RIP CURRENTS
morphodynamic simulations

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delft hydraulics
Rip currents are narrow and intense, offshore directed flows in the nearshore zone. A rip current is able to transport a considerable amount of water and sediment seaward and may dominate the morphological exchange between surfzone and the offshore.

Rip currents are not a permanent phenomenon. Under a combination of favourable hydrodynamic conditions they may be generated. Within half a day rips can develop more pronounced. The rip channel may migrate along the open beach and vanish after having persisted for some time, up to several months.

The objective of this study is to construct a simulation of the hydrodynamics and morphodynamics associated with an initial rip current. It is investigated in what way the simulations can be validated by the ARGUS video system. This system generates images of the nearshore zone on an hourly basis.

The simulations in this study are executed in DELFT2D-MOR, taking into account wave-current interaction.
Contents

List of Figures
List of Tables

Summary................................................................. 1

1 Introduction.......................................................... 1-1
   1.1 Rip current systems............................................. 1-1
   1.2 Objective...................................................... 1-2
   1.3 Coastal dynamics ............................................... 1-2
       1.3.1 Context ARGUS Program .................................. 1-3
       1.3.2 Computational morphodynamics............................. 1-3
   1.4 Outline ....................................................... 1-4

2 Description of Rip Currents........................................ 2-1
   2.1 Review of rip current related literature.................... 2-1
       2.1.1 Description of a rip system ....................... 2-1
       2.1.2 Hydrodynamic characteristics of a rip current .... 2-2
       2.1.3 Morphological rip current characteristics .......... 2-4
       2.1.4 Life cycle, migration and persistence ................ 2-5
   2.2 ARGUS images (Noordwijk Situation) .......................... 2-5

3 Models for Rips and their generation ............................ 3-1
   3.1 Hydrodynamic mechanisms on a longshore uniform topography 3-1
       3.1.1 Wave-wave interaction models ....................... 3-1
       3.1.2 Wave-current interaction models for rip currents .... 3-3
       3.1.3 Instability mechanisms ............................... 3-4
   3.2 Longshore non-uniformities in morphology ..................... 3-4
       3.2.1 Bottom topography ..................................... 3-4
       3.2.2 Presence of lateral boundary ......................... 3-6

4 Wave induced flow.................................................. 4-1
   4.1 Averaged Equations for free surface flow .................... 4-2
       4.1.1 Averaging ............................................. 4-2
       4.1.2 Mean momentum ........................................ 4-2
       4.1.3 Averaged equation of mass conservation ............... 4-3
4.2 Waves ................................................................................. 4–3
  4.2.1 Description of short waves on a current .................. 4–4
  4.2.2 Wave energy and flux .............................................. 4–4
  4.2.3 Energy equation ...................................................... 4–5
  4.2.4 Wave action ........................................................... 4–5

4.3 Wave-current interaction in a rip ............................................. 4–6
  4.3.1 Wave transformation ................................................ 4–6
  4.3.2 Forcing of the current .............................................. 4–7

4.4 Addition of wave mass flux ...................................................... 4–10
  4.4.1 Wave and current separation .................................. 4–11
  4.4.2 Mass flux .............................................................. 4–12
  4.4.3 Equations with mass flux formulation ..................... 4–14
  4.4.4 Hydrodynamics with mass flux .............................. 4–17

5 Simulating Rip Currents using DELFT2D-MOR ................................. 5–1
  5.1 Aim of the simulations .................................................. 5–1

5.2 Physical process .................................................................. 5–2

5.3 DELFT2D-MOR ................................................................. 5–3
  5.3.1 Compound model MAIN ........................................ 5–4

5.4 Wave-Current Interaction ...................................................... 5–6
  5.4.1 Convergence and steady state ................................. 5–6
  5.4.2 Structure of the hydrodynamic computation ............. 5–8
  5.4.3 wave-current interaction TIME TESTS ................ 5–11

5.5 Outline of the simulations ..................................................... 5–12

6 Domain ................................................................................. 6–1
  6.1 Boundaries ..................................................................... 6–1

6.2 Bottom topography .............................................................. 6–2
  6.2.1 Longshore uniform beach ...................................... 6–3
  6.2.2 Initial rip channel .................................................... 6–3

6.3 Evaluation of the domain related tests ................................... 6–4

7 Hydrodynamic Modules ............................................................ 7–1
  7.1 Wave Module ............................................................... 7–1
    7.1.1 Grid dimensions and boundary ............................ 7–1
    7.1.2 HISWA grid resolution ....................................... 7–1
    7.1.3 HISWA boundaries ............................................ 7–2
    7.1.4 Wave height variations ...................................... 7–2
    7.1.5 Obliquely incident waves ................................. 7–3
    7.1.6 Wave dissipation and set-up due to an opposing current 7–3
    7.1.7 Evaluation of the simulations with varied wave input 7–4
RIP CURRENTS
morphodynamic simulations

7.2 Flow module .................................................. 7–5
7.2.1 Domain .................................................. 7–5
7.2.2 Bed shear stress ........................................... 7–8
7.2.3 Viscosity .................................................. 7–9
7.2.4 Evaluation of the flow simulations ................. 7–9

8 Morphodynamics ................................................. 8–1
8.1 Structure of the morphodynamic simulations ....... 8–1
8.2 Transport module ............................................ 8–1
  8.2.1 Transport formulation of Bijker ......................... 8–2
  8.2.2 Sediment Transport Magnitude ......................... 8–2
8.3 Bottom Module ............................................... 8–3
8.4 Time scale of bottom changes in simulations ....... 8–3
8.5 Evaluation of the morphodynamic simulations ....... 8–4

9 Morphodynamics with addition of wave mass flux .... 9–1
9.1 Hydrodynamics with wave mass flux ................. 9–1
9.2 Morphodynamics with mass flux ....................... 9–2

10 Conclusions and recommendations ..................... 10–1
10.1 Conclusions ................................................. 10–1
  10.1.1 Wave-current interaction ......................... 10–1
  10.1.2 Mechanism of Rip current generation .......... 10–1
  10.1.3 ARGUS images ....................................... 10–4
  10.1.4 Addition of wave mass flux ...................... 10–4
10.2 Expectations .............................................. 10–4
10.3 Recommendations ......................................... 10–5

Acknowledgements

References

Appendices

A Series of rectified ARGUS Images (Sep - Dec 1995)

B Derivation of the Short-wave depth averaged Equations for free surface motion
  B.1 General fluid motion
  B.2 Averaged Equation of Mass Conservation
  B.3 Averaged Equations of Momentum Conservation
C Wave-Current interaction
   C.1 Overview of possible configurations
   C.2 Wave Current interaction

D DELFT2D-MOR Simulations
   D.1 Domain related tests
   D.2 Waves related tests
   D.3 Flow related tests
   D.4 Morphodynamic simulations

E Wave Mass Flux
   E.1 Wave energy equation including mass flux
   E.2 Hydrodynamic simulations with mass flux
   E.3 Morphodynamic simulations with mass flux
List of Figures

