical plane can vary in the horizontal direction. These variations give rise to a net wave-induced force. The net force in x-direction is described by:

$$F_{x} = -\left(\frac{\partial S_{xx}}{\partial x} + \frac{\partial S_{xy}}{\partial y}\right)$$

This is the force working in the cross-shore direction of the coast. For an alongshore uniform coast, the gradients in the y-direction are zero, then only the first term $-\frac{\partial S_{xx}}{\partial x}$ in F_x represents the force.

In the alongshore direction the force is:

$$\mathbf{F}_{\mathbf{y}} = -\left(\frac{\partial \mathbf{S}_{\mathbf{y}\mathbf{y}}}{\partial \mathbf{y}} + \frac{\partial \mathbf{S}_{\mathbf{y}\mathbf{x}}}{\partial \mathbf{x}}\right)$$

The same as for the force in the x-direction, if there's no gradient in the y-direction, the force contains only the first term and reads $-\frac{\partial S_{yy}}{\partial y}$.

2.6.9 Longshore sediment formulations

There are plenty of longshore sediment transport formulas. CERC is probably the most well-known formula. The CERC formula gives the bulk longshore sediment transport over the breaker zone, due to the action of waves approaching the coast under an angle. The CERC formula takes into account the breaker index, the significant breaking wave height and the wave angle at breaking.

The Kamphuis (1991) formula includes dependency between wave period, grain-size and beach slope.

Bayram, et al. (2007) developed a formula which is based on the fact that breaking waves mobilize the sediment, which is transported by a mean current.

CERC
$$Q = \frac{K_l}{16(s-1)(1-p)} \sqrt{\frac{g}{\gamma}} H_b^{\frac{5}{2}} \sin(2\theta_b)$$

Kamphuis (1991)
$$Q = \frac{2.27}{(\rho_s - \rho)g(1-p)} H_b^2 T_p^{1.5} i^{0.75} D_{50}^{-0.25} \sin 2\theta_b^{0.6}$$

Bayram, et al. (2007)
$$Q = \frac{\epsilon(Ec_g)_b \cos \theta_b}{(\rho_s - \rho)(1 - p)gw_s} V$$
$$V = \frac{5}{32} \pi \frac{\gamma \sqrt{g}}{c_f} \sin \theta$$

3. St Lucia environment

3.1 Study area

The St. Lucia estuarine lake system is located in the South African province Kwazulu-Natal which is located on the south-east coast of South Africa. The lake is situated between 27° 52'S to 28° 24'S and 32° 21'E to 32° 34'E. The lake is connected with a 21 km long channel, known as the St Lucia Estuary, which ends up in the Indian Ocean. South of the St Lucia inlet the Mfolozi River mouth is situated.



Figure 3.1: Overview location study area (Google earth)

In this study the focus is on the tidal channel between the Indian Ocean and the St Lucia lakes; the St Lucia Estuary, also known as "The Narrows" and especially focuses on the inlet channel which forms the entrance to the ocean. It also examines the influence of the Mfolozi River discharge. The entrance channel is the relatively narrow channel that transfers the tidal flow from ocean to the tidal basin and back again. The St Lucia inlet is currently closed and has been closed since 2002 and has been closed often before. Longer periods of droughts occur frequently in this area resulting in low riverine flows into the lake, and thereby resulting in low lake water levels.

For the purpose of this study two different states of the St Lucia inlet can be distinguished; a combined mouth and a separate mouth. When the system is combined; the Mfolozi River flows into the St Lucia estuary which connects with the Indian Ocean. In a separate state the St Lucia Estuary mouth is open and connected to the Indian Ocean while the Mfolozi River mouth flushes through a small channel into the ocean. The total length of the St Lucia Estuary is about 21 km long and very shallow with a mean depth of 1 meter. Tidal influence extends approximately ³/₄ of the length of the estuary.

3.2 Background of the study area

The St Lucia lake system is the largest estuarine lake in Africa and is part of South Africa's first World Heritage Site since 1999. It is currently known as the iSimangaliso Wetland Park which consists of the lake system; North Lake, South Lake, False Bay, St Lucia estuary and the wetlands. The World Heritage status gives recognition to the area's ecological processes, biodiversity, conservation history and outstanding natural beauty. The Park has a total area of 239,566 ha and includes unspoiled marine, coastal, wetland, estuarine and terrestrial ecosystems. Important elements in these systems include coral reefs, long sandy beaches, extensive coastal dunes, estuarine and freshwater lakes, inland dry savannah and woodlands and wetlands of international importance.

The presence of all these different environments provides critical habitats for a wide range of wetland, ocean and savannah species. A large diversity of species can be found in this area due to the presence of transitional and coastal location between tropical and temperate regions. Besides its World Heritage Site status which resulted in a global recognition, it also contains Ramsar sites, which are wetlands of international importance. One of these Ramsar sites is the St Lucia Lake system.

Turpie et al., (2002) assessed the importance of the St Lucia lake system according to the South African Estuarine context. St Lucia was ranked 5th out of 246 estuaries in South Africa in terms of its

Conservation Importance Rating, thereby contributing 44.9% to the calculated Estuarine Biodiversity of South Africa. It was also ranked 5th for its overall importance classified by the Estuarine Importance Rating. It was assigned the first position for its Botanical Importance Rating, first for its Fish Importance Rating and first in terms of its Bird Community Species Index (Turpie et al., 2002). Besides its importance in South Africa it is also the largest of only three estuarine coastal lake systems in the country. The system accounts for about 80 per cent of the estuarine area of the southern African sub-tropical region and 60 per cent of the estuarine area of the country, which makes the lake system the most important nursery ground for juvenile marine fish and prawns along the east coast.



Figure 3.2: The St Lucia estuarine lake system, with the combined and separated inlets (Lawrie and Stretch, 2011)

The St Lucia lake system (Figure 3.2) originates from erosion during past marine regressions and the following infilling with sediments, mainly during the Holocene transgression (Orme, 1973). Approximately 18000 years ago during the Glacial Maximum sea-level used to be 120 meter below current level. Sea-level rose until about 6000 years ago and reached its current level and remained more or less stable since then (Ramsay, 1996). After the rising of the sea-level, the total water surface area of the St Lucia lake was about 1165 km², and the length was about 112 km with water depth up to 40 meters deep (Orme, 1990). During the late Pleistocene and Holocene the St Lucia lake system accumulated large amount of sediments and transformed from a deep-water lake to a shallow estuarine lake (Van Heerden, 1976). Most parts of the lake have become very shallow with an average depth of < 1 m.

From Whitfield and Taylor (2009), looking at the period before 1950 when there was no human interference in the mouth, the St Lucia inlet used to be connected to the St Lucia Bay in which the Mfolozi River also entered (Figure 3.3a). In wet periods the input of fresh water was maximal and the water flowing from the Mfolozi River entering the Bay used to pass out to the sea on the ebb tide. In dry periods the joint St Lucia and Mfolozi mouth would tend to close and in this state, the Mfolozi River naturally diverted into the St Lucia lake system. The fresh water flowed through the Narrows into the St Lucia lake system and restored most of the water that was lost through evaporation. In this state the system would have a low probability of experiencing extreme hypersalinity (>60psu), especially South Lake where the effect of refreshing the water is the most effective (Taylor, 1993).

After the drought period, the rivers started to flow resulting in the water level within the whole system to rise and back up into the adjacent swamps. The water in front of the beach berm that was formed across the St Lucia inlet would then gradually back up until it reached levels about 3 to 3.5 m. before overtopping the berm (Huizinga and van Nierkerk, 2005). Another way of breaching the berm and opening the St Lucia inlet was possible by a flood; this flood was calculated to have a return period of 3-years (Lawrie and Stretch, 2008). When a breaching event occurred a large amount of water would escape through the inlet and maximum outflows of 1000 m³ per second would have been present, thereby eroding accumulated sediments in the St Lucia Bay.

With a closed mouth, all the backed up water would have expanded into the lower lying areas and flooding much of the Mfolozi and Mkhuze swamps. The large mean annual runoff of 920 x 10⁶ m³ of the Mfolozi River would make it unlikely to have the St Lucia Estuary mouth be closed for longer periods than 2 years (Huizinga and van Nierkerk, 2005).



Figure 3.3: Configuration of the St Lucia Estuary mouth region in (a) 1883/4 (modified from Methven, 1903), (b) 1937 and (c) 1966 (source aerial photography: Chief Directorate Surveys and Mapping, South Africa). From (Whitfield and Taylor, 2009)

Anthropogenic impacts started early in the 19th century. Around 1920 sugar cane farming became popular in the Mfolozi floodplain and swamps. The swamp areas were drained, and to prevent the land of low-lying farms from flooding, canals were excavated to remove floodwaters from the floodplain to the sea. Before sugar cane farming started, the Mfolozi swamps acted as a huge filter and sponge, catching sediments and releasing relatively fresh water into the St Lucia lake system.

During draught periods, this fresh water would prevent the St Lucia Lake system from becoming extreme hyper saline (Taylor, 1993).

The main excavated channel, Warner's Drain, was dug in the early 1930s through the Mfolozi swamps. Sedimentation of the St Lucia Bay was one of the consequences (Figure 3.3b). Around the 1940s the large sedimentation rates was of a big concern. In the drought of 1950 the whole mouth area was silted up, and the closure of the combined mouth was a fact. To save farmers from flooding their lands by backed-up water of the Mfolozi River, a canal was dredged through to the sea; by this the Mfolozi Estuary mouth was created. The new dredged mouth was located 1.5 km. south of the St Lucia Estuary mouth (Figure 3.3c). The Mfolozi mouth used to migrate at a speed of about 60 m a month northwards to join up with the St Lucia mouth. To keep the mouths separate, every few years

or two dredging at the original mouth location is necessary. Without the Mfolozi River inflow, the most important source of fresh water of the St Lucia Lake was abandoned. After the closure in 1950 it took dredgers 6 years to dredge the sediments away in order to open the St Lucia mouth. Additionally it took 8 years to dredge other marine and riverine sediments in the estuary and the Narrows. Management policy changed after this event to the idea that the mouth should be kept open at all times to maintain an estuary-sea link for fish and invertebrates; hence an on-going continuous dredging programme was established. Besides the dredging programme, the St Lucia inlet was stabilised by hard structures on both banks. The desired effect of keeping the mouth open was not successful, the self-scouring effect in the inlet did not function as planned and during drought periods the mouth still closed. In 1984, the hard structures and the dredger were washed away during a large flood associated with Cyclone Domoina. These hard structures were not replaced after the event (Van Heerden and Swart, 1986).

Drought periods are a well-known phenomenon in the northern part of KwaZulu-Natal and can last for many years. In the period 1967-1972 a severe drought occurred, the St Lucia mouth closed naturally but was dredged open artificially. The rivers entering St Lucia Lake stopped flowing and water was lost through evaporation. Seawater flowed into the system and replaced the evaporative losses with salt water. Marine sediments entering the inlet were calculated to be in the order of 240,000 m³ per year. Salt budgets suggest in the order of 20 million tonnes added to the existing salt loading within the lake. In this period the salinity in the lake rose to extremely high values > 100 psu in North Lake and False Bay. In combination with high water levels, maintained by the inflow through the mouth, the shoreline vegetation was severely affected. Plants that held the banks together were killed and erosion of islands took place that were important breeding sites.

Many attempts have been made to alleviate the hypersalinity of the St Lucia Lake. One of them was to excavate a canal through the upper part of the Mkhuze swamp, to let the water flow directly into the lake instead of flowing through the swamp. It was only a little fresh water so this did not lower the salinity in the North Lake sufficiently. For the swamp problems arose because parts of it dried up and severe erosion took place of the sides of the canal. Reinstating the link with the Mfolozi River was largely overlooked, as a fresh water source for the system.

In Table 3.1 an overview of the freshwater inputs of the St Lucia Lake system can be found and it can be seen that the Mfolozi River had the largest contribution.

Environmental component	Freshwater budget
Mean annual evaporation	-420 x 10 ⁶ m ³
Mean annual rainfall	+273 x 10 ⁶ m ³
Mean annual run-off of five rivers entering St Lucia	+362 x 10 ⁶ m ³
Mean annual groundwater inflow	+23 x 10 ⁶ m ³
Mean annual run-off of diverted Mfolozi River	920 x 10 ⁶ m ³

Table 3.1: Freshwater inputs and evaporation within the St Lucia system in an average year (modified from Huizinga and van Niekerk 2005). Note that before 2008 no Mfolozi River water entered the St Lucia system. By (Whitfield and Taylor, 2009).

After 1950 a lot of experience was gained from the drought periods, the scientific knowledge that was obtained led to a new management policy; at the start of the drought in 2001 the decision was taken to not artificially open the St Lucia mouth by dredging if the mouth closes. Then in 2002 the mouth closed naturally following the drought. At that time the measured salinity was about 14 psu which equates to a total mass of salt in the lake of about 4.9 million tonnes (Bate and Taylor, 2008). The severity of the drought caused the rivers entering the St Lucia Lake to dry out and stop flowing into the lake until March 2012 when heavy rainfalls occurred associated with Cyclone Irina.

In March 2007 the St Lucia sand berm was breached by particularly high tides combined with high waves generated by Cyclone Gamede. This event opened the mouth naturally by overtopping and eroding the inlet gorge, and allowing sea water to enter the St Lucia Lake system. This connection remained open for 175 days before it closed naturally. After closure in August 2007 the salinity was estimated to be 30 psu with a salt loading in the order of 9 million tonnes, this was double when the mouth closed in 2002.

A new management strategy following the onset of the drought since 2002 and the high salinity levels in the lake, decided to divert the Mfolozi River to let winter low-flows into the St Lucia Estuary. To make this possible three steps had to be undertaken before the link could be made.

First the sand bar on the beach between St Lucia and Mfolozi mouth (Figure 3.4c) had to be strengthened to prevent breaching by marine overwash. Next a small channel for the Mfolozi was dredged (Figure 3.4a), in case the Mfolozi received unseasonal heavy rains and the river came down in floods or if the water levels rise too high for sugar cane farming. The third step was to regulate the constricted section of the Back Channel (Figure 3.4b), this connection needed to be managed so that it could be closed off in case the Mfolozi came down in flood.

The idea was to let in low flows which carries relatively little sediments. This is possible when the Mfolozi mouth is closed, and water backs up in the Mfolozi swamp and floodplain. The sediments downstream of the river will settle because of the river flow entering an area with low waterlevel gradients. In this way sediment free-water can flow through the Back Channel which was originally an old mangrove-lined channel that was excavated in 1960. In the period from May 2008 to December 2008 roughly 15 million m³ of Mfolozi water entered St Lucia, but most of the water only reached the Narrows and South Lake. From September to January 2009 another 1.5 million m³ of Mfolozi water entered the St Lucia Lake system. Management was content with the result of freshwater of the Mfolozi River flowing into the St Lucia Estuary and lowering thereby salinity levels in The Narrows.



Figure 3.4: Photo of the Mfolozi and the St Lucia mouth region showing the main management interventions to bring Mfolozi River water to the St Lucia system. (a) Link. (b) Back Channel. (c) Reinforcement. (Photo: R. Taylor)

3.3 Climate conditions

According to Van Heerden (2011), the South African province KwaZulu-Natal, situated between latitudes 27°S and 31°S has a subtropical coastal and temperate inland climate. The occurrence of thunderstorms is not unusual and is common in the summer (October-March). Mid-latitude cyclonic activity contributes to the weather pattern in the winter (April-September).

Precipitation is the most essential climatic variable in this environment. On average the yearly rainfall in the province is 850 mm, although this amount is not equally distributed over the whole area.

3.3.1 Wave climate

Along this section of the South-African coast the most important marine physical forcing is the wave climate. Wave height and wave direction are the two most important characteristics in terms of beach and spit responses. Waves breaking at an angle induce a longshore current which generates longshore sediment transport. The available wave data are measured at Richards Bay which is 50 kilometres south from the St Lucia Inlet.

Ocean swell waves approach the coast with a mean wave direction of 138 degrees, this value is measured over the period 1999-2009. The overall average significant wave height over this period is 1.58 m. The average peak period over this period is 11 sec.



Figure 3.5: Wave rose 1999-2009

From the overall data also a typical year was selected, this is the 2006 wave climate. The wave rose in this year is quite similar to the overall wave rose. In Figure 3.6 seasonal wave roses are presented. The yearly averaged wave height, wave direction and wave period are: 1.60, 140 degrees and 11 seconds.