Figure 1-1 Example of rip currents (Inman et al. 1971) .................................................. 1-1
Figure 2-1 Component parts of a rip current system ......................................................... 2-1
Figure 2-2 Float-measurement on Oct. 2 (De Graaff (1995)) ............................................. 2-3
Figure 2-3 Profiles over the rip neck, De Graaff (1996) ..................................................... 2-4
Figure 2-4 ARGUS image with a rip .................................................................................... 2-6
Figure 2-5 Rectified ARGUS images with a rip at about x = -700 m. ............................. 2-6
Figure 2-6 Rectified ARGUS image with filter for intensity peaks ................................. 2-7
Figure 2-7 Cross shore intensity profile from rectified ARGUS image ............................. 2-8
Figure 3-1 Radiation stress set-down and set-up across the shoreline ............................. 3-2
Figure 3-2 Definition sketch for Deigaard's model ......................................................... 3-5
Figure 4-1 Set up on normal and rip transect ................................................................. 4-9
Figure 4-2 Bottom elevation for normal and rip transect ................................................ 4-9
Figure 4-1 Velocity profiles ............................................................................................. 4-11
Figure 4-2 Drift velocity as seen from a Lagrangian point of view .................................. 4-13
Figure 4-3 Constitutive velocity vectors ........................................................................... 4-17
Figure 5-1 DELFT2D-MOR Process tree (example) ....................................................... 5-5
Figure 5-2 The course of the iteration process ............................................................... 5-7
Figure 5-3 Structure 1.43.1.1 ......................................................................................... 5-8
Figure 5-4 Structures of the computation ........................................................................ 5-10
Figure 5-5 Structure 15.90.1.10 .................................................................................... 5-11
Figure 5-6 EPS time test for TRISULA period ............................................................... 5-12
Figure 6-1 Computational domain configuration ............................................................ 6-2
Figure 6-2 Bottom profiles with widened trough ........................................................... 6-2
Figure 7-1 HISWA grid dimensions and resolution ......................................................... 7-2
Figure 7-2 TRISULA boundary configuration ................................................................. 7-6
Figure 7-3 Weakly reflective boundary conditions ......................................................... 7-7
Figure 8-1 Schematisation of morphological development for less frequent and frequent feedback .................................................................................................................. 8-6
Figure 10-1 Effect of way of modelling and stage of development on dissipation and set up in a rip system ................................................................. 10-3
List of Tables

Table 6-1 Velocities and spacing on a uniform beach with various wave heights .......... 6–3
Table 6-2 Velocities in initial rip with various trough widths ........................................ 6–4
Table 7-1 Velocities in initial rip with various wave heights ........................................ 7–2
Table 7-2 Velocities for various angles of wave incidence and bottom profiles ............... 7–3
Table 7-3 Velocities and dimensions of initial rip system for various friction values .......... 7–8
Table 7-4 Velocities in rip for various viscosity values ............................................... 7–9
Summary

Rip currents are narrow and intense, offshore directed flows in the nearshore zone. A rip current is able to transport a considerable amount of water and sediment seaward and may dominate the morphological exchange between surfzone and the offshore. Rip currents are often located nearby lateral boundaries that obstruct the longshore current, but they can appear as well on long straight beaches. Rip currents can arise with a regular spacing along the beach. In case of a barred beach profile the return flow is concentrated in eroded channels across the bar.

Rip currents are not a permanent phenomenon. Under a combination of favourable hydrodynamic conditions they may be generated. Within half a day rips can develop more pronounced. The rip channel may migrate along the open beach and vanish after having persisted for some time, up to several months.

The objective of this study is to construct a simulation of the hydrodynamics and morphodynamics associated with an initial rip current. It is investigated in what way the simulations can be validated by the ARGUS video system. This system generates images of the nearshore zone on an hourly basis.

A rip current is a wave induced flow. Transforming waves exert a force on the fluid mass in which they propagate. In order to balance this force, a pressure gradient or a velocity will arise. This leads to variations in the local mean water depth and currents in the nearshore zone. At the same time the generated current influences the waves. A current in the nearshore zone opposing the waves, like a rip current yield an increased wave height and a reduced wave length so that the wave is steepened, even up to breaking.

For successful simulations of an evolving rip current, it is required to know plausible mechanisms that are involved in rip current generation. In literature several theories can be found, explaining the generation of rip currents. Most of them assume differences in wave set-up as mechanism that drives the current. This difference in wave set-up can originate from either hydrodynamic interactions or morphological irregularities.

The simulations in this study are executed in DELFT2D-MOR, a numerical model developed at Delft Hydraulics. It comprehends modules for simulating hydrodynamics and morphodynamics. In order to obtain a realistic and steady state wave-current field, iterations between wave- and current computations are required. The most direct feedback between the hydrodynamic modules along with a short period of the flow simulations give the best results.

A barred profile is used as input for the simulations. On a uniform beach, without any longshore perturbation, the simulations show weak, but clearly distinguishable circulation cells. Simulations starting with a small initial rip channel show stronger currents. Increasing the width of the inner trough increases rip velocities.

Increasing of the wave height at the offshore boundary results in increase of the rip current velocity. But, beyond a certain value of the applied wave height, the velocity decreases because of rise of the set-up at the rip head. Waves approaching under a relatively large angle ( > 5 dgr to shore normal) fade the effect of the initial rip channel away. Small angles
of wave incidence stimulate the development of a rip. On the one hand because the longshore current is much stronger than the current only generated by differences in set-up. On the other hand because, the longshore current drops sediment from the nearshore zone not directly at the offshore end of the rip channel but more downstream.

In the initial rip, the breaking of the waves due to the depth at the bar face, coincides with the maximum current velocity. This results in an increase in the dissipation intensity more than a shift of the dissipation peak down stream. So the wave dissipation, as observed on ARGUS images from rips in Noordwijk, is strongly related to the development of sill at the rip head.

Morphodynamic simulations show the deposition of a sill at the downstream end of the rip and a slight growth of the rip channel. The morphological development of the rip, resulting from simulations is strongly dependent on the course of the morphodynamic computation. Frequent feedback shows a splitting of the current and sideward discharge. Longer morphological time steps between calling of the hydrodynamic modules show that the rip current is excavating through the sill.

Addition of wave mass flux formulations to the model does not modify the flow pattern of the initial rip current significantly. Differences resulting from taking wave mass fluxes into account, arise from other computed shear stresses, because these are based on a different defined velocity. Computations with wave mass flux cause an increase of the sediment transport rates. This effect is not confined to the rip so the whole beach profile strongly changes.
1 Introduction

This Chapter gives a briefly introduces the phenomenon of rip currents. It gives an exposition of the objective of this study and an overview of the order and relationships of the following Chapters of this report.

1.1 Rip current systems

Rip currents are seaward oriented flows. They flow through the surfzone and are spatially narrow and intense. In the case of a coast with a longshore bar, this return flow often takes place through eroded channels across the bar: the rip channels. Rips have been the focus of considerable scientific and public interest. The former because of their ability to rapidly move water and sediment seaward, and the latter for their ability to do likewise with swimmers.

Rip currents are spatially periodic features on most beaches. The rips may be located nearby topographic constraints of the flow field such as groins or headlands but are nevertheless observed on long straight beaches at near-normal incidence, suggesting an alternative origin in the nearshore hydrodynamics or possibly morphology.

Rip currents evolve in time. They are generated under a combination of favourable conditions. In course of time the rip channels may develop pronounced, migrate along the beach or persist on a certain location for a period in the order of months. But as quick as the phenomenon appears, it may vanish and stay away.

Figure 1-1 Example of rip currents (Inman et al. 1971)
Thanks to recent advances in 2D modelling morphodynamics in a horizontal domain, it is possible to study the morphological development of a rip current. Its relevance for coastal engineering is because of the supposed ability of rip currents to dominate the exchange of sediment between surf zone and offshore. Rips may play a significant role in coastal erosion because they especially move rather large quantities of sediment away from the beach. Understanding of the phenomenon 'rip currents' and estimating their strength may be useful for the maintenance of coasts (and eventually the design of artificial coasts). As far as known this is the first attempt of simulating the morphodynamic development of an initial rip.

1.2 Objective

The primary objective of this study is to construct a computational hydrodynamic and morphodynamic simulation of an evolving rip current system and investigate its possible validation through the ARGUS video system. This system generates near-shore images on an hourly basis.

Fundamental questions which need to be answered:

- What is the best way, using DELFT2D-MOR, of simulating wave-current interaction?
- What is the driving mechanism behind a rip system?
  - It is known that a rip current is a wave induced flow. But the question how waves and flow exactly interact, so that a fast developing rip is generated, is still not answered satisfactorily. Can simulations of the development of a rip, using numerical models, give an explanation of the mechanisms and conditions that are required for rip generation?
- Can ARGUS-images be used to identify rip current development and validate simulations of evolving rips?
  - Measurements of bathymetries at different stages are required to validate simulations, but these are rare. Instead of bathymetries, ARGUS images might be used, provided that a good insight exists in the factors which influence the location of wave dissipation (as the use of these images is based on this dissipation location). Moreover, the aim of this study is the extension of ARGUS video applications for monitoring nearshore processes.
- What is the relative importance of the mass flux of waves?
  - Recent models of rip currents, like that of Deigaard (1990), impose a strong onshore mass flux. Is this conditional for a realistic simulation of rip currents?