The seasons in South-Africa are a little bit different from Europe, autumn is from April to end of May, winter from June to end of August, spring from September to end of October and summer takes five months from November to end of March. An obvious seasonal wave climate can be observed with highest waves in the spring and winter and moderate wave heights from summer to autumn. The mean significant wave height in spring is 1.73 m. for the other seasons the wave heights are; winter 1.62 m., autumn 1.62 m. and summer 1.53 m. As can be seen from the wave roses the direction is most southerly directed in the winter and autumn, the corresponding mean directions are respectively 148 and 145 degrees. In the summer and spring wave directions are more south-easterly directed with 136 degrees mean.



Figure 3.6: Typical seasonal wave in 2006 measured at Richards Bay, 50 km south of St Lucia

In Table 3.2 an overview of the occurrence of wave heights and wave directions between certain ranges is given. Wave heights between 1 - 2 m occur 75.8 % of the time in the year 2006. Waves higher than 2 m. occur 18.2% of the time. And waves higher than 3 m. are found in 3% of time in this year. Translating the percentages of occurrence to days of the year, waves higher than 3 m. are found 11 days in a year, waves higher than 2 m. occur 66 days in a year and waves between 1 and 2 meter 277 days in a year.

The direction of the offshore swell waves is in 64.6% of the time higher than 135 degrees, which is 45 degrees from shore normal. In 32.8% of the time the direction is between 150-160 degrees, this can be seen as the dominant wave direction. This means that the offshore waves approach the coast under an angle of 60 - 75 degrees. This doesn't mean the waves actually break at this angle, because of refraction the actual direction at the shore is much less than the offshore directions.

Wave	dir (deg)	60	75	90	105	120	135	150	165	180	195	210	225		
Hs (m)	75	90	105	120	135	150	165	180	195	210	225	240	Total	%
0,5	1	0	2	8	16	11	34	69	28	0	0	0	0	168	5,8%
1	1,25	4	19	72	81	55	91	211	58	1	0	1	0	593	20,4%
1,25	1,5	7	37	103	105	70	141	262	73	2	2	0	1	803	27,6%
1,5	1,75	7	10	60	65	62	89	158	39	8	4	2	2	506	17,4%
1,75	2	0	2	31	28	40	61	87	29	12	8	3	0	301	10,4%
2	2,25	0	0	10	10	11	59	53	21	9	5	0	0	178	6,1%
2,25	2,5	0	0	11	13	9	26	30	14	11	2	0	0	116	4,0%
2,5	2,75	0	0	4	17	12	18	27	9	7	0	0	0	94	3,2%
2,75	3	0	0	3	4	4	12	22	8	1	0	0	0	54	1,9%
3	3,25	0	0	3	5	5	7	16	7	2	0	0	0	45	1,5%
3,25	3,5	0	0	2	3	1	8	6	2	2	0	0	0	24	0,8%
3,5	3,75	0	0	0	1	1	2	8	1	0	0	0	0	13	0,4%
3,75	4	0	0	1	2	0	1	2	1	0	0	0	0	7	0,2%
> 4		0	0	0	0	0	0	3	1	0	0	0	0	4	0,1%
	Total	18	70	308	350	281	549	954	291	55	21	6	3	2906	
	%	0,6%	2,4%	10,6%	12,0%	9,7%	18,9%	32,8%	10,0%	1,9%	0,7%	0,2%	0,1%		

Table 3.2: Wave height and direction (2006), values are the 3-hrs intervals

In Table 3.3 the peak period with the corresponding wave height is given. In 53% of the time the period is higher than 11 seconds. The period is in 27% of the time between 8-10 seconds and in 38% of the time between 11-14 seconds. It can be seen that higher waves correspond to the longer wave periods.

Тр (s)	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19		
Hs (m)	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20	Total	%
0,5	1	0	0	0	6	7	15	13	16	42	22	22	19	6	0	0	0	0	168	5,8%
1	1,25	0	8	8	22	33	105	65	37	115	70	56	43	22	7	0	2	0	593	20,4%
1,25	1,5	2	17	13	21	70	130	70	50	93	113	109	71	30	10	0	4	0	803	27,6%
1,5	1,75	0	12	11	17	32	93	64	43	70	49	39	38	27	7	0	3	1	506	17,4%
1,75	2	0	1	10	14	12	55	26	29	42	33	27	24	19	6	0	3	0	301	10,4%
2	2,25	0	0	6	12	9	38	18	16	28	11	12	13	11	4	0	0	0	178	6,1%
2,25	2,5	0	0	3	10	4	18	17	12	23	7	6	4	8	2	0	2	0	116	4,0%
2,5	2,75	0	0	0	5	5	13	14	15	22	10	4	1	4	0	0	0	1	94	3,2%
2,75	3	0	0	0	0	4	7	5	5	8	10	3	5	4	1	0	1	1	54	1,9%
3	3,25	0	0	0	1	0	3	3	9	10	7	4	3	3	2	0	0	0	45	1,5%
3,25	3,5	0	0	0	0	1	3	6	0	5	4	3	1	0	1	0	0	0	24	0,8%
3,5	3,75	0	0	0	0	0	0	3	2	2	1	3	0	1	1	0	0	0	13	0,4%
3,75	4	0	0	0	0	0	0	0	1	5	0	0	0	1	0	0	0	0	7	0,2%
4	10	0	0	0	0	0	0	0	0	0	0	0	0	3	1	0	0	0	4	0,1%
	Total	2	38	51	108	177	480	304	235	465	337	288	222	139	42	0	15	3	2906	
	%	0%	1%	2%	4%	6%	17%	10%	8%	16%	12%	10%	8%	5%	1%	0%	1%	0%		

Table 3.3: Wave height vs peak period (2006)

3.3.2 Tidal range and tidal prism observations

Tidal characteristics are variable worldwide. To indicate in which tidal environment the tide can be classified, two main variables can be distinguished. These two important variables are the magnitude of the tidal elevation, and the tidal character, in which a distinction can be made between diurnal and semi-diurnal components.

The tidal character can be defined by the form factor F. This is the ratio of the amplitudes of the sum of the two main diurnal components K1 and O1 and the sum of the two main semi-diurnal components M2 and S2. F reads:

$$F = (K1 + 01)/(M2 + S2)$$

From Table 3.4 the components respectively K1, O1, M2 and S2 can be read. For the St Lucia environment the form factor reads 0.078, which categorises St Lucia in a semi-diurnal tidal regime.

Tide components





Frequency (cpd)

In Figure 3.7 a graphic overview is given of the ocean tides at Richards Bay. The following data is included in this figure:

ML	1.2 m
LAT and HAT	0 m – 2.47 m
MLWS and MHWS	0.27 m – 2.11 m
MLWN and MHWN	0.97 m – 1.48 m
LATOY and HATOY	0.04 m – 2.39 m

Table 3.5: Tidal data over a 19 year period



Figure 3.7: Ocean tides at Richards Bay (Ocean Rhythm: www.satides.co.za)

The above figure shows the mean or average tidal predicted values over a 19 year cycle.

Measurements in 2007 after Cyclone Domoina flooded St Lucia were done by Chrystal (2012), he surveyed the St Lucia mouth and measured the tidal prism, peak flows and tidal range in the period from 20 April to 22 June 2007. The location of the measurements was at the bridge which is 4.5 km. from the ocean inlet connection away. In Table 3.6 the observations are presented, they show a maximum recorded tidal prism of 1,333,668 m³ during ebb and 1,600,000 m³ during flood at a spring tidal stage. In this state the tidal range at sea was 1.80 m and in the estuary 0.70 m which is only 39% of the elevation at sea. The peak flows were measured to be 131 m³/s. In comparison with measurements done by Hutchison in 1972, the tidal prism was 1,829,419 m³ during ebb and 2,067,179 m³ during flood at a spring tidal stage.

In 2007 the flood volume varied from 495,003 m³ to 1,600,000 m³, and during ebb the volume varied from 332,996 m³ to 1,333,668 m³. The net volume of water entering the estuary was in four of the measurements positive, this points towards a net import of water and hence sediments transported into the estuary.

In 1972 Hutchison measured a flood volume ranging from 1,198,524 m³ to 2,067,179 m³ and during ebb the volume varied from 1,459,033 m³ to 2,984,691 m³. Measurements on 14 July 1972 show that an amount of 1,786,167 m³ water flushed out to the ocean. This can be explained by the fact that the period 1967-1972 was an extended drought period, which ended in 1972. Subsequent heavy rainfalls and rising water levels in the lake could be the reason that during ebb stage a large amount of water flowed out to the ocean.

Data	Tidal	Peak flows (m ³ /s)		Volum	ne (m³)	Net	Tidal ra	Estuary	
Date	stage	Flood	Ebb	Flood	Ebb	(m ³)	Sea	Estuary	range/ sea
07/04/20	Spring	131.0	68.0	1,600,000	1,333,668	266,332	1.80	0.70	0.39
07/05/25	Neap	48.7	24.8	851,397	332,996	518,401	0.53	0.23	0.44
07/05/28	Mid	57.8	24.4	822,980	445,099	377,881	1.19	0.28	0.23
07/05/31	Spring	63.9	25.6	965,566	454,445	511,121	1.45	0.32	0.22
07/06/22	Neap	49.7	30.9	495,003	594,964	-99,960	0.80	0.27	0.34
72/07/11	Spring	187.0	115.0	2,067,179	1,829,419	237,760	1.67	0.87	0.52
72/07/12	Mid	176.0	100.0	1,887,813	1,586,904	300,909	1.74	0.85	0.49
72/07/13	Mid	150.0	91.0	1,388,517	1,495,033	-106,516	1.63	0.72	0.44
72/07/14	Mid	105.0	147.0	1,198,524	2,984,691	-1,786,167	1.49	0.63	0.42

Table 3.6: Tidal range and prism observations (Chrystal, 2012).

Data obtained from Hutchison (1972)

3.3.3 Cyclones

Cyclones occur quite frequently on the eastern coast of South Africa. Data from South Africa, Madagascar, Mauritius and Reunion show that since 1927 approximately 10 tropical storms are generated every year in the tropical regions of the Indian Ocean (Dunn, 1984). From all of these storms, forty per cent of the cyclones are formed in the Mozambique Channel. After 1950, twelve cyclones have caused significant rainfall (in excess of 100 mm) over the province KwaZulu-Natal.

3.4 Longshore sediment transport rates

Longshore sediment transport on the South African coast was investigated by Schoonees (2000). On three locations measurements were done, these are; Durban Bight, the sand trap of the Port of Durban and Richards Bay. Richards Bay is the most representative location to get an indication at the St Lucia Estuary for the annual longshore sediment transport rates.

The way to measure the amount of total sand transport along the coast was done by dredging sand south of the harbour entrance and pumping it onto the near-northern beach. Because usually at the net longshore transport is towards the northeast.

At this location the net longshore transport is given in Table 3.7. The results at Richards Bay show a long-term net north-eastbound longshore transport of 850,000 m³/year varying from 420,000 to 2,120,000 m³/year.

Period	Net longshore transport rate (m ³ / year)
1979/1980	1,020,000
1980/1981	830,000
1981/1982	850,000
1982/1983	2,120,000
1983/1984	1,880,000
1984/1985	1,570,000
1985/1986	-420,000
1986/1987	-260,000
1987/1988	720,000
1988/1989	1,010,000
1989/1990	-60,000
1990/1991	1,220,000
1991/1992	980,000
1992/1993	440,000

Table 3.7: Net longshore sediment transport measured at Richard's Bay (Schoonees, 2000).

3.5 Mfolozi River catchment

Taylor (2011) gave a conceptual view of the floodplain processes that affect the Mfolozi/Msunduzi Estuary and the impacts that human activities had on the natural functioning of this basin.

The Mfolozi River catchment covers a total area of 11.070 km² (Garden, 2008). The river can be classified as very steep (Figure 3.8). In its natural state it transports a considerable amount of sediments delivered upstream where erosion takes place. The river has a highly variable flow regime (Garden, 2008) which can be characterised by a low base-flow and a few large, but brief floods each year. These floods are infrequent and short of duration but when they occur they carry much of the annual river flow (Taylor, 2011).



Figure 3.8: Profile of the Mfolozi River showing the change in gradient from source to sea (Looser et al., 1985).

The Mean Annual Runoff (MAR) of the Mfolozi is estimated at 940 x 10⁶ m³. The Mfolozi runoff shows significant variation between years with floods (e.g. 1983 with total runoff of 3996 x 10⁶ m³ or 420% of the MAR and drought years showing for example in 1982 a MAR of 142 x 10⁶ m³ which is 14% of MAR. Besides the yearly large variations, significant seasonal fluctuations occur between the high summer runoff (between January and March), more or less 47% of MAR and low winter runoff (between June and August) of 7% of the MAR.

Monthly flow volumes of $10 \ge 10^6$ m³ is exceeded about 80% of the time, while higher monthly flow volumes such as $100 \ge 10^6$ m³ is exceeded only 15% of the time

3.6 St Lucia mouth states, inlet hydrodynamics and offshore bathymetry

The St Lucia Inlet and the Mfolozi mouth in combined and separate states are situated in a highly dynamic environment. Hydrodynamics control the functioning of the system and the mouth states control the overall biophysical functioning of the system. The Mfolozi and St Lucia Estuaries can be classified as temporarily open/closed or seasonally open estuaries. Inlet instability leads to a closure of the mouth because of variable terrestrial inflow, high energy wave climate, micro-tidal range and a high rate of longshore sediment transport (Ranasinghe, Pattiaratchi and Masselink, 1999) and (Parkinson and Stretch, 2007).

Lawrie, Chrystal Clinton and Stretch (2011) investigated different mouth states of the St Lucia inlet. A water balance model of the St Lucia lake/estuary system was setup, with the capability to simulate average lake water level, salinity and mouth states (Lawrie and Stretch, 2010). The simulations covered a period from 1920-2010:

1) Separate inlets with mouth manipulation: the status during the period 1952-2002

In this state, the management strategy was to keep the St Lucia inlet separated from the Mfolozi River mouth. The St Lucia inlet maintained a permanently open state to allow fish and other aquatic biota into and out of the estuary (Whitfield and Taylor, 2009). Dry periods could then be compensated by sea water inflow, this occurred when the lake level dropped below estuary mean water level. Desiccation was not a concern because sea water kept flowing into the lake, however the salt loads increased the salt levels in the lake and hypersaline conditions would typically follow, especially in the northern part of the lake.

2) Separate inlets, no artificial mouth manipulation: the status since 2002

Since 2002 the management strategy changed, mouth manipulation ended and the mouth closed naturally. The long term implications were investigated with model simulations. Simulations without the Mfolozi link, and without mouth manipulation, indicated a closed mouth (88% of the time) in this scenario.

3) Combined mouth, no artificial mouth manipulation: the status before 1952

In this mouth state, the mouth was subject to minimal human impacts. Prior to 1950 during dry conditions the combined St Lucia and Mfolozi mouth would occasionally close and that the Mfolozi

would flow into St Lucia lake refilling water lost to evaporation and diluting salinity (Whitfield and Taylor, 2009). Simulations show that with a combined St Lucia/Mfolozi mouth, the mouth would be predominantly open (about 70% of the time). Drought periods would lower the lake levels, but the sea water influx would maintain lake levels at or near estuary mean water level. Salinity levels would slowly increase. Closure of the mouth could occur in extreme dry periods and the Mfolozi Rivier would then be diverted into the St Lucia lake, maintaining or increasing the water level and lowering the salinity levels.

In Figure 3.9 two pictures are presented in which the complex behaviour of the combined and separate mouths is visible. In Figure 3.9(a) a clear view of a well-developed flood delta and meandering mouth configuration is given. The northward migration of the spit barrier due to longshore sediment transport and the formation of a flood delta due to flood-tide induced sediment inflows are depicted in Figure 3.9(b).



Figure 3.9: Combined St Lucia and Mfolozi system (a), and an artificially separated St Lucia Inlet (b) (pictures courtesy of R. Taylor)

According to Bruun (1978) it is well known that large terrestrial flows and or a large tidal prism can maintain an open inlet by overcoming the factors that drive inlets to close, these factors are mainly wave and tide driven sediment transport processes.



Figure 3.10: St Lucia inlet 2007 by (Tayler, 2007).

The tidal inlet channel is commonly 75 - 150 m wide, and forms the only connection with the sea. Although there is very detailed bathymetry on the seaward side of the St Lucia Estuary mouth, this bathymetry is very old and originates from 1986, the period after cyclone Domoina opened the St Lucia mouth naturally. Since its location is situated in a highly dynamic environment it can be expected that the bottom profile is continuously changing due to the available sediment transport and Mfolozi River that discharges into the ocean. The bathymetry shows a steep profile with a mean slope of 1:100 m. stretching to the 3000 m. offshore depth contours. But close to the beach the profile is steeper and varies from 1:50 to 1:75.

According to Wright and Mason (1991) the grain size diameter is found to be in the range of 100 to 300 microns.



Figure 3.11: Offshore bathymetry in Delft3D



Figure 3.12: St Lucia offshore bathymetry data (I.L. Van Heerden 1986)

3.7 Summary of the study area

- Tidal prism observations at St Lucia are in the range of 300,000 m³ to 3,000,000 m³
- Measured longshore sediment transport at St Lucia are in the range of 420,000 m³/year to 2,120,000 m³/year and a net long-term amount of 850,000 m³/year
- From the tidal components it is found that the St Lucia environment is in a semi-diurnal tidal regime. The mean tidal range is 0.67 m. and the range from MHWS to MLWS is 1.84 m. and the range from MHWN to MLWN is 0.51 m
- Grain size diameters are in the range of 100 to 300 microns
- St Lucia Estuary dimensions; the length of the estuary is 21 km., the average width is 500 m. and the depth is on average 1 m
- St Lucia inlet dimensions; the inlet is commonly 75 -150 meter wide
- Mfolozi River discharge is yearly averaged 940 x 10⁶ m³, which is 30 m³/s
- Closure time after open mouth state is 175 days, the mouth closed in August 2007 before it was breached open by high waves (Cyclone Gamede)
- the P/M ratio according to Bruun (1978) is in the order of 2 which classifies the inlet in the range < 20 according to Bruun and Gerritsen (1960). These types of inlets are unstable. And the inlet may be closed by deposition of sediment during a storm
- According to the hydrodynamic classification (section 2.3), with a mean spring tidal range of 1.84 m. and a significant wave height of 1.58 m. the St Lucia inlet can be placed into a mixed energy (wave dominant) environment
- Episodic effects of cyclones
- Highly variable freshwater inflows; inter-annual and large seasonal variation
- The latest management policy is central in this research; the Mfolozi River has the possibility to join with the St Lucia Estuary and the St Lucia inlet functions in its natural state, no artificial mouth manipulation (dredging) takes place
- The St Lucia inlet has two different mouth states; a combined mouth by which the St Lucia Estuary and Mfolozi River share a common inlet to the sea. And a separate mouth such that the St Lucia inlet is kept separate from the Mfolozi River mouth

4. Delft3D Model Setup

4.1 Model setup

Delft3D developed by WL|Delft Hydraulics (now Deltares), is a fully integrated computer software package for a multi-disciplinary approach. It is developed for simulating 1D, 2D and 3D computations for coastal, river and estuarine areas. The computations that can be carried out are simulations of flow, sediment transport, waves, water quality, morphological development and ecology. The Delft3D program consists of several modules which are capable to interact with each other. In this study the modules used are Delft3D-WAVE, Delft3D-FLOW and Delft3D-MOR.

The model represents a schematised configuration of the St Lucia Estuary mouth. Numerous simulations will be made forced with different wave heights and a harmonic tide. This setup results in different P/M ratios (Table 3.6) covering the observed ratio at St Lucia. The computational model is presented in Figure 4.3; three different models (Scenario-a, Scenario-b, Scenario-c) are used to examine the stability of the inlet. The models have different basin area sizes: 0.5 km², 1.65 km² and 3.25 km², these values result from the following basin surface dimensions; a length of 3.5 km. in all the three scenarios and a width of 150 m, 500 m. and 1000 m. In the next paragraph a more detailed description of the basin area sizes is given. The basin has a uniform depth of 1 m. below mean sea level (MSL). The inlet which connects the basin with the ocean is 100 m. long and 150 m. wide. The sides of the inlet channel have been smoothened to a trapezoidal shape. The bottom profile at the seaward side has a concave equilibrium profile (Dean, 1991) in which the mean slope of the actual bathymetry at St Lucia is approached (section 3.6). The height of the barrier islands is set to +3 m above MSL.

In this study the Delft3D-FLOW module has been used with a depth averaged approach to solve the shallow water wave equations. The 3D-advection-diffusion equation as described in Lesser et al. (2004) is approximated by a depth-integrated advection-diffusion equation. A limitation of a depth-averaged approach is that the process undertow is not taken into account. Undertow is the flow under the wave crest which direction is offshore; it compensates the amount of cross-shore sediment transport onshore directed, by transporting it back offshore.

In contrast with this limitation of Delft3D, cross-shore wave processes are not taken into account, this effect is switched off by the input parameters SusW and BedW (Appendix 8.5). The currents in the surf zone generate the longshore sediment transport; the two main parameters controlling the

amount of transport in Delft3D are SusC and BedC (section 8.5). Both SusC and BedC are set to 5 which result in a multiplication of the currents in the surf zone, this is done to model the same amount of longshore sediment transport as observed at St Lucia (section 3.4). In Appendix 8.7, this influence is tested through several simulations and therefore decided to totally switch off the cross-shore transport.

The transport of sediment is calculated by the method of Van Rijn (1993). As stated in the Delft3D-FLOW user manual, the Van Rijn formula for sediment transport is a well-known approach and often used in coastal areas. The formulation accounts for both bed-load and suspended-load and it distinguish the two by a reference height at which below is treated as bed load and above is treated as suspended-load.

The grid used to calculate the hydrodynamics and morphology (flow grid) and the grid used for the evolution of the waves (wave grid) are presented in Figure 4.1. The length and width of the flow grid is 6000x7000 m. To get a certain realistic accuracy of the bed level evolution the grid cells in the area of interest has to be small enough. Therefore in the surf zone and at the location of the tidal inlet and basin the grid cells are at smallest 15x15 m. up to 200x100 m. at the boundaries of the grid.

The wave grid is copied from the flow grid and extents on the north and south boundaries with 6000 m. In the area of interest the wave grid correspond to the flow grid, with the same grid cell sizes. On the extended part the grid cells smoothly become larger to a maximum size of 600x100 m.



Figure 4.1: Wave grid (green) and hydrodynamic/morphologic grid (blue)

The time step of the Delft3D-FLOW controls both numerical stability and the accuracy of flow results. The Courant number is a useful relation between the time step and the grid cell size, and as a rule of thumb should not exceed a value of ten to secure a stable and accurate simulation. The Courant number reads:

$$CFL = \frac{\Delta t \sqrt{gH}}{\{\Delta x, \Delta y\}}$$

Where Δt is the time step [s], g is the acceleration of gravity [m/s²], H is the total water depth [m], and { Δx , Δy } is a characteristic value (in many cases the minimal value) of the grid spacing in either direction [m].

Several trial-and-error test simulations were performed with different time steps to discover the needed time step for accurate simulations. The required value of the time step is set to 6 seconds which is a conservative value. The corresponding Courant numbers in the area of interest are smaller than five.

In the model three open boundaries are applied at the flow grid. At the two lateral boundaries (the north and the south boundary), Neumann boundary conditions are imposed. This type of boundary condition is used to specify an alongshore water level gradient. On the seaward side a water level boundary is imposed with as forcing type a harmonic tidal signal. No gradient in the water level is caused due to tide therefore on the Neumann boundaries the tidal signal is set to zero.

The coupling between the FLOW and WAVE module is done with the so called 'online' approach. In this approach the flows, sediment transport and bottom updating are all calculated at the same time step. In this method the difference in time scales between flow and morphology are taken into account by the introduction of the morphological factor (Lesser et al., 2003). This factor simply multiplies the bed change rates by a constant factor.



Figure 4.2: Online coupling approach between FLOW and WAVE with a morphological scale factor

The morphological scale factor is set to 30 in combination with a simulation period of 10 days the total morphological simulation of the bed evolution is 300 days. This time-scale covers the expected time of closure of the St Lucia mouth system.



Figure 4.3: The three schematised models showing the flow domain and bathymetry representing the St Lucia Estuary mouth, the legend bar shows the depth below MSL

4.2 Scenarios

In addition to the model setup, the three scenarios (Figure 4.3) are described more in detail in this section. First of all the chosen scenarios are a result of different aspects. An important aspect for realizing the different models is the tidal prism data measured in 2007 while the mouth was at open state (section 3.3.2). Even though the data can be considered as not fully reliable due to eventual inaccurate measurement methods, it is at least a good indication and order of magnitude of what the actual data might be. Therefore this data will be used for the design dimensions of the basin surfaces of the different scenarios. The tidal range at the ocean and sea was also measured (section 3.3.2) and from Table 4.1 an indication of the estuary surface area can be found by dividing the total volume of flows during an ebb or flood cycle by the tidal range. In this table the surface area is calculated from the tidal range in the estuary. The obtained values of the basin surface areas, measured with the flood volume, are in the range of 1.8 km² to 3.7 km² while for the ebb volume this range is from 1.4 km² to 2.2 km². This is an important indication for the dimensions of the model which is used in Delft3D.

Tidal	Volume (m ³	Tidal ra	ange (m)	Surface area (Ratio tidal		
stage	Flood	Ebb	Sea	Estuary	Estuary flood	Estuary ebb	range
Spring	1,600,000	1,333,668	1.8	0.7	2,285,714	1,905,240	0.39
Neap	851,397	332,996	0.53	0.23	3,701,726	1,447,809	0.43
Mid	822,980	445,099	1.19	0.28	2,939,214	1,589,639	0.24
Spring	965,566	454,445	1.45	0.32	3,017,394	1,420,141	0.22
Neap	495,003	594,964	0.8	0.27	1,833,344	2,203,570	0.34

Table 4.1: Tidal prism (Volume), tidal range and basin surface areas

Looking at the dimensions of the St Lucia estuary, the length is 21 km. and the width is on average 500 m. The deepest part is in this shallow estuary is 1 meter. At the end is a threshold, which separates the lake from the estuary. This is mainly because of the low lake water levels, as a result of longer drought periods. As can be seen from the table above, the tidal prism is quite small and the corresponding surface area is also smaller than the actual dimensions and surface area of the estuary. The ratio of the tidal range between sea and estuary is an important factor for this difference; since the tidal range in the estuary is much smaller than at sea. Another reason from the observed dissipation of the tide could be due to bottom friction at the inlet and due to entry and exit expansion losses at the inlet.

In Figure 4.4 the used basins for the scenarios are presented, they cover a range of surface area's which were calculated in the above table.



Figure 4.4: Detailed overview of the surface area of the basin: 1) Scenario-a, 2) Scenario-b, 3) Scenario-c

In Table 4.2 the dimensions and quantities are given.

	Scenario-a	Scenario-b	Scenario-c
Length basin [m]	3,500	3,400	3,400
Width basin	100	500	1,000
Width of inlet with a water depth of -1 m below MSL [m]	100	100	100
Inlet width + 3 m above MSL [m]	200	200	200
Surface area below MSL [m ²]	503,313	1,653,119	3,232,332
Total volume below MSL [m ³]	366,178	1,632,722	3,006,530
Depth basin [m]	1	1	1

Table 4.2: Dimensions	three	scenarios
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4.3 Simulations

The scenarios include each 5 simulations where the wave height increases from the mean value to extreme wave heights. Higher and extreme wave heights are simulated to investigate the influence of longshore transport on the closure of the inlet system. Table 4.3 gives an overview of the used wave heights with the exceedance probabilities which are taken from the yearly wave climate (section 3.3.1). To reduce the scenarios and simulations, some parameters has been kept constant and average yearly values have been selected; these are the wave period (11 seconds), the wave direction (50 degrees from southeast) and the median diameter of the sand (300 µm). The tidal period (harmonic 12 hours) has also kept constant.

Hs	Pexceedence	Days
1.6	50%	182.5
2	18.40%	67.16
2.5	8.30%	30.295
3	3.20%	11.68
3.5	0.80%	2.92

Table 4.3: Selected wave height with exceedance probability and days of higher waves per year

The first simulation from each scenario is provided with average values as hydrodynamic forcing. This means the wave height and tidal amplitude are taken from the yearly wave climate. A wave height of 1.60 m. and a tidal range of 1.34 m. are selected.

In the second and third simulations of all the scenarios, a tidal range of 0.5 m. is selected, this correspond to the range between MLWN and MLWS from the tidal range data (Figure 3.7) this is done to simulate a situation of neap tide range in combination with higher wave heights than average. These inlets may be unstable due to low P/M ratios.

In the fourth and fifth simulations of the scenarios, a spring tide situation is modelled, a tidal range of 2 m. is applied which is just above the MHWS and MLWS range (Figure 3.7). In these simulations the wave height has increased to more extremer wave heights. The tidal prism will increase in these simulations too, but due to the higher wave heights still a low P/M ratio is expected.

By increasing the wave height in steps the longshore sediment transport also increases gradually. The tidal prism is for the most controlled by the tidal elevation and the basin surface area. By varying the tidal elevation and the wave height, different P/M ratios are modelled by which the stability of the

inlet can be examined. The different basin area sizes give another dimension in contrast with the P/M ratio, because a similar P/M ratio can result with different basin area sizes. In Table 4.4, Table 4.5 and Table 4.6 the three scenarios are given with their input values. The values for P and M_{tot} are extracted from the Delft3D numerical model results see (section 8.13, 8.14 and 8.15).

Scenario-a	H _s [m]	A _b [km ²]	A [m]	P [m ³]	M _{tot} [m ³ / year]	P/M
Sim-1a	1.6	0.50	0.67	550,000	226,000	2.43
Sim-2a	1.6	0.50	0.25	230,000	216,000	1.06
Sim-3a	2.5	0.50	0.25	230,000	1,372,000	0.17
Sim-4a	2.5	0.50	1	760,000	1,400,000	0.54
Sim-5a	3	0.50	1	760,000	2,900,000	0.26

Table 4.4: Setup scenario-a with: H_s = significant wave height, A_b = surface basin area, A = tidal amplitude, P = tidal prism, M_{tot} = total amount of longshore sediment transport

Scenario-b	H _s [m]	A _b [km ²]	A [m]	P [m ³]	M _{tot} [m ³ / year]	P/M	
Sim-1b	1.6	1.65	0.67	1,800,000	225,000	8	
Sim-2b	2	1.65	0.25	700,000	541,000	1.29	
Sim-3b	2.5	1.65	0.25	700,000	1,386,000	0.51	
Sim-4b	2.5	1.65	1	2,800,000	1,474,000	1.90	
Sim-5b	3	1.65	1	2,800,000	3,000,000	0.93	

Table 4.5: Setup scenario-b with: H_s = significant wave height, A_b = surface basin area, A = tidal amplitude, P = tidal prism, M_{tot} = total amount of longshore sediment transport

Scenario-c	H _s [m]	A _b [km ²]	A [m]	P [m ³]	M _{tot} [m ³ / year]	P/M
Sim-1c	1.6	3.23	0.67	3,400,000	260,000	13
Sim-2c	2.5	3.23	0.25	1,000,000	1,400,000	0.71
Sim-3c	3	3.23	0.25	1,000,000	2,800,000	0.36
Sim-4c	3	3.23	1	3,700,000	3,000,000	1.23
Sim-5c	3.5	3.23	1	4,300,000	5,270,000	0.82

Table 4.6: Setup scenario-c with: H_s = significant wave height, A_b = surface basin area, A = tidal amplitude, P = tidal prism, M_{tot} = total amount of longshore sediment transport

5. Simulation results of the scenarios

5.1 Model results Scenario-a: Small basin

The initial situation of the inlet in Scenario-a is presented in Figure 5.1



Figure 5.1: Initial situation of the bed level for scenario-a

5.1.1 Sim-1a: Small basin, average tide (1.34 m) and waves (1.6 m)

In sim-1a mean values as hydrodynamic forcing were chosen, the significant wave height is 1.6 m. and the tidal range is 1.34 m. During the simulation the current velocities in the inlet varied from 0.6 to 0.65 m/s, the same for ebb and flood tide. The instantaneous discharge through each tidal cycle can be seen in Figure 5.2 and vary from $35 \text{ m}^3/\text{s}$ at ebb tide to $55 \text{ m}^3/\text{s}$ at flood tide. From the cumulative discharge



Figure 5.2: Instantaneous discharge sim-1a

through the inlet (8.13) the tidal prism is observed, in this simulation the tidal prism is in the range of 550,000 m³. In combination with the total amount longshore sediment transport of 226,000 m³/year the P/M ratio is small with 2.43. Comparing the water levels in the ocean and bay, the basin has a maximum of 0.8 m. and the minimun lies between -0.41 and -0.28 m. at the end of the simulation. In the ocean the tidal range is constant with a value of 1.34 m. The ratio between

ocean and basin is at the end of the simulation 0.81. When looking at the morphological evolution of the inlet (Figure 5.4) a small shift of the tidal george can be observed in the northern direction, this is approximately 40 meters after 60 days. The direction of the tidal george bends to the north east, this deflection increases towards the end of the simulation. Comparing this deflection to the wave direction, a link can be



Figure 5.3: Water levels; basin and ocean sim-1a

found between the wave direction when the waves break. The waves break in the surf zone with an angle of 17 degrees (section 8.7) and the bending of the mouth is at the end of the simulations





Figure 5.4: Bed level evolution of the inlet channel of Sim-1a after 1) 60 days, 2) 120 days, 3) 180 days 4) 240 days, 5) 300 days and 6) Cross-sectional evolution of the inlet

approximately 20 degrees. Upstream of the inlet channel a distinct sand shoal is building which constricts the inlet, and causes the downdrift side to erode resulting in migration of the inlet channel northward. On picture 3) the same pattern can be seen only now the shoal has accumulated more sand, and has been grown more in volume causing the tidal george to migrate further to the north. At this stage the amount of inlet migration is 75 meter. After 240 days of morphological progress, the inlet george has reached its deepest point with a depth of 2.1 meters.