1.3 Coastal dynamics

Studying coastal dynamics, a distinction can be made between hydrodynamics and morphodynamics. Hydrodynamics itself refers to the motion of the sea water. Often a differentiation is made between waves and currents in order to schematise this motion. Watching the surfzone makes the distinction improbable but since no better way of schematising remains, it is the only practicable. In the constituent equations for coastal processes and consequently in the numerical models which can be used for simulations the same separation between waves and currents is carried through. This also applies for the description in this report.
Not only waves and currents do interact within the fluid motion, the underlying bottom does as well. When bottom changes are also taken into account there is a matter of hydrodynamics with evolving morphology and vice versa or morphodynamics.

### 1.3.1 Context ARGUS Program

By applying video imaging techniques, the knowledge of coastal dynamics can be enlarged considering that video monitoring provides useful information about hydrodynamics in the surfzone and the morphology of the coast. The ARGUS program offers great advantages, especially for the study of nearshore processes over kilometres of beach at any timescale (hours - decades) at relatively low cost. Moreover, continues monitoring, also during storm conditions, supplies valuable data which could not be obtained by field experiments. The images made on an hourly base are processed and added to a database.

In the framework of the Netherlands Centre for Coastal Research, Rijkswaterstaat strongly supports the development of ARGUS based techniques to monitor the nearshore zone. Currently ARGUS images are applied for both data-driven and process based modelling. In particular at Utrecht University efforts concentrate on the use of ARGUS video images as the source of information for data-oriented description of the appearance of the nearshore. The bright patterns of wave-breaking can be considered to reflect underlying bar bathymetry. Hence, changes in these patterns represent bar morphodynamics. By relating the behaviour of the patterns to the offshore measured forcing conditions (waves, tide) the knowledge of morphodynamics of a coastal system can be improved.

A second application concerns the use of ARGUS images in relation to process-based models, which is done particularly at Delft University and Delft Hydraulics. For instance, by interpreting the intensity distribution quantitatively as an expression of wave breaking, the bottom elevation can be modelled inversely. In that way, ARGUS images can independently, without field measurements on the beach, be used for accurate surveying of the nearshore zone.

This present study is part of the research on the application of numerical modelling in combination with ARGUS video images. Attention has so far been directed to:

- Quantification of bar bathymetry from video observations (Aarninkhof 1996)
- Intertidal beach level estimations from video images (Janssen 1997)
- Hydrodynamic modelling and video imaging of a rip current system. (de Graaff 1996)

This last mentioned study focused on the imitation of the dissipation pattern observed on video images by using hydrodynamic models. This present study continues and extends this effort by studying the wave current interaction of a wave induced flow with a changing morphology and interpreting the effect of wave current interaction on image intensities.

### 1.3.2 Computational morphodynamics

In this project, rips are studied, using numerical models developed at Delft Hydraulics. Not only hydrodynamics can be simulated but recent advances enable simulating morphodynamics as well. At Delft Hydraulics a lot of experience is acquired with computations of large scale morphodynamics. This study is a kind of test for the
applicability of DELFT2D-MOR in simulating the relatively small scale process of a rip current.

restrictions in this study

For the sake of simplicity several restrictions have been taken into account:

- Only wave-induced nearshore dynamics are considered, with zero mean flow far offshore and no tide;
- An open beach, no lateral boundaries constrain the flow;
- Simulations are aimed at steady state hydrodynamics. Accelerations of the fluid are suppressed. In contrast to the morphodynamics, these are treated quasi-steady;
- Linear wave theory is assumed to be valid. It is assumed that everywhere the bottom slope is so small that locally the expressions for phase, particle and group velocity in waves and pressure distribution, etc., derived for a horizontal bed, can be used;
- Only one wave condition is considered at a time. No changing wave boundary conditions are applied.
- Only initial rips are studied. No simulations assume fully developed channels.

1.4 Outline

The outline of this report is as follows.
In Chapter 2 account is given of the features of rip currents; appearance and characteristics as can be found in literature and through the present ARGUS observations, are presented. This information is used for verification of the output of the rip currents simulations. Chapter 3 deals with other efforts of modelling rip currents. Some of these are models explaining the generic origin of rips, others are attempts of simulating wave induced flow of a circulation cell. This literature review gives a better understanding of the processes involved and makes the evaluation of the added value of this simulation study possible.
Chapter 4 gives a derivation of the most important equations describing the mutual influence of waves and currents and the driving forces of wave induced flow are discussed. Together this gives an insight in the significance of the various effects and the limitations of the modelling of the physical processes.

The second part of this report treats the simulation efforts itself. Chapter 5 starts with the most important choices and reasoning that constitute the basis of morphodynamic rip current modelling using DELFT2D-MOR. Special attention is paid to the process of obtaining a steady state for the wave current interaction. A reliable outcome of the water movement is conditional for ongoing morphodynamic computations. On the basis of the simulations lies a domain, mapped on a grid with a bathymetry and boundaries, which comes up in Chapter 6.
Chapter 7 and 8 continue with presenting some of the results and sensitivity of various parameters and phenomena involved in hydrodynamic (waves and flow) and morphodynamic rip current modelling. The conclusions drawn at the end of Chapter 8 yield that, although an evolving rip current can be observed in the outcome of the simulations, the development is not as strong as could be expected.
Therefore, in Chapter 9 possible effective adaptations are started to be explored. An important question which will be answered, is whether incorporating the mass flux in the expressions involved has a significant effect on nearshore wave induced flows.
Conclusions drawn from these simulations and recommendations for further research on the generation of rip currents finally follow in Chapter 10.
2 Description of Rip Currents

This Chapter starts with a literature review on rip observations. Because extensive measurements of rip systems are lacking, these reports are an important means for validating the simulations discussed in Chapter 5 and onwards. After naming the constitutive rip elements, the partition in hydrodynamics and morphology is again applied. Besides, ARGUS video images are presented to get more insight in the dynamics of rips. Findings are reported in the second part of this Chapter.

2.1 Review of rip current related literature

As mentioned in the introduction, rip currents have (had) considerable scientific interest. Partly because under certain circumstances these rip currents are expected to dominate exchanges between the surf zone and offshore. But not only because of the morphological impacts rips have been studied, there exist also an interest in the mechanism that is able to generate rip currents.

2.1.1 Description of a rip system

A rip current is an intense, narrow, seaward flow of water from the surf zone. It is much shallower than it is wide. Rip currents are a morphologic phenomenon on a short time scale. Under favourable conditions, the time required for a rip to develop is less than 10 hours. This narrow seaward current disintegrates outside the surf-zone. As the seaward directed jet expands laterally, a mushroom-shaped structure (vortex pair) develops at the head, with counter rotating eddies that may or may not be of equal size. This vortex pair may detach from the stem of the rip current. Substantial amounts of water are moved offshore.

![Diagram of rip current system](image-url)

Figure 2-1 Component parts of a rip current system
• The constitutive elements of a rip-current system are (see Fig. 2-1):
  • the feeder currents, running along the shore at a short distance of the actual rip current
  • the rip neck, in which the water transported by the feeder current merges and is transported seaward
  • the rip head, formed when the rip current loses velocity, expands and vanishes.

A rip current system can be part of a nearshore circulation pattern. In nearshore circulation, the following phenomena can be considered:
  • the mass transport in onshore direction due to breaking waves
  • the wave induced longshore currents passing into the feeder channels
  • the water transported back through the line of breakers by the rip currents

**Rip Current Intensity**

No unambiguous way of characterising rips exists. Shepard et al. (1941) introduced the concept of rip current intensity which appeared to be represented by the flow in the rip neck. In this study the flow velocity in the rip channel crossing the bar, is an often used quantity for comparison purposes.

**2.1.2 Hydrodynamic characteristics of a rip current**

**Waves Height**

Shepard et al (1941) found a strong relationship between wave height and rip current intensity. The largest rip current intensities correspond with the largest waves. They found that if waves were smaller than 0.6 m, then rip currents would disappear.

Short (1985) showed that rips accompany moderate to high waves and are associated with intermediate beach types. Rip spacing follows wave conditions, spacing, size and intensity increase as waves rise.

**Angle of wave incidence**

Waves reaching a beach generate a variety of nearshore currents, the pattern depending in large part on the angle the waves make with the shoreline. When the wave crests are parallel or nearly parallel to the shoreline, according to Komar et al. (1986), the nearshore currents are dominated by a cell circulation with seaward-flowing rip currents.

In Noordwijk (private observations), it was observed that initial rip currents appear with incident waves with small angles relative to the beach normal. When waves incident on a larger angle and generated longshore currents, the rip currents are easily imperceptible.

**Rip Current magnitude and direction**

Sonu (1972) reports that in his observations on a beach in Florida, the circulations were pulsational. Occasional strong outflows at surfbeat frequencies caused water to escape from
the circulation. But also on the tidal time scale differences in velocities were observed, mainly due to the intensity of wave breaking on bars.

He also observed that, although a feeder channel existed close to the shore line, the streamlines (measured with floaters) failed to define a distinct feeder current. Generally a feeder current was more readily distinguished during the time of small wave activity. In a high surf, proportionately more water was fed into the rip directly from its sides than through the feeder channel. Whereas the shoreward currents were of the order of 20 to 30 cm/s the outflow velocities in the rip attained as much as 2 m/s. (with waves of $H_b=0.395$ m, $T_b=5.0$ s).

![Diagram of float-measurement on Oct. 2 (De Graaff 1995), Noordwijk '95 field campaign]

Float measurements of De Graaff (1995) et al. in figure (2-3) show local but present feeder currents.

Cowell et al. (1993) observed, rips determined by lateral boundaries, which extended several hundreds of meters through the surf zone. He does not mention the morphology involved. Smith et al., found rip extension about 150 m and 0.5 m/s as average speed, approaching 0.7 m/s in some events.

Davidson-Arnott and Greenwood (1974) observed rips with speeds of 0.5-0.75 m/s in a barred inshore with 2-3 m high waves and a $T=4-5$ s. While Bowman et al. (1987), watched the beach in Tel Aviv. Maximum rip velocity in rough sea conditions was reported to be in the range of 1-2.5 m/s. Under the prevailing calm wave conditions and the barred bathymetry in the study area, the rip velocity ranged between 25 and 55 cm/s. Pulsations of current where a clear characteristic.

My impression of developing rips during Noordwijk field campaign (1996) was that rip velocities in a beginning rip channel of over 1 m/s were not rare. This is confirmed by observations of De Graaff (1995). During the Noordwijk '95 field campaign with a measurement frame in the rip channel, velocities of over 1.2 m/s were read.

**Tide**

Shepard et al. (1941) mention that rip currents were more pronounced at low tide, but no evident relation of the rips to spring and neap tide was observed.
McKenzie (1958) reports that a variation of the tide frequently has a marked effect on a rip current system. Many other authors do not pay very much attention to the influence of the tide.

2.1.3 Morphological rip current characteristics

**Beach profile**

Rip currents are commonly observed on long, straight beaches lacking the irregular offshore topography necessary to produce wave refraction. In such cases, a number of rip currents generally are observed with a regular spacing (roughly four times the width of the surf zone). There certainly is no abundance of information about relations between the beach slope and the occurrence of rip currents.

**Rip channel**

An important result of the study of Shepard et al. (1941) was the observation of the existence of channels in the path of the rip current. They report that such a channel is cut to a depth of about one meter below its surroundings with the deepest entrenchment near the point where the channel assumes an outward direction.

![Figure 2-3 Profiles over the rip neck, De Graaff (1996) Noordwijk '95 field campaign](image)

The profiles of De Graaff (1995) in Figure 2-3, measured over the crest of the nearshore bar, show a U-shaped channel with depth of 1 m. Personal observations concern strong ripples in the rip neck: irregular waves in the floor with holes of about 20 cm and a spacing of bumps of about 70 cm are observed.

Kroon (1997) describes the relationship of rip-channel deepening and narrowing in the inner nearshore bar from low-energy conditions towards high-energy conditions. Rip-channel widening and shallowing in the inner nearshore bar occurs when energy conditions
subside. This seems to agree with the findings of Sonu (1972) about the feeder channels (see section 2.1.2).

**Sediment transport**

Rip currents can achieve high velocities and act together with the wave action to rearrange the beach sediments. Most commonly, the rip currents transport sand offshore to beyond the breaker zone, hollowing out embayments at the waterline. In those embayments, coarse material (e.g. seashells) accumulates at the waterline (personal observation).

No observations were found in literature which mention the erosion of farther offshore situated bars in the rip current direction or development of a berm in the rip head area.

**Spacing**

Numerous studies of nearshore circulation have been conducted to explain why rip currents often have very regular spacing along the beach, see section 3.2. Rips on the Dutch coast do have a longshore spacing that differ from tens of meters on the beach to hundreds of meters in the nearshore area. The longshore spacing of rip-channel changes with energetic conditions. Kroon (1997) mentions spacings between 350m to about 600m.

**2.1.4 Life cycle, migration and persistence**

Rip channels in the inner nearshore bar are not a permanent feature. They can develop within hours. McKenzie(1958) observed that under moderate swell conditions, small and numerous rips appeared. These currents ran only a short distance across-shore before vanishing. Under heavy swell conditions, fewer but more active rips developed. This is in accordance with own observations at the Noordwijk beach.

Short (1985) observed in Australia that rip migration can result from both shore normal and oblique incident waves. In case of shore normal waves, the growth of rip spacing is associated with rising waves, during which rips may shift laterally. A shift in wave direction to oblique waves induced migration of rips. The migration rates averaged between 10 and 40 m per day. However during high wave events, rips can migrate down drift under oblique waves at rates of meters per hour. Also these statements are confirmed by personal observations in Noordwijk.

**2.2 ARGUS images (Noordwijk Situation)**

Since 1992, monitoring of nearshore morphodynamics is carried out using video images, within the framework of the ARGUS program. Noordwijk at the central Dutch coast is one of the locations were data are collected on an hourly basis. The ten minutes time averaged images show bright, longshore intensity bands, indicating the locations where waves preferably break.
Using ground control points, these time exposures can be converted into rectified images. Under moderate to high wave conditions, the bands indicating wave breaking are clearly visible (see for example Figure 2-6). The curve, bended offshore, in this band indicates the possible presence of a rip current.

Video imaging offers great advantages for monitoring the nearshore zone. But while the video imaging techniques are an indirect way of getting information about the underlying topography some reservations should be kept in mind. In particular the fact that supposed rips are only recognisable under moderate to high energetic conditions. From the images alone it can not be concluded that when they are not visible on an image, they do not exist nor that they do not evolve. Yet, although we have no really continuous monitoring technique it is very much better, than what would be possible without.
Appendix A shows a series of rectified ARGUS images taken in the period of September till December of 1995. About September 29, during the Noordwijk ‘95 field campaign, a pronounced rip evolved. De Graaff (1995) extensively treats the hydrodynamic conditions at that time. From studying series of ARGUS images, taken in a period of months and combining them with wave data, the following conclusions are drawn:

- A rip current shows a an offshore directed ‘bend’ in the longshore band of breaking waves;
- The size of the bulge in the band in cross shore direction is about 30 m;
- The size of the bulge in the band in longshore direction is about 150 m;
  Recognisable) rips are formed under higher energetic wave conditions;

![Figure 2-6 Rectified ARGUS image (Sep. 29 , GMT 10:14 ) with filter for intensity peaks](image)

- Rips are not recognisable in the outer nearshore bar. Intensity peaks in longshore direction in the dissipation patterns are perceptible. It is assumed possible that these peaks indicate the location of initial rips;
- Clear rips develop in case of nearly perpendicular incident waves;
- Some rips develop stronger, while rips in between disappear;
- Rips cutting the inner nearshore bar, usually aren’t prolongings of discharge runnels through the swash bar;
- Initial rips can occur in the period of hours;
- Rips can persist on the same location for months (see Appendix A);
- Rips expand, fade away, before vanishing caused by a strong longshore current.