Looking at the longitudinal profile of the inlet (Figure 5.5), after 300 days a small flood delta originate from deposits of sediment carried into the inlet with the tidal currents. Only one main channel has formed in the inlet which shows a meandering character like a river. This inlet has a typicall bar-bypassing at the outer region of the inlet and outside of the basin there has been no formation of an ebb tidal delta, this is due to the high wave climate and relatively small tidal currents.



Figure 5.5: Longitudinal profile sim-1a

The inlet imports a nett amount of sediments of about 24,000 m³ (Figure 5.20), on a coast with a littoral drift of 226,000 m³/year updrift of the inlet. Downdrift of the inlet on a yearly basis the nett amount is 203,000 m³/year. All the sediments that have been accumulated by the inlet are stored for the most part in the shoal while less sediments are deposited at the flood delta.

5.1.2 Sim-2a: Small basin, neap tide range (0.5 m) with average waves (1.6 m)

The main difference in sim-2a from sim-1a is the tidal forcing which is neap tide with a range of 0.5 m. For the rest of the input the hydrodynamics are similar. During simulation the flow discharges vary from 14 m³/s at ebb tide to 20 m³/s at flood tide. Current velocities vary from 0.4 m/s at ebb tide and 0.6 m/s at flood tide. Water levels in the basin vary from 0.35 m. to 0.41 m. at high tide to -0.08 m. to -0.02 m. at low tide. The range of the water level in



Figure 5.6: Instantaneous discharge sim-2a

the ocean is 0.5 m. So the ratio between ocean and basin is in this simulation 0.86.

Cumulative discharges (8.13) show this simulation has a tidal prism of $230,000 \text{ m}^3$. In combination with the total yearly amount of longshore sediment transport of $216,000 \text{ m}^3$ the P/M ratio is 1.06.

The morphological behaviour of the inlet was analysed (Figure 5.8) and shows after 60 days small growth of sand shoal constricting the inlet. After 120 days the rate of growth is much more visible. The tidal gorge has moved a bit northwards (20 m.). After 180 days the inlet has been constricted by a sand shoal which protrudes 0.10 m. above MSL. Still an open connection with the ocean is maintained.

The period after 180 days of simulation shows that



Figure 5.7: Water levels; basin and ocean sim-2a

the tidal gorge has become somewhat shallower and more constricted. An open connection between basin and ocean is maintained till the end of simulation time. The depth in tidal gorge changes from its deepest point after 180 days with 1.5 m. to 1.2 m. at the end of simulation time.





Figure 5.8: Bed level evolution of the inlet channel of Sim-2a after 1) 60 days, 2) 120 days, 3) 180 days 4) 240 days, 5) 300 days and 6) Cross-sectional evolution of the inlet

Looking at the main difference between sim-1a and sim-2a in this simulation the formation of the sand shoal grows stable in time. There's a distinct absence of a flood delta due to low tidal currents carrying less amounts of sediment into the inlet. Inlet migration in this scenario is quite limited, which is also due to the low tidal currents, because larger currents have a larger eroding effect and hence a stronger migrative character.

5.1.3 Sim-3a: Small basin, neap tide range (0.5 m) with high waves (2.5)

Sim-3a is hydrodynamic the same as the previous simulation, only the wave height has increased and hence the amount of longshore sediment transport increased as well. The current velocities through the inlet vary from 0.25 m/s to 0.35 m/s before the inlet closes, and during the closure at ebb tide the current velocity is only 0.12 m/s. The discharge through the inlet has a range from 10 m³/s to 15 m³/s at ebb tide and 18 m³/s to



Figure 5.9: Instantaneous discharge

25 m³/s at flood tide. The tidal prism (8.13) is the same as sim-2a with 230,000 m³. But now the yearly total longshore sediment transport is 1,372,000 m³. This results in a very low P/M ratio of 0.17. Water levels in this simulation in the basin are varying from 0.4 m. to 0.55 m. above MSL at high water and 0 m. to 0.10 m. at low water before the mouth closes.

The offshore water level range is 0.25 m. above and below MSL. The tidal range ratio between the basin and ocean is roughly 0.96. A big difference between ocean and basin is the shift to higher high water levels and lower lows in the basin. This can be explained by the higher amount of discharge flowing in to the basin, and a lower amount flowing out during ebb tide, also regarded as a flood dominant system.



Figure 5.10: Water levels; basin and ocean sim-3a

The morphological evolution of the inlet was investigated (Figure 5.12), after 60 days the inlet

has been constricted by a spit headland both on the updrift and downdrift side of the inlet. There is not a distinct formation of a flood delta, because the flood currents are too weak. The tidal gorge curves toward the north as a result of the longshore drift going from south to north. After 180 days the inlet gets clogged up in front of the inlet by a formation of a distinct sand bar.



Figure 5.11: Longitudinal profile sim-3a

Continuous bar by-passing in front of the inlet, due to high wave action and low tidal currents result in the growth of a spit headland downdrift of the inlet. At a certain moment the bar updrift grows until the front of the inlet is blocked, probably due to low ebb currents which allow the sediments to settle in front of the inlet. As a final result, the inlet tries to maintain its connection with the ocean, but due to the retardation of the exchange of water with the ocean and the backbarrier and the associated lower tidal currents, the inlet finally closes due to the formation of a coastal barrier in front of the inlet. In Figure 5.11 the longitudinally evolution of the bed level in the inlet can be seen. After 60 days the inlet channel has a depth of approximately 2 m. below MSL. After 120 days the spit headland starts to grow more distinct and the depth in the inlet decreases. After 180 days the bar in front of the inlet starts to form a serious blockade for the exchange of water and stability of the inlet. And finally after 240 days the inlet is closed by a coastal barrier and the exchange of water is not significant anymore, only water overtopping at high tide intrudes the basin (Figure 5.12). Along the coast transport of sand is 1,372,000 m³/year and the amount of sand that accumulated in the inlet was 13,000 m³ (Figure 5.20).



Figure 5.12: Bed level evolution of the inlet channel of Sim-3a after 1) 60 days, 2) 120 days, 3) 180 days 4) 240 days, 5) 300 days and 6) Cross-sectional evolution of the inlet

5.1.4 Sim-4a: Small basin, spring tide range (2 m) with high waves (2.5)

Sim-4a is the scenario in which spring tide is modelled; a tidal range of 2 m. is imposed. The wave height is the same as in the previous simulation, so the main difference is a higher tidal prism with respect to sim-3a. The current velocities in the inlet are in the range of 0.4 m/s to 0.8 m/s at ebb tide, and 0.4 to 0.9 at flood tide.

The instantaneous discharge through the inlet at ebb tide is 45 m³/s and 90 m³/s at flood tide. The amplitude of the tidal signal in this simulation is 1 m. so the water level in the ocean has a range of 2 m. In the basin the range is in the beginning 1.65 and changes to 1.5 at the end of simulation. The ratio between ocean and basin changes from 0.83 to 0.75.



The tidal prism (8.13) is 760,000 m³, and the

Figure 5.13: Water levels; basin and ocean sim-4a

total longshore sediment transport is calculated to be 1,400,000 m³/year. The P/M ratio is a little bit higher than sim-3a but still very low with 0.54. The morphological evolution was investigated (Figure 5.14), after 60 days a spit headland has formed at the updrift side of the inlet mainly caused by longshore sediment transport. The tidal gorge has been slightly deflected to north east direction, because of a prevailing littoral drift going from south to north. After 120 days the tidal gorge has been migrated north with a distance of 75 m. and the channel has adopted a depth of 2.1 m. The formation of a flood delta has now been developing and two channels are formed.





Figure 5.14: Bed level evolution of the inlet channel of Sim-4a after 1) 60 days, 2) 120 days, 3) 180 days 4) 240 days, 5) 300 days and 6) Cross-sectional evolution of the inlet

These two channels are the main ebb dominated inner channels, which carry the ebb flows out of the basin. At the end of the tidal gorge the flood ramp starts, this is a steadily shallowing section of the flood delta, and divides in two channels which are in this case functioning as the inner ebb dominated channels. At the sides of the ebb channels, channel margin levees have developed.

These are submarine bars formed where the sediment transporting capacity of the ebb currents decrease along the edge of the laterally diverging ebb tidal jet.

In Figure 5.15 a distinct pattern can be found which shows the formation of the flood delta, and the main channels that are formed. At the end of the simulation the sand bar that has formed in front of the inlet protrudes with

almost 1 m. above MSL. The spit headland seems not to be the main reason of constricting the inlet, but the sand bar does. During simulation the outer side of the inlet is subject to continuous bar-bypassing. At a certain point the bar in front of the inlet forms a blockade whereby a reduced amount of discharge is observed, see (Figure 5.16) between 7 Jan and 9 Jan. The sand bar is washed away during the







Figure 5.16: Instantaneous discharge sim-4a

next tidal cycle, whereby a peak discharge of 50 m³/s is flushing out. The inlet channel has now migrated more northward, and maintains still an open connection with the ocean.

5.1.5 Sim-5a: Small basin, spring tide range (2 m) with extreme waves (3 m)

Sim-5a is similar to sim-4a except that a higher wave is selected, which result in more sediment transport along the coast. The range of the current velocities during ebb tide before the inlet closes varied between 0.7 m/s and 0.9 m/s. The currents during flood tide varied between 0.8 m/s and 1.05 m/s. The maximum instantaneous discharges varied during the simulation from 44 m³/s to 59 m³/s at ebb tide and at flood tide



this range is 80 m³/s to 100 m³/s before the inlet closes. At the last tidal exchange before closure of the inlet, the discharge through the inlet at flood tide is 53 m³/s and at ebb tide 14 m³/s.

The corresponding current at ebb tide while the inlet closes is 0.11 m/s. Water levels in the basin vary from 1.2 m. to 1. 38 m. above MSL at flood tide before closure results, and vary from -0.58 m. to -0.25 m. under MSL before closure.

The ratio between basin and ocean is 0.89 in the beginning and 0.82 before closes.



The tidal prism (8.13) is the same as the previous simulation because the same tidal

Figure 5.17: Water levels; basin and ocean sim-5a

range is selected and is 760,000 m³. But the total longshore sediment transport is now much higher with 2,900,000 m³/year resulting in a P/M ratio of 0.26. The morphology of the inlet channel was investigated (Figure 5.18), a spit headland formation is observed after 60 days at the updrift side of the inlet. The tidal gorge has migrated north with 30 m. and adopted a depth of 2.5 m. A formation of a flood delta is visible but not distinct, after 120 days a more clear view of a flood delta is





Figure 5.18: Bed level evolution of the inlet channel of Sim-5a after 1) 60 days, 2) 120 days, 3) 180 days 4) 240 days, 5) 300 days and 6) Cross-sectional evolution of the inlet

observed. The front side of the coast is subject to a typically bas-bypassing coast, which can be seen at the picture showing the evolution after 120 days. The sand bar moved along of the inlet while the inlet gorge maintained its connection with the ocean. A shoal on the downdrift side of the inlet has been formed and prograde into the southern direction. This goes on until the exchange of water between basin and ocean is completely blocked. And after 195 days the inlet definitive closes.

5.2 Analysis scenario-a

Five simulations were performed and different mechanisms regarding inlet dynamics were observed. The figure below represents the simulations that closed (red) and the other inlets that maintained an open connection during simulation time. In Figure 5.20 the bed level evolution, the amount of longshore transport and the sediment import into the basin is presented.



Figure 5.19: P/M ratio vs. tidal amplitude scenario-a

The first simulation "sim-1a" shows an inlet that maintains an open connection during the simulation period. Despite the P/M ratio being lower than 20. According to Bruun (1978) inlets with a P/M ratio lower than 20 are unstable and may close during a storm due to a relative low tidal prism. However, in this case the inlet maintains its connection during the 300 morphological days of simulation. At the end the inlet depth decreases and thereby also the cross-sectional area. This suggests that the inlet becomes less stable. A longer simulation period is required to clarify whether the inlet would remain open or would continue to tend towards ultimate closure.

Sim-2a gives information about what the tidal influence is on the simulation with respect to sim-1a. A smaller tide results in less sediment import to the flood delta, and also less migration due to the weaker inlet currents. The inlet depth decreases slightly approaching the end of the simulation period. The waves that produce the littoral drift are not dominant enough to close the inlet in this setup. Again a longer simulation time would give more information about the closure of the system.

Sim-3a illustrates that closure of the inlet results after imposing a higher wave height. A wave height of 2.5 m. was selected with an occurrence probability of 8.3%, which translates to 30 days per year statistically. The P/M ratio dropped significantly because the longshore transport increased a lot. The P/M ratio of 0.17 resulted in the closure of the system after 240 days.

Sim-4a shows that under the influence of a spring tide, the inlet imports more sediment (Figure 5.20) which builds a larger flood delta than the previous simulation. The same wave conditions are applied as in Sim-3a but a higher P/M ratio helps to maintain a longer open connection. It is clear that the inlet gets more constricted; the depth and the cross-sectional area are decreasing which suggest the inlet is unstable and might close by longshore transport should the simulation times be extended.

Sim-5a shows an inlet that has closed due to a bar-bypassing coast due to longshore sediment transport. An extreme wave height was selected, namely 3 m. with a probability of exceedance of 3.2%. Higher waves occur only 12 days per year. The P/M ratio is low at 0.26 and a decreasing cross-sectional area with a reducing tidal prism finally led to closure after 195 days.



Figure 5.20: Evolution of the inlet after 300 days (Scenario-a), with corresponding total longshore sediment transport [x1000 m³/year]

5.3 Model results scenario-b: Medium basin



The initial bed level at the inlet is presented in Figure 5.21

Figure 5.21: Initial situation of the bed level for scenario-b

5.3.1 Sim-1b: Medium basin, average tide (1.34 m) and waves (1.6 m)

In sim-1b the mean values for the wave height and the tidal elevation are selected (4.3) like sim-1a from Scenario-a. The only difference in this scenario is the basin surface area that is widened to 500 meters; a larger tidal prism can be expected. The inlet itself has the same dimensions.

During simulation time the depth averaged velocity is in the range of 0.65 m/s to 0.8 m/s and some peaks in the range from 0.9 m/s to 1.1 m/s during ebb tide and 0.85 m/s to 0.95 m/s during flood tide with peaks to 1.2 m/s. The maximum instantaneous discharge through the inlet has a range at

ebb tide of 115 m³/s to 150 m³/s. The flood flows have a range from 175 m³/s to 205 m³/s. The water levels in the basin are 0.74 m. at high tide, and at low tide the water level decreases from -0.45 m. to -0.33 m. Ocean water levels vary with a range of 1.34 m. The ratio between basin and ocean decreases from 0.88 to 0.80. As stated earlier a larger tidal prism can be expected and when looking at the tidal prism for this



Figure 5.22: Water levels; basin and ocean sim1-b

simulation (8.14), an amount of 1,800,000 m³ is flowing into and out of the basin during half a tidal cycle. With the same wave forcing as in sim-1a the amount of longshore transport is 225,000 m³/year, resulting in a P/M ratio of 8. The morphological evolution of the bed level was investigated (Figure 5.24). After 60 days the tidal gorge has adopted a depth of 4.1 m. and has migrated 30 m. from the centre of the inlet.

At this stage a shoal has formed updrift of the inlet, tidal currents in the inlet channel and the involved capacity of removing the sediments erode the inlet channel and force the shoal to grow in the direction of the basin. A flood delta develops at the end of the tidal gorge. After 120 days, a more distinct flood delta has been developed with two main inner ebb-dominated channels, one on the top side eroding the bank



Figure 5.23: Instantaneous discharge sim1-b

and one below the flood delta. After 60 days a small ebb delta is visible which divides the ebb tidal flows into two channels. After further progress the ebb delta is washed away, apparently by strong ebb tidal currents and high waves that break in the surf zone generating longshore currents.