Aarninkhof (1996) quantified the relation between image intensities and the bathymetry. He related the image intensities to the roller energy density $E_r$ divided by the squared phase speed. Although the exact relation between pixel intensity and $E_r$ (or an other quantity) is not fully understood yet, we do know that it has anyhow something to do with wave dissipation.
Figure 2-7 Cross shore intensity profile from rectified ARGUS image

Figure 2-8 shows two cross shore intensity profiles from the ARGUS images of Sept.29 1995. One is taken through the rip section, while the other is taken in a 'normal' transect which is not disturbed by a rip current. The right hand side peak is the waterline dissipation. The larger peaks indicate the breaking on either the nearshore bar or the rip current.
3 Models for Rips and their generation

In this Chapter theories of rip currents, as found in literature, are discussed. The knowledge of these theories can be used by the design of simulations in the next Chapters. A number of possible mechanisms have been proposed for the generation of rip currents. They can be classified in two categories, including subdivisions:

1. Hydrodynamic mechanisms (on a longshore uniform topography)
   a) Wave-wave interaction
   b) Wave-current interaction
   c) Instability mechanisms

2. Morphology induced mechanisms (longshore non-uniform beach)
   a) Longshore variation in bottom topography
   b) Presence of lateral boundaries

Efforts have been concentrated both in describing the nearshore current field and in determining the spacing between rip currents.

3.1 Hydrodynamic mechanisms on a longshore uniform topography

3.1.1 Wave-wave interaction models

One hypothesis for the mechanism that generates rip currents is that they are caused by longshore variations of the wave height. These induce variations longshore of the radiation stresses that in turn drive rip currents. Bowen and Inman (1969) showed that longshore variations of the breaker height force currents that flow seaward at locations where the wave heights are lowest.
Figure 3-1 a) Radiation stress set-down and set-up across the shoreline; b) Radiation stress gradients along the shore showing the relationship to set-up and set-down, and the water drift patterns (Carter, 1988)

**Intersecting wave trains**

Dalrymple (1975) proposed an alternative mechanism for rip current generation on a plane beach, based on the spatial variations of the mean water level caused by two intersecting wave trains of the same period. He considered two small amplitude waves propagating obliquely to the shoreline in such a way that they intersected. The combination of the two wave trains resulted in periodic longshore variations of the wave height, of the velocity potential, and of the wave set-down offshore the breaker line.

**Synchronous edge waves**

At the shoreline, some energy from incident waves may be transferred to secondary wave motions called edge waves. They differ from normal reflected waves in that they are resonant. Bowen and Inman (1969) found that the non-linear interaction between edge waves and normally incident waves produces a longshore variation in wave height which in turn, may generate rip currents. Edge waves are the trapped modes of longshore wave
motions that can occur in the surfzone and may be progressive, that is, move longshore, or standing, that is stationary. Edge wavelength measured parallel to shore could be responsible for rip spacing. Bowen and Inman presented the rip current spacing as:

$$L_e = L_e = \frac{gT^2}{2\pi} \sin [(2n + 1)\beta]$$

(3.1)

where, \( L_e \) is the rip current spacing, \( L_e \) the edge wave length, \( g \) the gravitational acceleration, \( T \) the incident wave period, \( \beta \) the planar beach slope and \( n \) is the mode of the edge wave, which is equal to the number of zero crossings of the water surface elevations in the offshore direction.

Standing waves are produced in the surfzone when incoming incident and infragravity waves interact with totally or partially reflected waves of a similar period. Gravity edge waves (in leaky modes) and infragravity edge waves (groupiness, surf beat) may also generate periodic longshore wave height variations. Infragravity waves can be generated by several mechanisms, one of which is wave groupiness. We refer to Oltman-Shay (1989).

Bowen and Inman (1969) were able to demonstrate rip-current generation by edge waves in a laboratory wave basin, but their hypothesis remains to be tested on ocean beaches. Dalrymple and Lozano (1978) envision a problem arising from the explanation of rip currents by edge waves. Rip generation by edge waves assumes total reflection from the beach, corresponding to surging waves. This was also shown by Guza, Inman and Bowen (1975, 1976) who found that wave breaking strongly damp out edge wave motion. Therefore the edge wave model for rip currents is apparently restricted to steep beaches and surging incident waves.

Oh (1994) puts forward another objection to the edge wave theory: “As given by equation (3.1), it is always possible to select a combination of wave period, edge wave mode numbers and beach slope, which can nearly match the observed spacing.”

### 3.1.2 Wave-current interaction models for rip currents

Following Bowen’s work, many theories of current generation in the littoral zone area have completely uncoupled the waves from the currents that they generate. This means that neither the wave number nor the energy density is affected by the currents: there is neither refraction nor energy exchange. The interaction considered has in those early ’70’s models been all one way, from waves to currents, through the divergence of wave radiation stresses in the surf zone.

LeBlond & Tang (1974) added the coupling between waves and current. But they conclude that the coupling does not modify the flow field appreciably, although it changes the wave energy distribution across the surf zone. They also found that for the same flow field the longshore varying energy density at the breaker line (which causes the currents) is now increased: the energy interaction resists the flow, and larger wave perturbations are needed to induce rip currents. LeBlond & Tang (1974) neglected lateral friction in their model but they indicate that it could play a significant role in the dynamics of rip currents.

Dalrymple and Lozano (1978) developed two analytical models which extend the work of LeBlond & Tang. One with the effect of the currents on the incident wavelength, and a second model that considers the refraction of the waves as well.
In the first model no rip currents appear. In the second model an additional wave-current interaction effect, the refraction of the waves on the current, is also included. Steady longshore periodic nearshore circulation cells are generated. The refraction of the waves by the outgoing rip currents causes them to impinge on the beach obliquely, thus generating longshore currents flowing towards the location of the rip. Though these models account for wave-current interaction by using a local (current affected) dispersion relation, they do not reckon with this interaction in the formulation of the wave dissipation expressions. Only a relation between wave height and water depth is used.

3.1.3 Instability mechanisms

The third subcategory of models, assuming longshore uniform bathymetry, hypothesises that the state of longshore uniform wave set-up could be unstable under certain conditions and that the instability manifests itself in terms of periodic nearshore circulation cells. These models mainly predict the spacing between the rips by solving eigenvalue problems. They do not predict the strength of the rip currents. In a sense, this category overlaps for instance with the wave-wave interaction models since interacting waves can cause longshore perturbations and resulting instabilities.

Something similar is the theory that oblique incoming waves, which generate a longshore flow in the inner trough, will contain meandering patterns, due to shear instabilities. Sonu (1972) observed wave induced nearshore circulation and meandering currents on a beach with smooth offshore topography and surf zone undulations under essentially uniform breaking wave heights. He found that the rip current patterns followed the same spacing as the ‘meandering’ wavelength.

Hino (1974) proposed a rip current generation model based on stability analysis of a initially uniform plane beach but allowing a feedback between the movable bed and the flow field. He found the most preferred spacing of rip currents of about four times the surf zone width.

This category of theory is still subject of further investigation.

3.2 Longshore non-uniformities in morphology

The second category of mechanisms have in common that rip currents are driven by longshore variations in radiation stresses. Bowen & Inman (1969) have shown that the cell circulation is produced by longshore variations in wave breaker heights, which in turn produce longshore variations in the wave set-up. The set-up will raise the water in the nearshore zone to higher levels shoreward from positions of large breakers than shoreward of smaller breakers. As a result will converge in the area’s of low set-up and turn seaward as a rip current.

3.2.1 Bottom topography

Concentration of streamlines due to sloping beach

Arthur (1962) obtained a model which showed that an increasing depth in the flow direction would tend to concentrate the streamlines and strengthen the flow. He used the steady state
shallow water continuity and momentum equations, in which the pressure gradient term was the only remaining driving force term. Arthur noted that the friction would tend to broaden streamlines of the rip current. Although this model is not a rip generic models, it can explain how rips subsist due to the bottom topography.

Bars

On a multiple bar coast, like many parts of the central Dutch coast, bars usually begin as a shoreline attachment and gradually, looking in longshore direction, the distance to the shore increases. This leads to diversified submarine topography although the entire picture shows relatively straight shore lines. Waves converge at the heads of bars and diverge at troughs extending through the breaker zone. This results in longshore gradients in radiation stress and consequently in nearshore circulation systems. Longshore currents adjacent to the shore diverge from areas of wave convergence and flow seaward as rip currents at areas of wave divergence. Because the slanting bars have considerable length this only explains large rip spacing.

Deigaard (1990)

Deigaard (1990) developed an analytical model for the hydrodynamics of rip currents. He assumed the longshore variation of the set-up to be caused by the fact that the waves are breaking on the bar and not at the locations of the rip channels. The only breaking in the rip

![Diagram of a coast with a bar and rip current system from Deigaard's model (Deigaard 1990)]

Figure 3-2 Definition sketch for a coast with a bar and rip current system from Deigaard's model (Deigaard 1990)

transects is at the shoreline. He also assumed the transport of water, as a kind of mass flux, over the crest of the nearshore bar to be governed by an expansion loss-type law. From a set of three equations (continuity, momentum equation for the mean flow in the trough disregarding bed friction, and discharge over the bar) the variation of the mean longshore current and the set-up along the bar can be obtained. The rip discharge follows from continuity considerations.
3.2.2 Presence of lateral boundary

The presence of lateral boundaries (e.g. breakwaters) introduces a longshore non-uniformity that can drive rip currents. Points, breakwaters and piers all influence the circulation pattern and alter the direction of the currents flowing along the shore. On the down current side nearby obstructions, a pronounced rip often extends seawards.

Wind and Vreugdenhil (1986) studied rip current generation near structures. Computational modelling, which they used as one of the first, makes it possible to keep all the terms in the equations of motion for the mathematical analysis. Wind and Vreugdenhil concluded that the rip current is dominated by convection. When the convective terms where disregarded in the momentum equations, the rip vanished. The bottom friction determined the order of magnitude of the velocity of the rip current. In accordance with Arthur (1962) they found that the bottom topography plays a role in the convergence and divergence of the streamlines. For a given forcing, the total circulating flow rate was regulated by the bottom friction rather than the magnitude of the viscosity for the lateral shear stresses.
4 Wave induced flow

Nearshore circulation is an important cause of morphological changes. Nearshore circulation refers to the current patterns generated by waves, wind and tidal motion. In this study, only wave induced flow is considered. A rip current system is an example of a wave induced flow.

Waves influence the time mean condition of the fluid mass in which they propagate. They exert a force on it. Such a force can have a net time-averaged non-zero resultant which causes a net momentum flux. Yet, to balance the resulting force, a pressure gradient or a flow velocity will arise. It leads to variations in the local mean water depth (set down and set up) and currents in the nearshore zone.

But at the same time the current influences the waves. A current in the nearshore zone opposing the waves, like in case of a rip current, yield an increased wave height and a reduced wave length so that the wave is steepened, even up to the point of breaking.

Describing the mutual interaction of waves and current correctly is complicated. At least an averaging approximation is necessary to model a wave induced flow. It is necessary to decide which part of the total motion can be considered the current part and which part is the wave motion as such.

A simplistic approach is to describe the current as separated from the waves artificially. The problem, however, is that the two parts of motion cannot be separated because the order of magnitude of the terms is the same. A way out here is to define the mean motion by time-averaging over a wave period. The averaged equations have been given by Phillips (1977) and Dingemans (1997).

Starting-point, in section 4.1, are the continuity equation and the momentum equations for incompressible flow. These are averaged over the short wave period and the water depth. (For a derivation see appendix B)

In modelling nearshore hydrodynamics the energy of the waves can be described in an energy equation (section 4.2.3), making use of linear wave theory. The solution of the energy equation will supply information on the variation of the wave height $H$. $H$ is the main input parameter for the equations which describe the radiation stress forcing ($S_{ij}$). Whereas the solution of the continuity and momentum equations will provide information about water level variations ($\zeta$) and currents ($u_i$) induced by this forcing. Therefore, the prediction of the wave height by an energy consideration, in particular inside the surf-zone, is important for successful modelling of all the wave-generated nearshore phenomena.

For a definition sketch see appendix B.

The equations derived in this chapter are averaged over depth. Still, a schematisation can be made for the nature of the movement of the water particles. So, a refinement is introduced in the description of the averaged movement. This is worked out for the wave mass fluxes in section 4.4.
4.1 Averaged Equations for free surface flow

4.1.1 Averaging

Assuming that the vertical velocity profile in the nearshore zone is quite uniform, the flow can be modelled with the depth-averaged Reynolds equations for unsteady flow assuming a hydrostatic pressure distribution over the vertical. Therefore a mean horizontal velocity \( U_i(x,t) \), defined by (Dingemans 1997), is introduced:

\[
U_i(x,t) = \frac{1}{h} \left( \int_{-d}^{\zeta} u_i(x,z,t) \, dz \right)
\] (4.1)

The time span over which is averaged is the wave period. The horizontal velocity components \( u_i(x,z,t) \) can be written as separated in a mean velocity and an a residual velocity part:

\[
u_i(x,z,t) = U_i(x,t) + u_i(x,z,t)
\] (4.2)

In definition of the horizontal mean velocity, \( U_i \) is in fact drawn up out of an ambient current part plus the effect of the mass flux due to the wave motion alone. It follows from this definition that:

\[
\left( \int_{-d}^{\zeta} \bar{u}_i(x,z,t) \, dz \right) = 0
\] (4.3)

but the means in time of the residual velocities, without integration over the instantaneous depth are not equal to zero.

4.1.2 Mean momentum

In its simplest form the mean motion \( U(x,t) \) is described by the mean momentum equation (appendix B)

\[
\rho h \frac{\partial U_i}{\partial t} + \rho h U_j \frac{\partial U_i}{\partial x_j} + \rho h g \frac{\partial \zeta}{\partial x_i} + \frac{\partial S_{ij}}{\partial x_j} + \langle \tau_i \rangle - \rho h u_H \frac{\partial^2 U_i}{\partial x_i^2} = 0
\] (4.4)

where:

- \( p \) = pressure
- \( S_{ij} \) = radiation stress
- \( \tau \) = shear stress
- \( h \) = water depth (including set-up)
- \( u_H \) = horizontal turbulent viscosity

The various terms in (4.4) represent (respectively):

1. the temporal acceleration
2. the convective accelerations
3. the pressure gradients
4. the interaction between mean flow and the short waves
5. the interaction between mean flow and the bottom
6. the interaction between mean flow and the turbulence

The calculation of a flow field, using this equation is adequate for currents which can be considered to be independent of depth. Dingemans questions whether this is a valid assumption for wave induced currents. It is common to consider the ambient current field to be independent of the depth and the influence of the current on the wave properties is taken into account by using the Doppler shift.

The radiation stress tensor represents the excess momentum fluxes due to the residual velocities \( \tilde{u}_j \) and yields, following from averaging of the momentum equation:

\[
S_{ij} = \left\langle \int_0^\xi \left( \rho \tilde{u}_i \tilde{u}_j + p \delta_{ij} \right) dz \right\rangle - \frac{1}{2} \rho gh^2 \delta_{ij} \tag{4.5}
\]

where \( \delta \) is the unit tensor (\( \delta_{ii} = 1 \) if \( j=i \), and vanishes otherwise). This flux of momentum is put together by contributions of:

1. the total flow of momentum due to the wave-generated motion
2. the force per unit length exerted by the dynamic pressure (caused by the presence of waves) on a vertical element of length \( dz \), obtained by subtracting the force per unit length exerted by the hydrostatic pressure (in the absence of waves) from the total pressure;

4.1.3 Averaged equation of mass conservation

The averaged equation of mass conservation can be written as (see app B 2):

\[
\frac{\partial (\xi)}{\partial t} + \frac{\partial}{\partial x_i} [hU_i] = 0 \tag{4.6}
\]

where \( hU_i \) is the total mean volume flux through a vertical section.