Figure 5.24: Bed level evolution of the inlet channel of Sim-1b after 1) 60 days, 2) 120 days, 3) 180 days 4) 240 days, 5) 300 days and 6) Cross-sectional evolution of the inlet

After 180 days the tidal gorge has a depth of approximately 4 m. and the inlet location has been stabilised at the same location. Very little migration takes place during the rest of the simulation. In later stages of the simulations (Figure 5.24 4 & 5) the flood delta is well developed and a the presence of two main inner ebb dominated channels can be seen. Spill over channels can be found between the ebb channels. The sand spit formed updrift of the inlet has now been migrated to the upper side of the basin forcing the tidal gorge to bend in the same direction. After 300 days of morphological simulation the inlet still maintains an open connection between bay and ocean and doesn't seem to become unstable in the short term with this hydrodynamic forcing.

5.3.2 Sim-2b: Medium basin, neap tide range (0.5 m) with high waves (2 m)

In sim-2b a wave height of 2 m. is selected and a small tidal elevation representing neap tide is chosen. Compared to sim-2a the wave height is increased; this is done to compensate the larger surface area causing a larger tidal prism. The depth averaged velocity during the simulation is in the range of 0.65 m/s to 0.8 m/s during ebb tide and 0.55 m/s to 0.7 m/s during flood tide.

The maximum instantaneous discharge through the inlet varies at ebb tide with 40 m³/s to 50 m³/s. The flood flows have a range from 50 m³/s to 60 m³/s. The water levels in the basin are 0.4 m. at high tide, and at low tide the water level increases from -0.03 m. to 0.03 m. above MSL. Ocean water levels are constant with a range of 0.5 m. The ratio between basin and ocean decreases from 0.9 to





0.8. In this simulation the tidal prism (8.14) is 700,000 m³ and the longshore sediment transport is 541,000 m³/year. A small P/M ratio is found with 1.29.

The morphological behaviour of the bed level was evaluated (Figure 5.27). After 60 days the

evolution of the inlet is characterised by a small growth of a shoal updrift of the inlet channel. The inlet channel bends to the north east, and at the inner side of the coastal barrier at the downdrift side of the channel also a shoal has formed. As a result of the longshore sediment transport and small tidal currents a small flood delta has been formed which is clearer after 180 days.



Figure 5.26: Instantaneous discharge sim-2b

On both sides of the flood delta two channels have been formed which are the main inner ebb channels which carry the ebb currents out of the basin. Eventually the updrift shoal gets larger and more distinct constricting the inlet. An open connection is still maintained, but at the end of simulation time the channel has migrated north with approximately 100 m. No ebb delta has been



Figure 5.27: Bed level evolution of the inlet channel of Sim-2b after 1) 60 days, 2) 120 days, 3) 180 days 4) 240 days, 5) 300 days and 6) Cross-sectional evolution of the inlet

formed, this is due to the low tidal currents which doesn't transport much of the sediments out to the ocean, and if they do the longshore currents pick up the sediments transporting it to the north along the coast. In this simulation the bar-bypassing mechanism is not distinct active, therefore the sand shoal on the downdrift side isn't accumulating enough sand to grow in the updrift direction constricting the inlet. The inlet gorge is now able to migrate steady without intervention of sand accumulated at the downdrift barrier. After 300 days the inlet still maintained an open connection but from the tidal the tidal prism evolution (Figure 8.5) and discharge through the inlet (Figure 5.26) and also the water levels in the basin, this simulation seems to keep an open connection for some time after the end of simulation.

5.3.3 Sim-3b: Medium basin, neap tide range (0.5 m) with high waves (2.5 m)

In sim-3b the wave height is increased to 2.5 m. representing high wave climate and the same tide conditions are selected as in sim-2b representing neap tide. The depth averaged velocity during the simulation is in the range of 0.65 m/s to 0.8 m/s during ebb tide and 0.55 m/s to 0.65 m/s during flood tide. The maximum instantaneous discharge through

the inlet varies at ebb tide with 50 m³/s in the beginning to 38 m³/s before closing. The flood flows vary from 62 m³/s to 44 m³/s. The water levels in the basin are varying from 0.46 m. to 0.5 m. at high tide, and at low tide the water level increases from MSL to 0.10 m. above MSL. Ocean water levels are constant with a range of 0.5 m. The ratio between basin and ocean decreases from 0.92 to 0.8. The P/M ratio in this simulation is 0.51, the corresponding tidal prism





simulation is 0.51, the corresponding tidal prism (8.14) is 700,000 m³ and the longshore sediment transport is $1,386,000 \text{ m}^3/\text{year}$. The morphological behaviour of the bed level was evaluated

(Figure 5.30). After 60 days the evolution of the inlet is characterised by a small growth of a shoal updrift and downdrift of the inlet channel. The tidal gorge bends to the north east as a result of the littoral drift along the coast. As a result of the longshore sediment transport and small tidal currents the sand shoal on the downdrift side of the inlet grows more in the direction of the basin.



Figure 5.29: Instantaneous discharge sim-3b





Figure 5.30: Bed level evolution of the inlet channel of Sim-3b after 1) 60 days, 2) 120 days, 3) 180 days 4) 240 days, 5) 300 days and 6) Cross-sectional evolution of the inlet

Along the coast continuously a bar-bypassing mechanism can be observed, the interaction between tidal currents and littoral drift promotes this process, because tidal currents are relatively low compared to the currents along the coast. In comparison to the previous simulation the longshore transport is much higher and the P/M ratio is much lower. The closure of the system results after 195 days and occurs during the start of the ebb tidal cycle, the transition between flood tide and ebb tide. At the start of the tidal cycle the currents are weak and therefore the inlet is more exposed to longshore processes that drive sand accumulation in front of the inlet.

5.3.4 Sim-4b: Medium basin, spring tide range (2 m) with high waves (2.5 m)

In sim-4b the same settings were used as in sim-3b except the tidal elevation that changed from 0.25 m. to 1 m. The depth averaged velocity during the simulation is around 1 m/s during ebb tide and 0.8 m/s during flood tide. The maximum instantaneous discharge through the inlet varies at ebb tide

around 160 m³/s and flood flows vary from 320 m³/s to 240 m³/s. The water levels in the basin are varying around 1.14 m. above MSL at high tide, and at low tide the water level increases from -0.6 m. to -0.3 m. below MSL. Ocean water levels are constant with a range of 2 m. The ratio between basin and ocean decreases from 0.87 to 0.72. The P/M ratio in this simulation is 1.90 with its corresponding tidal prism of 2,800,000





 m^3 and the total amount of sand from littoral drift is 1,474,000 m^3 /year. The morphological behaviour of the bed level was evaluated (Figure 5.33). After 60 days the tidal gorge has a depth of 4

m. and the inlet migrated north with 50 m. A clear presence of a flood delta has been developed, with most of its features such as the main inner ebb channels, ebb spits, flood ramp, spillover channels and the ebb shield. In the first 180 days the tidal gorge migrates 150 meters to the north and on the updrift side a shoal has been formed. Deposits from longshore sediment transport is the main reason for the formation of



Figure 5.32: Instantaneous discharge sim-4b

this shoal. After 240 days this shoal gets larger and grows at this stage more in the downdrift





Figure 5.33: Bed level evolution of the inlet channel of Sim-4b after 1) 60 days, 2) 120 days, 3) 180 days 4) 240 days, 5) 300 days and 6) Cross-sectional evolution of the inlet

direction which is more clear at the end of the simulation. The total amount of migration is 280 meters. From Figure 5.32 it can be seen that the amount of inflowing water gets less and also in the chapter 8.14 the tidal prism get reduced.

5.3.5 Sim-5b: Medium basin, spring tide range (2 m) with extreme waves (3 m)

In sim-5b, the wave height has been increased to 3 m. compared to the previous simulation. The other input parameters are similar. The depth averaged velocity during the simulation is around 1 m/s during ebb tide and 0.8 m/s during flood tide.

The maximum instantaneous discharge through the inlet varies at ebb tide around 160 m³/s and flood flows vary from 320 m³/s to 240 m³/s. The water levels in the basin are varying around 1.14 m. above MSL at high tide, and at low tide the water level increases from -0.6 m. to -0.3 m. below MSL. Ocean water levels are constant with a range of 2 m. The ratio between basin and ocean decreases from 0.87 to 0.72. The P/M



Figure 5.34: Water levels; basin and ocean sim-5b

ratio is now 0.93 due to the same tidal prism as in sim-4b but a higher longshore amount of sediment of 3,000,000 m³. The morphological behaviour of the bed level was evaluated (Figure 5.45).

The same pattern is observed as in the previous simulation, but due to the higher amount of longshore sediment transport some differences occur. The same features of a flood delta are present but a different morphology is observed due to more sediment transported into the basin. A big difference is the tidal gorge that now shifts more into the basin and doesn't migrate much north in the first 180 days. Then at a certain point



Figure 5.35: Instantaneous discharge sim-5b

the vast amount of longshore sediment transport provides the updrift shoal enough sand to accumulate more and eventually leads to growth in the downdrift direction. A large spit across the





Figure 5.36: Bed level evolution of the inlet channel of Sim-5b after 1) 60 days, 2) 120 days, 3) 180 days 4) 240 days, 5) 300 days and 6) Cross-sectional evolution of the inlet

initial entrance has now been formed and the tidal gorge has now migrated with 300 m. At the end of the simulation the depth of the tidal gorge decreases and the cross-sectional area gets smaller.

5.4 Analysis scenario-b

Scenario-b was analysed in the previous section and in the figure below the simulations are placed with respect to the P/M ratio and the tidal amplitude. The red marker indicates closure of the inlet, the others stayed open during the simulation time. In Figure 5.38 the bed level evolution, the longshore amount of sediment transport and sediment import into the basin is presented.



Figure 5.37: P/M ratio vs tidal amplitude scenario-b

What can be observed from Sim-1b is that a larger basin results in larger tidal prism, and hence more sediments flowing into the system under the same littoral drift conditions. The tidal gorge has a more stable location and the depth remains more or less the same at 4 meters. The large sandbar on the updrift shoal is able to grow because of the discharge rate that is not strong enough to break through this bar. The cross-sectional area increases at the end of simulation time, and the tidal prism stays constant which suggest the inlet might stay open for a longer period.

From Sim-2b the main observation and difference with sim-1b is the influence of a higher wave height and a low tidal regime. The tidal currents are relative weak and therefore don't transport much sediments into the inlet. Little migration takes place, and at the end of the simulation the depth and cross-sectional area decreases, this is in line with the tidal prism that decreases. Longer simulation time would give a better understanding on the eventual closure of this inlet as this is expected.

In Sim-3b the wave height increased to 2.5 m and its probability of exceedance is of 8.3% which can be seen as a medium to extreme wave height. Higher waves occur 30 days per year. In combination with the selected neap tide range, the closure results after 195 days. The closure mechanism is mainly triggered by bar-bypassing mechanism and relative weak inlet currents and occurs during ebb tide.

In Sim-4b the effect of the tide can be evaluated. A spring tide range is in this setup responsible for more import of sediments and the formation of a flood delta. The combination of littoral drift and tidal currents are responsible for the high amount of inlet migration. This inlet is in terms of migration is not stable, but due to relative high tidal currents an open inlet is maintained during simulation. Another sign of the instability of the inlet is the decreasing cross-sectional area of the inlet with a lowering tidal prism. This suggests that the inlet might close over a longer time period.

In Sim-5b a higher wave height was imposed; 3.5 m. with an exceedance probability of 3.2% which can be seen as an extreme wave height. Higher waves statistically occur only 12 days per year. The combination of spring tide and the wave forcing seem to result in more or less the same morphological development as Sim-4b. At the end of the simulation the depth decreases and thereby also the cross-sectional area. It indicates the inlet is getting more unstable and the decreasing tidal prism clarifies this.



Figure 5.38: Evolution of the inlet after 300 days (Scenario-b), with corresponding total longshore sediment transport [x1000 m³/year]

5.5 Model results scenario-c: Large basin

 $f_{1,2}^{4,10} \xrightarrow{bed level in water level points (m)}{f_{1,2}^{4,10}} \xrightarrow{bed level in water level points (m)}{f_{1,2}^{4,10}} \xrightarrow{f_{1,2}^{4,10}} \xrightarrow{f_{1,2}^{$

The initial bed level at the inlet is presented in Figure 5.39

5.5.1 Sim-1c: Large basin, average tide (1.34 m) and waves (1.6 m)

In sim-1c the mean values for the wave height and the tidal elevation are selected (4.3) like sim-1a and sim-1b. This scenario is modelled with the largest basin width of 1000 m. raising the total surface area to 3.23 km². During simulation time the depth averaged velocity is in the range from 1.25 m/s

to 1 m/s and during ebb tide and 1.3 m/s to 0.7 m/s at flood tide. The maximum instantaneous discharge through the inlet varies at ebb tide around 210 m³/s and flood flows are in the range of 350 m³/s. The water levels in the basin go from 0.7 m. above MSL to -0.3 below MSL. Ocean water levels are constant with a range of 1.34 m. The ratio between basin and ocean 0.75 Looking at the P/M ratio, since the basin surface





area has increased the tidal prism has also gone up to 3,400,000 m³, the longshore sediment

transport is the same as in the first simulation from the previous scenarios with 260,000 m^3 /year. This results in a P/M ratio of 13. The morphological behaviour of the bed level was evaluated (Figure 5.42), and the development of a flood delta can be seen, with all the usual features common to these elements.



Figure 5.41: Instantaneous discharge sim-1c





Figure 5.42: Bed level evolution of the inlet channel of Sim-1c after 1) 60 days, 2) 120 days, 3) 180 days 4) 240 days, 5) 300 days and 6) Cross-sectional evolution of the inlet

5.5.2 Sim-2c: Large basin, neap tide range (0.5 m) with high waves (2.5 m)

In sim-2c a wave height of 2.5 m. is selected and a small tidal elevation representing neap tide is chosen. During simulation time the depth averaged velocity is in the range from 0.8 m/s

to 0.95 m/s during ebb tide and 0.75 m/s. at flood tide. The maximum instantaneous discharge through the inlet varies at ebb tide around 60-80 m³/s and flood flows are in the range of 75-90 m³/s. The water levels in the basin go from 0.4 m. to 0.45 m. above MSL for high water and from 0.05 m. to 0.15 m. above MSL for low waters. Ocean water levels are constant with a range of 0.5 m. The ratio



Figure 5.43: Water levels; basin and ocean sim-2c

between basin and ocean is in the beginning of the simulation 0.8 and goes eventually to 0.6 before

closing. The P/M ratio is 0.72 with the corresponding tidal prism of $1,000,000 \text{ m}^3$ and the longshore sediment transport of $1,400,000 \text{ m}^3$ /year.

The morphological behaviour of the bed level was evaluated (Figure 5.45), after 60 days a clear direction of the inlet gorge to the north east can be found. Continuous bar-bypassing along the inlet controls the inlet stability. After 120 days,



Figure 5.44: Instantaneous discharge sim-2c

a small flood delta develops and on the updrift side of the inlet a sand shoal has formed and grows further downdrift constricting more and more the inlet. The migration of the inlet is quite small and as can be seen on picture 6). After 180 days a shift in the direction of the inlet gorge can be seen to





Figure 5.45: Bed level evolution of the inlet channel of Sim-2c after 1) 60 days, 2) 120 days, 3) 180 days 4) 240 days, 5) 300 days and 6) Cross-sectional evolution of the inlet

the south east direction, this is due to bar-bypassing in front of the inlet, whereby the sand on the downdrift side of the inlet is deposited and grows in the updrift direction. Eventually a spit across the inlet channel will form and the relatively low tidal currents can't maintain an open connection. After 210 days the inlet has been closed.

5.5.3 Sim-3c: Large basin, neap tide range (0.5 m) with extreme waves (3 m)

In sim-3c the wave height has increased to 3 m. and thereby increasing the longshore amount of sediment transport. A small tidal elevation representing neap tide is chosen. During simulation time the depth averaged velocity during ebb flows are in the range from 0.8 m/s to 0.9 m/s, for the flood flows the currents are in the range of 0.7 m/s to 0.8 m/s.