4.2 Waves

Firstly, in this section, the equations governing the wave motion will be given. After that these are used as the wave momentum contribution in expressions for the driving forces of the mean flow. In section 4.2.1 a description is given of short waves transformed by a current. Next the energy equation for the fluctuating and mean motion is mentioned. These equations can be written as reported in section 4.2.3 as an action balance. In section 4.2.4 is shown that it is convenient to write the energy balance as an action balance because it compromises the wave current interaction. Of course, in a rip current system there is no conservation of energy, so dissipation expressions are needed as well.
4.2.1 Description of short waves on a current

The surface elevation for a periodic, uni-directional, small amplitude wave (measured in a frame of reference moving with the current) is expressed as:

\[ \zeta(x,t) = a \cos(\sigma t - k \cdot x + \phi) \]  

(4.7)

where \( \zeta \) represents the surface elevation, \( a \) is the wave amplitude, \( \sigma \) is the relative frequency, \( k \) is the wave number vector and \( \phi \) an initial phase.

For constant water depth and a uniform and homogeneous current field with velocity \( U \), the wave number \( k \), the relative frequency \( \sigma \) (measured in a frame moving with the current) and the absolute frequency \( \omega \) (measured in a frame fixed to the bottom) are in the linear approach interrelated through the relation:

\[ \omega = \sigma + k \cdot U \]  

(4.8)

This 'Doppler' relation assumes a uniform velocity profile. The relative (or intrinsic) frequency is given by the dispersion relation:

\[ \sigma^2 = gk \tanh kh \]  

(4.9)

The phase velocity \( c \) of the wave relative to the current is:

\[ c = \frac{\sigma}{k} \]  

(4.10)

and the group velocity or propagation velocity of wave energy \( c_g \) (relative to the current) is given by:

\[ c_g = \frac{\partial \sigma}{\partial k} = \frac{1}{2} \left[ 1 + \frac{2kh}{\sinh 2kh} \right] c \]  

(4.11)

Nearshore wave propagation is characterised primarily by depth-induced shoaling, refraction and breaking. Effects of wave-induced currents usually are of secondary importance. But in case of a rip current systems those effects are exactly significant.

4.2.2 Wave energy and flux

Characteristic for waves is that they contain energy and can transfer energy during wave propagation.

The total energy contained in a wave consist of

1. potential energy resulting from the water surface elevations \( E_{\text{pot}} \);
2. kinetic energy resulting from the fluid particle motions \( E_{\text{kin}} \).

The total energy and the energy transfer determine the way waves change during propagation.

The total wave energy \( E \) per unit area is given by:

\[ E = E_{\text{pot}} + E_{\text{kin}} = \frac{1}{8} \rho g H^2 \]  

(4.12)
Small amplitude waves transfer energy in horizontal direction during wave propagation. To be able to propagate itself, a wave must transfer energy to the fluid in front of the wave. This fluid may be in rest but can have a velocity as well. The rate at which energy is transferred by the waves is the wave energy flux. The energy flux of the residual motion in the direction of wave propagation is the work done per unit time by the dynamic pressure force during a wave period and can be written as (Mei (1983)):

$$\bar{F} = \left( \int_{-d}^{z} \frac{c_s}{u} \, dz \right) = c_s \bar{E}$$  \hspace{1cm} (4.13)

where $\bar{E}$ is the energy density of the residual motion (see next section).

### 4.2.3 Energy equation

The energy equation for the combined wave and current motion is needed in wave-averaged models to determine the wave height variations. In the simplest situation of one-dimensional wave propagation without ambient currents, the energy balance equation becomes (Dingemans 1997)

$$\frac{\partial}{\partial x} \left( c_s E \right) + D = 0$$  \hspace{1cm} (4.14)

where $D$ is the dissipated power per unit horizontal area. $D$ is composed of the dissipation due to wave breaking and due to friction. But in case of a rip current this is no effective description, since the mutual interaction between waves and current must be taken into account. Therefore a balance equation for the energy is derived from the scalar product of the momentum equation with the velocity vector. The average energy $E$ can be split in a contribution due to the residual motion, $\bar{E}$, alone and the effect of the mean motion, similar to the velocity separation. Dingemans (1997) gives a derivation of a time averaged and depth integrated energy balance:

$$\frac{\partial \bar{E}}{\partial t} + \frac{\partial}{\partial x_i} \left[ \bar{F}_i + U_i \bar{E} + U_j S_{ij} \right] - U_i \frac{\partial S_{ij}}{\partial x_j} = -\varepsilon$$  \hspace{1cm} (4.15)

where $\varepsilon$ is the rate of energy dissipation per unit area.

This equation describes the exchange of energy between waves and currents. Physically it means that work done by the radiation stress to the current tends to reduce the wave energy (if both are working in the same direction).

### 4.2.4 Wave action

In stead of using an energy balance, like in the previous subsection, an action balance can be used. The advantage of using the action balance is that in the presence of currents, energy density is not conserved but action density is. The action balance reads:

$$\frac{\partial}{\partial t} \left( \frac{E}{\sigma} \right) + \frac{\partial}{\partial x_i} \left( \frac{E}{\sigma} \left[ U_i + (c_{sj}) \right] \right) = -\frac{\varepsilon}{\sigma}$$  \hspace{1cm} (4.16)
Sobey (1988) and Mei (1983) state that the wave energy equation (4.15) is in fact equivalent to the wave action equation (4.16).

Since in this study of rip currents, only steady state cases for both waves and currents are considered, the conservation of wave action reduces to

$$\frac{\partial}{\partial x} \left[ \frac{E}{\sigma} \left[ U_i + (c_e)_i \right] \right] = -\frac{E}{\sigma}$$  \hspace{1cm} (4.17)

Thus, by solving (4.17) the wave height can be determined everywhere provided that the transfer of energy in source term by wave dissipation and the current is known.

### 4.3 Wave-current interaction in a rip

An essential aspect of the phenomenon of rip currents is the ensemble of waves and currents. The current will influence some of the wave characteristics, and, in their turn, the waves will drive the current. This mutual influence is subject of this section. This section concludes with balancing forces which limit the growth of the wave induced flow.

#### 4.3.1 Wave transformation

Clearly, in case of rip currents, the current is in general in opposite direction to the wave propagation direction. Any way, this will lead to a local accumulation of energy.

The wave height will increase and the wave length will decrease. The relative wave frequency will change because the wave celerity will change.

In case of flow opposing the waves several interrelated effects can be illustrated by the wave action equation:

1. decreasing U, in an opposing current U is negative if the wave direction is positive and consequently a local increase in energy (Eq. 4.17).
2. decreasing wave length, increasing wave number \(k = 2\pi/L\)
3. small increasing relative frequency given the dispersion relation (Eq. 4.9)
4. decreasing phase velocity because the increase of the wave number counts heavier than the increase of relative frequency (Eq. 4.10).
5. decreasing wave group velocity, thus the energy density will increase in order to keep the equation in balance (Eq. 4.11 and 4.17)
6. increasing absolute frequency because of increase in \(\sigma\) and \(k\) (Eq. 4.8 and 4.9)

All together this results in an increase of the energy density per unit area.

### Wave refraction

Still there is an other effect which does increase the energy density in the rip, namely current refraction. Current refraction accumulates energy in the rip because the wave rays concentrate on the rip current. But because the wave energy equations used originate from wave ray theory. This effect is not grasped in the equations in this section, in an horizontal 2D space it will result in an energy flux in the direction of the rip current. Anticipating the outcome of the simulations of Chapter 7, it can be remarked here already that this effect has a very limited influence in case of the current velocities involved here.
Wave breaking

An increase in Energy will lead to increasing wave height and wave steepness, and hence there will be often more dissipation.

It is not possible to separate the different origins for dissipation. For instance, both shoaling due to bottom slopes and wave current interaction enlarge the wave number and that resulting in increased wave steepness which is the main factor in the rate of dissipation.