The maximum instantaneous discharge through the inlet varies at ebb tide around 95 m³/s to 65 m³/s and flood flows are in the range of 75 m³/s to 90 m³/s. The water levels in the basin go from 0.4 m. to 0.45 m. above MSL and peaks with 0.51 m. before the inlet closes. The low waters go from 0.04 m. +MSL to 0.12 m. +MSL. Ocean water levels are constant with a range of 0.5 m. This gives this system a ratio between basin and



Figure 5.46: Water levels; basin and ocean sim-3c

ocean of 0.78 to 0.6 just before closing. Looking at the P/M ratio of 0.36 it's a very small ratio and closure of the system will result. The corresponding tidal prism is $1,000,000 \text{ m}^3$ and the sediment

transport along the coast is 2,800,000 m³/year. In Figure 5.48 the morphological evolution of the inlet is presented. During the first 60 days the tidal gorge adopts a depth of 3 m, and takes an oblique direction to the north east. On the coast a typical bar-bypassing mechanism takes place in front of the inlet. Tidal currents are weak compared to the littoral drift on the coast. Eventually after 120 days a small flood delta



Figure 5.47: Instantaneous discharge sim-3c





Figure 5.48: Bed level evolution of the inlet channel of Sim-3c after 1) 60 days, 2) 120 days, 3) 180 days 4) 240 days, 5) 300 days and 6) Cross-sectional evolution of the inlet

has developed, whit two ebb channels. The sand bar in front of the inlet keeps on passing the inlet and eventually the inlet currents cannot maintain an open connection because the eroding capacity becomes too weak. A spit across the inlet finally result in the closure of the system after 135 days.

5.5.4 Sim-4c: Large basin, spring tide range (2 m) with extreme waves (3 m)

In sim-4c, the same wave height as in sim-3c is selected; the only difference in this simulation is the tide which has now an amplitude of 1 m. The depth averaged velocity during the simulation is around 1.25 m/s during ebb tide and 1.5 m/s during flood tide. The maximum instantaneous

discharge through the inlet varies at ebb tide around 300 m³/s and 200 m³/s and flood flows vary from 550 m³/s to 325 m³/s. The water levels in the basin are varying around 1.14 m. above MSL at high tide, and at low tide the water level increases from -0.6 m. to -0.3 m. below MSL. Ocean water levels are constant with a range of 2 m. The ratio between basin and ocean decreases from 0.87 to 0.72. The P/M ratio in





this simulation is 1.23 with the corresponding longshore transport of $3,000,000 \text{ m}^3/\text{year}$ and a tidal prism of $3,700,000 \text{ m}^3$. From the discharges and tidal prism observations it can be seen that the tidal

prism decreases which means the stability of the system also decreases. The morphological behaviour of the bed level was evaluated (Figure 5.51). A well visible flood delta develops with all its features; a flood ramp, flood channels, ebb shield, ebb spits and spillover lobes (see also 2.2). So from the previous simulation and this we see a clear difference that caused the increased tidal elevation. By the increase of tidal amplitude the



Figure 5.50: Instantaneous discharge sim-4c



flood currents increases as well, and this is the main driving mechanism that builds this flood deltas.


Figure 5.51: Bed level evolution of the inlet channel of Sim-4c after 1) 60 days, 2) 120 days, 3) 180 days 4) 240 days, 5) 300 days and 6) Cross-sectional evolution of the inlet

From figure 6) the amount of inlet migration and the growth of the updrift sand shoal which protrudes above MSL can be seen. The total amount of inlet migration after 300 days is 500 m. That is approximately 50 m. per month. Looking at the stability, the location of the inlet is not stable since continuous migration takes place. A combination of factors is the result of this, first of all both the tidal prism and longshore sediment transports are high. The growth of the updrift sand shoal is stimulated by the main inner ebb channel on the updrift side that gets blocked by this bar. Now the ebb channel is more directed to the north and therefore sand deposits on the sand shoal is easier. We see that the downdrift coastal barrier is fully eroded away by strong concentrating ebb currents. At the end of the simulation we see the tidal prism becomes smaller and smaller but the inlet keeps an open connection with the ocean.

5.5.5 Sim-5c: Large basin, spring tide range (2 m) with extreme waves (3.5)

In sim-5c the wave height increased to 3.5 m. and the same tidal elevation as sim-4c has been simulated. The only difference is a higher longshore sediment rate and thus a lower P/M ratio.

The depth averaged velocity during the simulation is around 1.25 m/s during ebb tide and 1.5 m/s during flood tide. The maximum instantaneous discharge through the inlet varies at ebb tide around 300 m³/s and 200 m³/s and flood flows vary from 550 m³/s to 325 m³/s. The water levels in the basin are varying around 1.14 m. above MSL at high tide, and at low tide the water level increases from -0.6 m. to -0.3 m.



Figure 5.52: Water levels; basin and ocean sim-4c

below MSL. Ocean water levels are constant with a range of 2 m. The ratio between basin and ocean decreases from 0.87 to 0.72. The P/M ratio is 0.82 with a longshore transport rate of 5,270,000

m³/year and the tidal prism being 4,300,000 m³. Looking at the morphological development (Figure 5.54), in the first 60 days the inlet gorge has a depth of 6 m. a distinct development of a flood delta is viewed and a small ebb delta with two marginal flood channels can be seen. One that flows to the north and the other going south. After 120 days the ebb delta is not present anymore, high waves and longshore sediment





transport wash away these sand bars. Quite the same pattern is observed as in the previous simulation but in sim-5c more migration of the inlet takes place. In figure 6) the location of the inlet





Figure 5.54: Bed level evolution of the inlet channel of Sim-5c after 1) 60 days, 2) 120 days, 3) 180 days 4) 240 days, 5) 300 days and 6) Cross-sectional evolution of the inlet

can be seen and the amount of migration. After 300 days the rate of migration is in the order of

5.6 Analysis scenario-c

Scenario-c was analysed in the above section and the figure below shows the simulations with respect to the P/M ratio and the tidal amplitude. The red marker indicates closure of the inlet, the others stayed open. The second figure in this section (Figure 5.56) shows the bed level evolution of all the simulations with corresponding longshore sediment transport and sediment import into the basin.



Figure 5.55: P/M ratio vs. tidal amplitude scenario-c

From Sim-1c the influence of a larger basin can be seen in comparison with Sim-1a and Sim-1b. A larger P/M ratio results due to the larger surface area and hence a larger tidal prism. The import of sediments to the basin is almost the same as in Sim-1b. Due to higher discharge rates through the inlet a larger flood delta develops. Also no sandbar accumulates on the updrift side of the inlet; this is mainly because of the eroding effects of the larger discharge rates that carry the sediments further into the basin.

In Sim-2c the influence of a neap tide range with a medium to extreme wave height of 2.5 m. was selected. The wave height has an occurrence probability of 8.3% of the time which is about 30 days in a year. The tidal gorge does not migrate but a deflection of about 20 degrees can be related to the wave breaking angle which is 20 degrees. After 210 days the inlet closes off by a clear bar-bypassing mechanism and a large sand shoal that blocks the inlet channel.

Sim-3c has the same tide but an extreme wave height is imposed with 3 m. of which only 3.2% of the waves exceed this height. In this setup the mechanism as in Sim-2c is observed only the closure of the inlet now results in 144 days.

Sim-4c has been setup again with spring tide and an extreme wave height of 3 m. Migration takes place with a constant rate which is due to large inlet discharge flows and also large littoral drift. The cross-sectional area is getting smaller which is also in line with the decreasing tidal prism.

In Sim-5c the influence of a more extreme wave height is elaborated. This simulation is setup with a wave height of 3.5 m. and has an exceedance probability of 0.8%, higher waves occur only 3 days per year. The larger tidal basin results in a higher tidal prism and more sediment import.



Figure 5.56: Evolution of the inlet after 300 days (Scenario-c), with corresponding total longshore sediment transport [x1000 m³/year]

5.7 Sensitivity simulations with different input parameters

To get a better understanding on the accuracy of the input parameters in Delft3D and their influence on the closure of the system and the amount of longshore sediment transport, some simulations are selected to be simulated again but with different input parameters. The three parameters that have been varied in the new simulations are; H_s = significant wave height, D_{50} = the median diameter of sediment, and both SusC & BedC; the current-related suspended sediment transport factor and current-related bed-load transport factor.

The simulations selected are given in the next table with their original values. The arrow indicates the change from the original value to the used parameter.

Scenario-a	H _s [m]	SusC & BedC	\mathbf{D}_{50}	Α	P [m ³]	M _{tot} [m ³ / year]	P/M
		factor	[µm]	[m]	x 1000	x 1000	
Sim-1aa	1.6 → 2	5 → 2	300→200	0.67	450	512	0.88
Sim-3aa	2.5	5	300→200	0.25	200	3,068	0.07
Sim-5aa	3	5 → 1	300→200	1	750	1,422	0.5
Scenario-b	H _s [m]	SusC & BedC	\mathbf{D}_{50}	Α	P [m ³]	M _{tot}	P/M
		factor	[µm]	[m]	x 1000	[m ³ / year]	
Sim-1bb	$1.6 \rightarrow 2$	5 → 2	300→200	0.67	1,800	540	3.33
Sim-3bb	2.5 → 3	5 → 2	300→200	0.25	600	2,900	0.2
Sim-5bb	3 → 3.5	5 → 1	300→200	1	2,850	2,900	0.98
Scenario-c	H _s [m]	SusC & BedC	\mathbf{D}_{50}	Α	P [m ³]	M _{tot}	P/M
		factor	[µm]	[m]	x 1000	[m ³ / year]	
Sim-1cc	1.6 → 3.5	5 → 1.5	300→200	0.67	3,250	4,300	0.75
Sim-2cc	2.5	5 → 2	300→200	0.25	1,000	1,300	0.76
Sim-3cc	3	$5 \rightarrow 2$	300→200	0.25	800	2,900	0.27

Table 5.1: Input values with the changed parameters

5.7.1 Model results sensitivity simulations

The results of the sensitivity simulations are presented and elaborated in this section; the tidal prism results are compared with the original simulations from the scenarios. From these figures the timescale of closure can be extracted. A sudden drop in the exchange of water between basin and ocean can be seen as the closure of the inlet system.

Looking at the first three simulations (Table 5.2), the tidal prisms are displayed; the first simulation Sim-1aa shows almost no differences except a small difference in tidal prism. The blue line is just under the red line which indicates this effect. Looking at the morphological development at the end of simulation time (Figure 5.58: and Figure 5.20) similar mouth behaviour is observed compared to Sim-1a. The P/M ratio dropped a little to 0.88 from 2.43 due to a higher amount of longshore sediment transport. But after 300 days an open connection is still maintained.

Sim-3aa shows that in the beginning the tidal prism is exact the same as in sim-3a, but due to the dropped P/M ratio from 0.17 to 0.07 the inlet closes in this simulation after 113 days, 127 days faster than the original simulation. In this situation only the D_{50} decreased to 200 microns. The influence on longshore transport was an increase with a factor two.

Sim-5aa shows after progress in simulation time the same tidal prism but in the first 60 days a slightly higher amount of flood flows are observed. In this run the P/M ratio is higher due to a lower longshore amount of sediment transport. SusC & BedC reduced from 5 to 1, so this should give a factor 5 in reduction to the longshore transport, but due to the lower D_{50} , the total reduction in longshore transport is a factor of 2.5. A lower D_{50} contributes to more longshore sediment transport, this is in line with the Kamphuis formula (2.6.9) since it has a negative power of -0.25. The inlet maintains an open connection during the simulation period, although a decrease in the tidal prism points towards closure.





The tidal prism results of the sensitivity simulations compared with scenario-b are presented in Table 5.3.

Sim-1bb has a slightly smaller tidal prism than Sim-1b but not marginally. The P/M ratio dropped from 8 to 3.33 because the longshore transport increased to 540,000 m³/year. Despite the reduction of SusC & BedC to 2, the increased wave height from 1.6 to 2 and a reduction of the D₅₀ to 200 μ m were the two responsible factors for a higher longshore drift.

Sim-3bb has a smaller tidal prism compared to Sim-3b, a difference of 100,000 m³ is observed. It has also a lower P/M ratio; the ratio dropped from 0.51 to 0.21. The longshore transport almost doubled to 2,900,000 m³/year. The closure time shortens with this setup from 195 to 174 days. The changed parameters were the wave height, it changed from 2.5 to 3 m. and a decrease of SusC and BedC from 5 to 2. The D⁵⁰ decreased to 200 microns.

Sim-5bb has a slightly larger tidal prism but shows the same trend during the simulation. The P/M ratio is almost the same with 0.98 from the original ratio of 0.93. While the SusC & BedC value reduced from 5 to 1 and the wave height raised from 3 to 3.5 the longshore transport was still the same with 2,900,00 m³/year. This means that the wave height of 3.5 m in combination with a D₅₀ of 200 microns produce the same amount longshore transport as the original simulation. Although a lower D₅₀ increases sediment transport, the wave height contributes the most since this value has a nonlinear relation in the known sediment transport formulas. Looking at the morphology at the end of simulation time (Figure 5.58) and comparing them with the results of Sim-5b (Figure 5.38) the main difference observed is the migration of the inlet of Sim-5b while no migration takes place in Sim-5b whereas Sim-5b migrates 300 m. north.





The tidal prism results of the sensitivity simulations compared with scenario-c are presented in Table 5.4.

Sim-1cc has a lower tidal prism and has a slightly negative slope which indicates that a little bit more water flows into the basin than out each tidal cycle. The P/M ratio dropped from 13.08 to 0.75 and therefor the stability decreased. The longshore transport in this setup has increased from 260,000 m³/year to 4,300,000 m³/year. The wave height increased from 1.6 to 3.5 and the SusC & BedC factor decreased to 1.5. From the tidal prism signal of this simulation (blue line) the closure of the inlet results after 210 days. For the longshore sediment transport it can be concluded that the normalized Delft3D amount, if we divide the observed longshore transport with 1.5, the longshore transport is, 2,866,666 m³/year. So both the wave height and the grain size increased the longshore sediment transport.

Sim-2cc follows the same trend concerning the tidal prism but it has a slightly lower tidal prism. The P/M ratio almost remained the same but increased a little from 0.71 to 0.77. The longshore sediment transport decreased with 100,000 m³/year. The observed differences were due to only two different parameters; D_{50} went to 200 microns and the SusC & BedC factor went down from 5 to 2. The closure time in this simulation is 278 days while sim-2c closed after 210 days.

Sim-3cc follows the same trend as sim-3c but as simulation time increases the tidal prism lowers. And due to a higher longshore sediment transport of 100,000 m³/year and a slightly lower tidal prism, the P/M ratio dropped to 0.28 from 0.36. The inlet remains longer open for one tidal cycle, in terms of morphology this is approximately 15 days.





5.8 Analysis sensitivity simulations

From the above simulations the closure time scale and the parametrical influence on the tidal system was evaluated. In the figure below (Figure 5.57) an overview is given of the simulations that closed. The arrows show how the sensitivity simulations relate to the scenario simulations in terms of closure time. Two inlets; Sim-3aa and Sim-3bb, have a lower P/M ratio and have a shorter closure time. Sim-2cc has a higher P/M ratio and show an extended closure time. Sim-3cc has a lower P/M ratio, but show a longer closure time.



Figure 5.57: Closed inlets; scenario and sensitivity simulations



Figure 5.58: Evolution of the inlet after 300 days (Sensitivity-simulations), with corresponding total longshore sediment transport [x1000 m³/year]

5.9 Mfolozi River discharge influence

From all the initial simulations of the three main scenarios, five of the simulations where closed before end of simulation time. These are; sim-3a, sim-5a, sim-3b, sim-2c and sim-3c. The corresponding P/M ratios are respectively 0.17, 0.26, 0.51, 0.71 and 0.36.

Two extra simulations were performed with a river discharge added in the model. The discharge flows from the basin into the ocean which represents the Mfolozi River in a combined state with the St Lucia estuary. The influence of the discharge will be investigated and the results are presented in the next two sections.

The picture below was taken at the end of my visit in South Africa. At that time the Mfolozi flushed out to the Indian Ocean.



Figure 5.59: Mfolozi River mouth, separate from the St Lucia inlet, May 2012

5.9.1 Sim-5d: Small basin, neap tide range (0.5 m) extreme waves (3.5 m) and a river discharge (5 m³/s)

This simulation has to be compared with simulation "sim-5a" (5.1.5) because it has the same hydrodynamic forcing from the ocean side, the only difference is the influence of a river discharge which represents the Mfolozi river. The discharge is set to 5 m³/s and the flows out of the basin. It represents a yearly runoff of 158×10^6 m³, which is 16% of the mean annual runoff (see chapter 3.5). Comparing the water levels in the basin, not much differences are



Figure 5.60: Water levels; basin and ocean sim-5d

found except little bit higher basin water levels. Looking at the ratio of the basin and ocean; it starts with a ratio of 0.85 and when the inlet closes the ratio is 0.93. From the discharges through the inlet

some differences are observed. The flood discharges are now less strong varying from 75 m³/s to 85 m³/s while the ebb flows are stronger and in the range of 55 m³/s to 65 m³/s and a peak before closure of 85 m³/s. Because of the discharge also the tidal prism has been influenced, in Figure 5.63 a comparison between the simulations can be seen. The tidal prism in this simulation is still flood dominated; shorter flood duration then ebb.



Figure 5.61: Instantaneous discharge sim-5d

But during ebb tide now 950,000 m³ is flowing out the basin while during flood 760,000 m³ flows into the basin. With the same amount of longshore transport of 2,900,000 m³/year, the P/M ratio now is a little bit higher with 0.33 then in sim-5a with 0.26. The morphology of the inlet is presented in Figure 5.62 and shows similarities between the two runs, but what is clear is that less amount of





Figure 5.62: Bed level evolution of the inlet channel of Sim-5d after 1) 60 days, 2) 120 days, 3) 180 days 4) 240 days, 5) 300 days and 6) Cross-sectional evolution of the inlet

sediment is brought into the system favouring a smaller flood delta. The large amount of longshore transport and the low P/M ratio causes the inlet to close. A bar-bypassing mechanism still is the dominant process that forces the inlet to close. The time period of closure extended compared to sim-5a from 195 days to 210 days in this simulation.



Figure 5.63: Tidal prism of sim-5d and sim-5a

5.9.2 Sim-3d: Medium basin, neap tide range (0.5 m) with high waves (2.5 m) and a river discharge (20 m³/s)

This simulation is similar to "sim-3b" from scenario-b (5.3.3) only now a discharge of 20 m³/s is added that flows into the basin and flushes out to the ocean. This represents a yearly runoff of 631×10^6 m³, which is 66% of the mean annual runoff.

Comparing the results with sim-3b the water levels are slightly higher and after 180 days (7 Jan) the water levels are rising to 0.6 m. above MSL.



Figure 5.64: Water levels; basin and ocean sim-3d

The low waters are around 0 to 0.08 m. above MSL, hence the ratio between the ocean and bay is in the first period 0.9 and 1.08 at the end of simulation time. The reason that higher water levels in the basin are found are due to the discharge in the basin and the sand shoal that forms a sort of barrier in front of the inlet blocking the outflow. At this stage flows are strong enough to maintain a

connection. Looking at the instantaneous discharge through the inlet and comparing them with sim-3b, the differences can be seen in Figure 5.65. Larger ebb flows are observed of 70 m³/s to 80 m³/s the first 180 days and thereafter peaks to 125 m³/s. Flood flows are quite constant in the range of 40 m³/s to 50 m³/s. The tidal prism can be seen in Figure 5.67, where both the simulations are included. The main difference is the ebb flow that carries 200,000 m³

the north from shore normal direction in the first 180 days.



Figure 5.65: Instantaneous discharge sim-3b

more than the flood flows carry into the basin. The amount of the ebb flows is 950,000 m³ while the flood flows are 750,000 m³. At the end of simulation time the ebb flows are 1,500,000 m³ whereas the flood flows are 500,000 m³. This difference leads to more stability which can be seen in the cross-sectional area of the inlet (Figure 5.66-6), the depth of the inlet develops from 2.5 m. after 60 days to a depth of 5 m. at the rest of simulation. The inlet system changes from a flood-dominant system to an ebb-dominated system regarding sediment transport. All the sediments that flow into the basin are transported back to the ocean. Also the inlet gorge stabilizes, but when looking at morphological evolution of the inlet (Figure 5.66) the tidal gorge bends approximately 25 degrees to





Figure 5.66: Bed level evolution of the inlet channel of Sim-3d after 1) 60 days, 2) 120 days, 3) 180 days 4) 240 days, 5) 300 days and 6) Cross-sectional evolution of the inlet

The ebb currents are diverted by the longshore currents which are stimulated by the angle of the breaking waves. Between 180 and 240 days a shoal in front of the inlet forms and due to enormous supply of sand by longshore transport this shoal finally will form a spit from the updrift side of the inlet. The sand accumulates at the end of the spit and causes the inlet to migrate north by eroding the downdrift barrier, this can be seen after 300 days of simulation



Figure 5.67: Tidal prism of sim-3d and sim-3b

time. From the ratio between ocean and bay it can be seen that a larger water level difference results that occurs after 180 days. The retardation of the exchange of water between ocean and bay is the result of this rise. This makes the inlet vulnerable to breaching and closure of the old inlet location. A longer simulation would clarify the development.

5.10 Analysis discharge simulations

In the above section two simulations were carried out with as extra input a discharge, this represents the influence of the Mfolozi River discharge.

In Sim-5d only 16% of the mean annual runoff or 5 m³/s was applied and simulated. The effect of the discharge helps to keep the inlet longer open for 15 days. The effect on the tidal prism resulted in more net ebb flow than flood flow, but still a flood dominant system was observed. the closure mechanism was compared to Sim-5a similar. Longshore transport and a bar-bypassing coast resulted in the closure.

The second discharge simulation; Sim-3d, shows that due to a discharge of 20 m³/s, 66% of the mean annual runoff, the inlet system is more stable. This can be seen from the cross-sectional area of the inlet, and the depth that is more constant. But due to the high rate of longshore transport a spit forms at the end of the barrier. As described by Davis and Fitzgerald (2004) in model 1: "Inlet migration and spit breaching" the same mechanism is also observed in this simulation. The discharge



influence shows that a longer open connection is maintained than the same simulation without discharge. A longer simulation would give more understanding in an eventual breaching process or closure of the system. Although this is not simulated it can be expected that the inlet will close due to retardation of the exchange of water between the basin and ocean.

Figure 5.68: Model 1: Inlet migration and spit breaching

5.11 A-P relationship

The A-P relationship described in section 2.5.5 describes the relation found from O'Brien (1969). From the scenarios a set of simulations were further investigated with regarding to the A-P relationship. A comparison of the Delft3D simulations is made with other numerical studies; (Tran et al., 2011) and (Lam, 2009) but also earlier studies about natural tidal entrances in Florida (Powell, Thieke and Mehta, 2006).

In the figures below the Delft3D simulations are compared with O'Brien's findings. The crosssectional development as a function of the tidal prism is plotted; the initial point of the inlets are all located with the same starting cross-sectional area of 131 m². The dotted line connects the simulation points from start to end in which the filled markers are the end points after 300 days of simulation.



Figure 5.69: A-P relationship for sim-1a, -1b and -1c

Figure 5.70: A-P relationship for sim-4a, -4b and -4c

The found C-value for the first set of inlets (Figure 5.69) is $1.3 \ge 10^{-4}$ with q = 1. This was done using a linear correlation function which resulted in a fit with $R^2 = 0.98$. This value can be considered as close to the found value of O'Brien from his study in which he limited the entrances to only 8 non-jettied entrances; he found there a $C = 1.08 \ge 10^{-4}$ and q = 1. The grey dotted line shows a 25% confidence interval.

Looking at the second sets of inlets (Figure 5.70) a C value of 6.5 x 10^{-5} is found with q = 1. The corresponding R² = 0.92. The found value by O'Brien under the assumption of Escoffier that the equilibrium velocity u_{eq} is approximately 0,9 m/s, the associated values; q and C are respectively 1 and 7.8 x 10^{-5} . These inlets fall between the 25% confidence interval.

The last experiment consists of two inlets which were two simulations from the sensitivity analysis. The results with regard to the C-value are found with a linear fit with a R² value of 1. In this experiment the relation between the cross-sectional area and the tidal prism is A = 8.63×10^{-5} P. C = 8.63×10^{-5} .

Comparing the results with the data from Powell et al; 67 sandy entrances in Florida, data from Tran et al. 5 experiments with Delft3D



Figure 5.71: A-P relationship for sim-1aa and sim-1bb

and Lam's results, the overall C-value found from the simulations done in this study from this study show that St Lucia has an average C-value of 9.46 x 10⁻⁵. The R² value is 0.65, this can be explained by the fact that different simulations where combined with different littoral drift values and hence a different wave forcing.



Figure 5.72: Comparison C-values

The found C-values with Delft3D and the compared C-values found in nature show that Delft3D is capable of finding a good correlation between the cross-sectional area and the tidal prism. This suggests that the numerical model is capable of producing decent results with regard to the hydrodynamics and morphological development.

6. Conclusions & recommendations

This study investigates the hydrodynamics and morphodynamic behaviour of the St Lucia estuary mouth, in order to get a better understanding on the closure mechanism and the processes that influence the mouth dynamics. In addition the influence of the Mfolozi River discharge was investigated. After conducting a literature study where the basics regarding tidal inlets was studied and information was gathered regarding closure mechanisms and stability relationships, a process-based model; Delft3D was executed, to carry out simulations of a schematised situation of the St Lucia Estuary mouth.

In this chapter the main conclusion is written followed by an elaboration of the research and sub questions in relation to the modelling results. Subsequently recommendations will be made.

6.1 Conclusion

Research question:

• What is the main cause of the closure of the St Lucia inlet and which characteristic processes influence the morphological behaviour of the inlet?

From the three scenarios and the conducted sensitivity simulations in total ten simulations did close. All the simulations that closed had a P/M ratio below 1. The observed processes and mechanisms of these simulations were mainly relative weak tidal currents in combination with strong longshore currents generating sediment transport. Spit growth on the updrift and downdrift side of the coastal barriers constricted the inlets in combination with bar-bypassing in front of the inlet were the main driving mechanisms of the closure. The simulations show that the closing inlets were subject to a flood-dominated character which means stronger flood currents and weaker ebb currents. All of the inlets that closed did that during ebb tide which can be explained by lower tidal currents and stronger interrupting longshore currents with the additional effect of an inlet that has already been narrowed by updrift and downdrift spit growth. Although this closure mechanism is observed it must be kept in mind that the observed closure process is based on model simulations in which morphological up scaling is applied and therefore the closure at ebb tide is triggered by a numerical effect and does not necessarily mean that in nature or at St Lucia these systems close during ebb tide. The influence of a smaller D_{50} results in an increase of the amount of longshore sediment transport. Less migration of the tidal gorge due to more sediment bypass along the inlet channel. There is also less sediment import into the estuary and therefore less spit growth updrift of the barrier.

Sub questions:

• What are the governing characteristics of the hydrodynamic processes such as tides, waves and currents?

St Lucia is situated at a very dynamical environment having a high energy wave climate, a lowmesotidal regime and a high rate of longshore sediment transport. The wave climate shows a seasonal pattern with high waves in spring and moderate waves in the summer. The overall yearly wave data demonstrates that the yearly significant wave height is 1.6 meter and the governing extreme wave heights used in the simulations are increasing from 2 to 3.5 meters. In the table below the corresponding exceedance probabilities are given. The high rate of longshore transport is related to on the one hand the high wave energy climate but also due to the high offshore wave angle to shore normal direction. A mean wave direction of 50 degrees from southeast direction is found to be representative.

Hs	P _{exceedence}	Days
1.6	50%	182.5
2	18.40%	67.16
2.5	8.30%	30.295
3	3.20%	11.68
3.5	0.80%	2.92

• How do the hydrodynamic processes influence the morphological behaviour of the St Lucia estuary inlet?

Under influence of the most important hydrodynamic forcing parameters; waves and tide the following influence of the tide is found:

- A neap tide range of 0.5 meter result in low tidal currents who are not able to handle large quantities of sediment to the estuary. In terms of morphology of the estuary and inlet, it can be concluded that a low tidal range results in a small or no formation of a flood delta. Small tidal currents do not result in inlets that migrate but rather maintain its location. The eroding

capacity is not strong enough for maintaining a minimal cross-sectional area and therefore these inlets have not a stable equilibrium in terms of A-P relationship or in contrast with the Escoffier curve. These inlets are rather placed below the critical velocity curve of Escoffier and therefore closing. These mechanisms are found in the simulations Sim-3a, Sim-3b, Sim-2c and Sim-3c.

- A spring tide range of 2 meter is favouring sediment import due to strong tidal inlet currents. By more sediments imported into the estuary a clear development of a flood delta is observed. In terms of the location stability and the migration of these simulations it can be concluded that a spring tide in combination with strong littoral drift emphasizes the effect of migrating inlets, this can be seen in simulations Sim-4b, Sim-5b, Sim-4c and Sim-5c. In terms of cross-sectional stability, the simulation time is too short to say something about the equilibrium state of the inlets. But from the A-P relationship the inlets with spring tide, show that even these simulations tend to go to closure because both the tidal prism and crosssectional area are decreasing. See simulations Sim-4a, Sim-4b and Sim-4c.
- What is the influence of the discharge of the Mfolozi River?

The Mfolozi River is modelled with a small percentage of its mean annual runoff; 66 % and 16%. The influence of a discharge of 5 m³/s helps to maintain the inlet to be open for one tidal cycle or in terms of morphology 15 days. In terms of tidal prism, it maintains a flood-dominant system but stronger ebb flows are observed.

The influence of 20 m³/s transforms the inlet from a system that closed in 195 days, to a system that maintains its connection for 300 at least days. The inlet system changes from a flood dominant system to an ebb dominant system regarding sediment transport. Strong ebb flows result in a more stable cross-sectional area of the inlet, but under strong wave energy conditions with a large longshore sediment transport rate, a large sand shoal eventually forces the inlet to migrate and becomes vulnerable for closure. This was illustrated in Sim-3d.

• What is the influence of waves on sediment transport under specific representative wave conditions?

Waves are the prime movers of sediment and Delft3D is able to produce a longshore amount of sediment transport which is well in accordance to the known theoretical formulas such as the

Kamphuis (1991) formula and the formula proposed by Bayram et al (2007). Keeping in mind the limitation of cross-shore transport in Delft3D, controlling the longshore transport by manipulating input parameters that multiply suspended and bed load transport due to the currents in the surf zone, is found to be a successful way to conduct a research regarding inlet dynamics and closure mechanisms. What must not be ignored is the fact that the physical processes changed by switching off cross-shore transport and thereby not all the physical processes such as depth changes, current differences and turbulence in front of the inlet.

• What are the timescales for closure and what is the relationship to longshore transport rates?

From the scenarios and sensitivity simulations the timescale of closure in terms of P/M ratio is found to be in the range of 113 to 278 days. Dependant on the P/M ratio the closure in days can be seen as a good correlation with the P/M value. Lower values have higher longshore sediment transport, while higher values have a more dominant tidal prism. In contrast with Bruun (1978) his proposed findings state that P/M ratio > 20 is required for infinite closure time. The simulated scenarios are all below this ratio; it can be concluded that the ones that closed had all a lower ratio than 1, but for the other simulations a longer simulation period would give a better understanding of the stability and to which equilibrium state the inlet develops.



6.2 **Recommendations**

For future studies at St Lucia there are several interesting topics, they are all related to coastal engineering. The following topics are possible for future research:

- The development of a detailed model of the St Lucia Estuary mouth, forced with a seasonal varying wave climate
- Influence of cross-shore sediment transport on the closure of the inlet
- Modelling cyclonic events with extreme wave heights
- Simulating breaching events of the St Lucia berm due to rising lake water levels

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8. Appendix

8.1 Delft3D-FLOW

Delft3D-FLOW solves the unsteady shallow water equations in 2D or 3D for an incompressible fluid, under the shallow water and Boussinesq assumptions. In the vertical momentum equation the vertical accelerations are neglected, which leads to the hydrostatic pressure equation. This makes it able to calculate non-steady flow and transport phenomena that result from tidal and meteorological forcing or wind stress at the free surface. The computations are completed on a rectangular or curvilinear, boundary fitted grid. The system of equations consists of the horizontal equations of motion, the continuity equation, and the transport equations for conservative constituents. In combination with a set of boundary and initial conditions the hydrodynamics are provided to the sediment transport equations which determine the morphological changes at the bed level.

The FLOW module used for this study is the 2D mode. This approach means that only one layer is used for the water depth (depth-averaged approach). The governing equations of the flow module are the depth averaged continuity equation and the depth-averaged momentum equations in horizontal direction.

8.2 Delft3D-WAVE

Delft3D-WAVE uses the third-generation SWAN model see (Holthuijsen, Booij and Ris, 1993) for the simulation of the evolution of random, short-crested wind generated waves. The governing equation used by SWAN is the discrete spectral action balance equation which is fully spectral in all directions and frequencies. This means that the short-crested waves can propagate from widely different directions simultaneously.

The SWAN model accounts for refractive-propagation due to current and depth and represents the processes of wave generation by wind. The dissipative terms in the equation represent whitecapping, bottom friction and depth-induced wave breaking and non-linear wave-wave interactions both quadruplets and triads explicitly with advanced formulations. Wave blocking by currents is also explicitly represented in the model.

8.3 **Delft3D equations**

Neglecting the evaporation and precipitation the depth-averaged continuity equation reads as follows:

$$\frac{\partial \eta}{\partial t} + \frac{\partial (d+\eta)u}{\partial x} + \frac{\partial (d+\eta)v}{\partial y} = 0$$

The momentum equations in x- and y-direction are:

$$\frac{\partial u}{\partial t} + u\frac{\partial u}{\partial x} + v\frac{\partial u}{\partial y} + g\frac{\partial \eta}{\partial x} - fv + \frac{\tau_{bx}}{\rho_w(d+\eta)} - \frac{F_x}{\rho_w(d+\eta)} - \upsilon\left(\frac{\partial^2 u}{\partial x^2} + \frac{\partial^2 u}{\partial y^2}\right) = 0$$
$$\frac{\partial v}{\partial t} + u\frac{\partial v}{\partial x} + v\frac{\partial v}{\partial y} + g\frac{\partial \eta}{\partial x} + fu + \frac{\tau_{by}}{\rho_w(d+\eta)} - \frac{F_y}{\rho_w(d+\eta)} - \upsilon\left(\frac{\partial^2 v}{\partial x^2} + \frac{\partial^2 v}{\partial y^2}\right) = 0$$

where,

 η = water level [m], d = water depth [m], u,v =depth averaged velocity [m/s], f = Coriolis parameter [s⁻¹], F_{x,y} = x- and y-component of external forces [N/m²], ρ_w = mass density of water [kg/m³], υ = diffusion coefficient (eddy viscosity) [m²/s], g = gravity of acceleration [m/s²], $\tau_{bx,y}$ = x- and y-component of the bed shear stress [N/m²].

The WAVE-module uses SWAN, to the governing equation to describe the evolution of the wave spectrum is the spectral action balance equation:

$$\frac{\partial}{\partial t}N + \frac{\partial}{\partial x}c_{x}N + \frac{\partial}{\partial y}c_{y}N + \frac{\partial}{\partial \sigma}c_{\sigma}N + \frac{\partial}{\partial \theta}c_{\theta}N = \frac{S}{\sigma}$$

with:

 $N(\sigma, \theta)$ = the action density spectrum, σ = relative frequency (observed in a frame of reference moving with the current velocity), θ = wave direction (direction normal to the wave crest of each spectral component), c_x and c_y = propagation of action, c_σ = propagation velocity in σ -space, c_{θ} = propagation velocity in θ -space and $S(\sigma, \theta)$ = source term representing the effects of generation, dissipation and non-linear wave-wave interactions.

8.4 Transport formulations for non-cohesive sediment

The sediment transport and morphology module supports both bed-load and suspended load transport of non-cohesive sediments and suspended load of cohesive sediments. In this study non-cohesive sediments are considered. There are several formulations which can be chosen. In Table 8.1 an overview of the formulations are given.

Formula	Red load	Waxaa
Formula	Ded load	waves
Van Rijn (1993)	Bed load + suspended	Yes
Engelund-Hansen (1967)	Total transport	No
Meyer-Peter-Muller (1948)	Total transport	No
General formula	Total transport	No
Bijker (1971)	Bed load + suspended	Yes
Van Rijn (1984)	Bed load + suspended	No
Soulsby/Van Rijn	Bed load + suspended	Yes
Soulsby	Bed load + suspended	Yes
Ashida-Michiue (1974)	Total transport	No

Table 8.1: Additional transport formulations

8.5 Delft 3D settings

Table 8.2: Delft3D input settings

Parameters	Descriptions	Value
Δt	flow time step (s)	6
ę₩	water density (kg/m ³)	1025
К	horizontal eddy viscosity (m^2/s)	1
Ν	horizontal eddy diffusitivity (m ² /s)	0,1
n	Manning	0,026
Strwav	stress formulation due to wave forces (-)	Bijker
Sediment		
QS	Specific density (kg/m ³)	2650
SedDia	Median sediment diameter (d ₅₀) (mm)	300

RelDens	Dry bed density (kg/m ³)	1600
InitSedThick	Initial sediment layer thickness at bed (m)	10
Numerical parameters		
•		Grid cell entres and
Dryflood	Drying and flooding check	faces
DepCell	Depth specified at	Grid cell entres
TresDep	threshold depth (m)	0,1
SmoTime	Smooting time (min)	60
ForFilter	Forester filter (horizontal)	true
Morphology		
MorUpd	Update bathymetry during FLOW simulation	true
DensIn	Include effect of sediment on fluid density	false
EqmBc	Equilibrium sand concentration profile at inflow boundaries	true
MorFac	Morpholgical scale factor (-)	30
MorStt	Spin-up interval before morphological changes	720
SedThr	Minimum depth for sediment calculation	0,35
AksFac	van Rijn´s reference height factor	1
Thresh	Threshold sediment thickness (m)	0,050001
RWave	Estimated ripple height factor (-)	2
ThetSD	Factor for erosion of adjacent dry cells	0,5
SusC	Current-related suspended sediment transport factor	5
BedC	Current-related bed-load transport factor	5
SusW	Wave-related suspended transport factor	0
BedW	Wave-related bed-load sediment transport factor	0
AlfaBs	Longitudinal bed gradient factor for bed load transport for formulation 1	1
AlfaBn	Transverse bed gradient factor for bed load transport for formulation 2	1,5
	Neumann boundary condition for non-cohesive suspended	
NeuBcSand	sediment concentrations at inflow boundaries	TRUE
Wave		
spectrum	shape of wave spectrum (-)	JONSWAP
peakenh	peak enhancement factor JOHNSWAP spectrum (-)	3,3
wave	wave-related water level setup (-)	FALSE
forcing	computation of wave forces (-)	Radiation stresses
breaking	depth-induced breaking model (-)	B&J model
alfa	coefficient for wave energy dissipation in the B&J model (-)	1
gamma	breaker parameter in the B&J model (-)	0,73
triads	non-linear triad wave-wave interactions (-)	false
bottom	bottom friction formulation (-)	JONSWAP
coeff	coefficient for bottom friction (-)	0,067

diffraction	diffraction taken into account	FALSE
wind	formulation for exponential wave growth (-)	FALSE
white	formulation for white capping (-)	FALSE
quadruplets	quadruplet wave-wave interactions (-)	FALSE
refrac	refraction is activated for waves propagation (-)	TRUE
freqshift	frequency shift is activated for wave propagation (-)	TRUE

8.6 Longshore sediment transport theory vs. Delft3D



In this paragraph the longshore sediment transport is presented.





8.7 Influence of SusW, BedW, SusC and BedC on longshore and cross-shore transport
8.8 Wave angle when breaking

Simulation	α0 [°]
Sim-1a	17
Sim-2a	17
Sim-3a	20
Sim-4a	20
Sim-5a	22
Sim-1b	17
Sim-2b	19
Sim-3b	21
Sim-4b	20
Sim-5b	22
Sim-1c	16
Sim-2c	20
Sim-3c	23
Sim-4c	22
Sim-5c	25
Sim-1aa	18
Sim-3aa	20
Sim-5aa	22
Sim-1bb	18
Sim-3bb	22
Sim-5bb	23
Sim-1cc	24
Sim-2cc	20
Sim-3cc	22

8.9 C-values

Powell et al. (2006), O'Brien (1969) and St Lucia.



Table 8.3: Comparison of findings for comparable inlet situations (Stive and Rakhorst, 2008)

Author's	С	q	Tidal prism	Location
O'Brien (1969)	1.08 10-4	1	Mean spring	8 non-jettied entrances US
Powell et al (2006)	6.25 10-5	1	Mean spring	66 Florida entrances
Eysink (1990)	7.0 10-5	1	Mean tide	Dutch Wadden Sea entrances
Rakhorst (2007)	5.65 10-5	1	Max of mean ebb or flood	Dutch Western Wadden Sea entrances
Rakhorst (2007)	7.75 10-5	1	Max of mean ebb or flood	Dutch Eastern Wadden Sea entrances
Vd Kreeke & Haring (1979)	8.2 10-5	1	?	Dutch Zeeland entrances



8.10 Sediment transport through the inlet from Sim-a

Figure 8.1: Sediment transport through the inlet Sim-a



8.11 Sediment transport through the inlet from Sim-b

Figure 8.2: Sediment transport through inlet Sim-b



8.12 Sediment transport through the inlet from Sim-c

Figure 8.3: Sediment transport through the inlet Sim-c



8.13 Tidal prism / cumulative discharge through the inlet; Sim-a

Figure 8.4: Cumulative discharge through the inlet Sim-a



8.14 Tidal prism / cumulative discharge through the inlet; Sim-b

Figure 8.5: Cumulative discharge through the inlet Sim-b



8.15 Tidal prism / cumulative discharge through the inlet; Sim-c

Figure 8.6: Cumulative discharge through the inlet Sim-c



8.16 Instantaneous discharge through the inlet Sim-a

Figure 8.7: Instantaneous discharge through the inlet Sim-a



8.17 Instantaneous discharge through the inlet Sim-b

Figure 8.8: Instantaneous discharge through the inlet Sim-b



8.18 Instantaneous discharge through the inlet Sim-c

Figure 8.9: Instantaneous discharge through the inlet Sim-c



8.19 Water levels in the basin and ocean Sim-a

Figure 8.10: Water levels of the basin and the ocean Sim-c



8.20 Water levels in the basin and ocean Sim-b

Figure 8.11: Water levels in the basin and the ocean Sim-b



8.21 Water levels in the basin and ocean Sim-c

Figure 8.12: Water levels in the basin and the ocean Sim-c



8.22 Longshore sediment transport along the coast Sim-a



8.23 Longshore sediment transport along the coast Sim-b



8.24 Longshore sediment transport along the coast Sim-c





8.25 Longitudinal evolution of the inlet Sim-a

Figure 8.16: Evolution of the longitudinal profile of the inlet Sim-a



8.26 Longitudinal evolution of the inlet Sim-b

Figure 8.17: Evolution of the longitudinal profile of the inlet Sim-b



8.27 Longitudinal evolution of the inlet Sim-c

Figure 8.18: Evolution of the longitudinal profile of the inlet Sim-c





Figure 8.19: Depth averaged velocity of Sim-a





Figure 8.20: Depth averaged velocity of Sim-b

8.30 Depth averaged velocity in the inlet



Figure 8.21: Depth averaged velocity of Sim-c

8.31 All data simulations

	distance	from	coast [m]		220.00	240.00	380.00	250.00	340.00	220.00	300.00	380.00	240.00	340.00	200.00	350.00	480.00	300.00	480.00	250.00	250.00	350.00	250.00	425.00	350.00	425.00	340.00	420.00
		fidal prism	bservations	t Lucia [m ³]	332996	445099	454445	495003	594964	822980	851397	965566	1198524	1333668	1388517	1495033	1586904	160000	1829419	1887813	2067179	2984691						
Observed	longshore	transport T	St Lucia o	[m ³ /year] S	-420000	-260000	-60000	440000	720000	830000	850000	000086	1010000	1020000	1220000	1570000	1880000	2120000										
	Time	to	closure	[d]			240		195			195				210	135				113			174		210	278	158
				m ³ /s																								
				H_0/L_0 [-]	8.48E-03	8.48E-03	1.32E-02	1.32E-02	1.59E-02	8.48E-03	1.06E-02	1.32E-02	1.32E-02	1.59E-02	8.48E-03	1.32E-02	1.59E-02	1.59E-02	1.85E-02	1.06E-02	1.32E-02	1.59E-02	1.06E-02	1.59E-02	1.85E-02	1.85E-02	1.32E-02	1.59E-02
				4/A	0.645714	0.617143	3.92	4	8.285714	0.128571	0.309143	0.792	0.842286	1.714286	0.074286	0.4	0.8	0.857143	1.505714	1.462857	8.765714	4.062857	0.308571	1.657143	1.657143	1.228571	0.371429	0.828571
			ot / 5	3/year] N	45200	43200	274400	280000	580000	45000	108200	277200	294800	600000	52000	280000	560000	600000	1054000	256000	613600	1422000	270000	1450000	2900000	3666666.667	650000	1450000
			M	= P/M [m	2.4	1.1	0.2	0.5	0.3	8.0	1.3	0.5	1.9	0.9	13.1	0.7	0.4	1.2	0.8	0.0	0.1	0.5	3.3	0.2	0.0	0.8 28	0.8	0.3
		Bayram et	al.	[m ³ /year]	67824	64053	235979	257034	602631	70899	128330	333160	337880	608846	68795	322298	693923	711389	1192519	286272	537252	1094827	286272	975676	1788375	1728232	556011	979058
			Kamphuis	[m ³ /year]	99723	99723	235455	235455	342644	99723	156502	241199	235455	342644	97832	235455	349898	342644	484981	166633	258253	382076	166633	376331	513456	523585	260574	376331
		h, [m]			3.3	3.7	6.11	5.15	6.4	3.02	4.65	5.74	5.15	6.27	3.02	5.66	1.7	6.3	8.03	4.05	5.85	6.1	4.05	7.25	L	8	5.65	7.2
		H, [m]			1.52	1.52	2.24	2.24	2.64	1.52	1.85	2.24	2.24	2.64	1.53	2.24	2.64	2.64	3.05	1.84	2.23	2.65	1.84	2.63	3.04	3.04	2.24	2.63
		α ₀ [°]			17	17	20	20	22	17	19	21	20	22	16	20	23	22	25	18	20	22	18	22	23	24	20	22
		H ₀ [m			1.60	1.60	2.50	2.50	3.00	1.60	2.00	2.50	2.50	3.00	1.60	2.50	3.00	3.00	3.50	2.00	2.50	3.00	2.00	3.00	3.50	3.50	2.50	3.00
		D50[m]			0.0003	0.0003	0.0003	0.0003	0.0003	0.0003	0.0003	0.0003	0.0003	0.0003	0.0003	0.0003	0.0003	0.0003	0.0003	0.0002	0.0002	0.0002	0.0002	0.0002	0.0002	0.0002	0.0002	0.0002
		T [s]	2		11.00	11.00	11.00	11.00	11.00	11.00	11.00	11.00	11.00	11.00	11.00	11.00	11.00	11.00	11.00	11.00	11.00	11.00	11.00	11.00	11.00	11.00	11.00	11.00
		P/M	-		2.43	1.06	0.17	0.54	0.26	8.00	1.29	0.51	1.90	0.93	13.08	0.71	0.36	1.23	0.82	0.88	0.07	0.53	3.33	0.21	0.00	0.76	0.77	0.28
	2	Mtot	[m ³ / yea		226,000	216,000	1,372,000	1,400,000	2,900,000	225,000	541,000	1,386,000	1,474,000	3,000,000	260,000	1,400,000	2,800,000	3,000,000	5,270,000	512,000	3,068,000	1,422,000	540,000	2,900,000	2,900,000	4,300,000	1,300,000	2,900,000
		Susc/Bedc			5.0	5.0	5.0	5.0	5.0	5.0	5.0	5.0	5.0	5.0	5.0	5.0	5.0	5.0	5.0	2.0	5.0	1.0	2.0	2.0	1.0	1.5	2.0	2.0
		[m]			50,000	30,000	30,000	000,000	60,000	800,000	000,000	000,000	800,000	800,000	400,000	000,000	000,000	700,000	,300,000	50,000	000,000	50,000	800,000	000,000	600,000	250,000	,000,000	00,000
		- - -	Ε		0.67 5	0.25 2.	0.25 2.	1.00 74	1.00 7	0.67 1.	0.25 70	0.25 70	1.00 2.	1.00 2	0.67 3.	0.25 1.	0.25 1.	1.00 3.	1.00 4.	0.67 4	0.25 20	1.00 7	0.67 1.	0.25 60	1.00 2.	0.67 3.	0.25 1.	0.25 8
		A_{h} $[m^2]$			350,000.00	350,000.00	350,000.00	350,000.00	350,000.00	1,750,000.00	1,750,000.00	1,750,000.00	1,750,000.00	1,750,000.00	3,500,000.00	3,500,000.00	3,500,000.00	3,500,000.00	3,500,000.00	350,000.00	350,000.00	350,000.00	1,750,000.00	1,750,000.00	1,750,000.00	3,500,000.00	3,500,000.00	3,500,000.00
		Ĕ	Ξ		1.60	1.60	2.50	2.50	3.00	1.60	2.00	2.50	2.50	3.00	1.60	2.50	3.00	3.00	3.50	2.00	2.50	3.00	2.00	3.00	3.50	3.50	2.50	3.00
					Sim-1a	Sim-2a	Sim-3a	Sim-4a	Sim-5a	Sim-1b	Sim-2b	Sim-3b	Sim-4b	Sim-5b	Sim-1c	Sim-2c	Sim-3c	Sim-4c	Sim-5c	Sim-1aa	Sim-3aa	Sim-5aa	Sim-1bb	Sim-3bb	Sim-5bb	Sim-1cc	Sim-2cc	Sim-3cc