The upper limit for the wave height is restricted due to the maximum wave steepness \( (H/\lambda) \) or due to a maximum wave height/water depth ratio \((H/h)\).

From theoretical considerations the limiting steepness is found to be (van der Velden (1995):

\[
\left[ \frac{H^2}{\lambda} \right]_{\text{max}} = \frac{1}{4} \tanh \left[ \frac{2\pi h}{\lambda} \right]
\] (4.18)

Due to the opposing current, which causes increasing wave steepness, waves could be breaking on a location more offshore compared with the situation in absence of a current, due to a strong energy flux offshore in the rip. But it could also be that the location of breaking does not shift but only the intensity of breaking increases at a location.

4.3.2 Forcing of the current

The wave induced flow can be determined from the averaged momentum equation (#4.4). In fact this is the force balance between the gradient of the radiation stress, the shear stresses and convective accelerations. For the computation of the wave-driven current field, additional equations are needed to describe these forces involved. Expressions are given in this section. Distinction can be made between the driving force (first subsection) and balancing forces. A steady state for the hydrodynamics is assumed, so all the derivatives with respect to time can be cancelled. Further it is assumed that waves propagate perpendicular to the shoreline.

Driving force: radiation stress

The radiation stress is in fact the wave-induced contribution of horizontal momentum to the mean balance of momentum. Because a rate of transfer of momentum or, in other words, an excess momentum flux is equivalent to a force \((\text{d}(mv) = Fdt)\), the change in radiation stress is the wave-induced contribution of the force to the water through which the waves propagate.

For the computation of the wave-driven current field, driving forces have to be determined from the wave description. This can be achieved in essentially two ways:

1. The wave dissipation is used directly for the determination of the driving forces, viz.:

\[
F_i = \left( \frac{D}{\sigma} \right) k_i
\] (4.19)
2. The effect of the waves on the mean current, given by the divergence of the radiation stress, is used to determine the driving force

$$F_i = -\frac{\partial S_{ij}}{\partial x_j}$$  \hspace{1cm} (4.20)

Conventional expressions for the radiation stress tensor, like:

$$S_{xx} = \left( \frac{1}{2} + \frac{2kh}{\sinh 2kh} \right) E \hspace{1cm} \text{(with} \hspace{1cm} E = \frac{1}{8} \rho g H^2 \text{)}$$

$$S_{yy} = \left( \frac{kh}{\sinh 2kh} \right) E$$

$$S_{xy} = 0$$  \hspace{1cm} (4.21)

are in fact not practicable because they pass over the influence of the current on $S_{ij}$, which is inaccurate in this case, as the ratio $c_i/c$ can not be supposed constant. More correct is making use of the wave potential in the linear theory of time harmonic waves and invoking the geometric optics approximation. Dingemans, Radder and De Vriend (1987) found for the radiation stress tensor:

$$S_{ij} = \frac{E}{2} \left[ \frac{k_i k_j}{k^2} \frac{2c_s}{c} + \left( \frac{2c_s}{c} - 1 \right) \delta_{ij} \right]$$  \hspace{1cm} (4.22)

Numerical differentiation of this tensor, with higher order spatial derivatives, can lead to poor results. Therefore they suggest using the first approach for calculating the driving forces. Yet, this means neglecting the contribution of waves, which do not contribute to the local value of the dissipation but only steepen on the ambient current field. And as indicated by Dingemans (1997), this is a deficiency when modelling circulations due differences in set-up and notably set-down. With (4.19) it is impossible to describe set-down, though this occurs in reality. The accumulation of energy where the current is opposing the waves results in a positive gradient of the radiation stress and consequently set down (if a irrotational motion is supposed). This in turn results in an even stronger driving force and higher velocities in the rip.

**balancing force in cross shore direction: Wave set up**

**uniform beach**

On a longshore uniform beach, where no horizontal currents are generated, the bed shear stress is small for the case of zero net cross shore flow (Zyserman 1989). Therefore it can easily be seen that a gradient in the cross shore flux of cross shore momentum results in a slope of the mean water surface:

$$\rho g \frac{\partial \bar{v}}{\partial x} + \frac{\partial S_{xx}}{\partial x} = 0$$  \hspace{1cm} (4.23)

where $x$ is the direction perpendicular to the shore. This is a simplistic reproduction of equation 4.4 for a one dimensional case, neglecting time dependence and shearstresses. In
the surf zone, where the waves break and decrease in height, the gradient of $S_{xx}$ is negative. To balance this the mean water level must experience an elevation over the S.W.L.

**beach with rip channel**

In the situation with a rip channel, the wave breaking is not uniform longshore. The relative deeper waters of the rip channel can cause the incoming waves not to break near the bar but closer to the shore line. The non-uniformity in the conditions of wave breaking longshore has as result that the values of the set-up are also variable longshore. The wave set-up is larger behind the bar and smaller at the rip. The resulting pressure gradient accelerates the water towards the rip. In the previous section is described how an increase in flow intensity causes additional breaking. The resulting effect, and the relative importance of both causes of wave dissipation (depth and/or opposing current), should follow from the simulations.

![Figure 4-1 Set up on normal and rip transect](image1)

![Figure 4-2 Bottom elevation for normal and rip transect](image2)
**bottom friction**

Bottom shear stress is modelled using:

\[
\tau = \frac{1}{2} f_w \rho U_i \bar{u}_b
\]  
(4.24)

where \(\bar{u}_b\) is the velocity at the bottom and \(U\) the absolute velocity. Because of the square relationship with the velocity, the value of the friction factor \(f_w\) has a significant effect. Differences in set-up between the normal and rip channel transect can only be maintained by addition of the shear stress terms to the momentum equation.

\(f_w\) can be a constant (chezy parametrisation) or a function of the water depth (White-Colebrook)

**convection**

\[
\rho h U_j \frac{\partial U_i}{\partial x_j} = -\rho g \frac{\partial \bar{z}}{\partial x_i} - \frac{\partial S_{ii}}{\partial x_j} \langle \tau_i \rangle + \rho h v_s \frac{\partial^2 U_i}{\partial x_i^2}
\]  
(4.25)

For a steady state situation, only the convective terms of the acceleration are present. These terms express the changes in the velocity of the steady, depth averaged current as a function of the position. Any change of the velocity of the flow result also in a change of its momentum. Since any change in momentum requires a force to produce it, the convective terms must be taken into account in the equations for the force balances.

**viscosity**

\[
\rho h U_j \frac{\partial U_i}{\partial x_j} + \rho h g \frac{\partial \bar{z}}{\partial x_i} + \frac{\partial S_{ii}}{\partial x_j} \langle \tau_i \rangle = \rho h v_s \frac{\partial^2 U_i}{\partial x_i^2}
\]  
(4.26)

Part of the momentum is consumed by the turbulent motion. The viscosity terms cause a smoothing out of small scale gradients in the flow. A turbulent closure model is required for solving these terms. Zyserman (1989) reports that the contribution of the turbulent velocities to the total cross-shore flux of cross shore moment was small compared to that due to the periodic wave motion. For the width and extension of the rip current in the nearshore zone it is of more importance.

**4.4 Addition of wave mass flux**

In the preceding sections it was shown that waves transport momentum with a certain speed. The resulting flux of momentum is represented in the radiation stress terms in the momentum equations. All in all, besides momentum, waves do transport mass as well. This mass flux by the wave motion is subject of this section. In fact introduction of wave mass fluxes in the description of the water motion is a refinement in the description of the depth averaged water motion.
The approach of wave mass fluxes assumes a separation between waves and flow. The description of this separation is treated in the first part of this section. Next is a derivation of expressions for the wave mass flux itself. Section 4.4.3 describes the way the equations of mean motion must be adapted. The treatment of the energy equation can be found in appendix E. In Chapter 9 the effects of the addition of wave mass fluxes on the simulations of rip currents are discussed.

4.4.1 Wave and current separation

Assuming that the motion of the fluid can be described by an ambient flow with superimposed wave motion according to

\[ u_i = u_{\text{flow},i} + u_{\text{wave},i} \]  \hspace{1cm} (4.27)

leads, when integrating over the wave period, to

\[ \langle u_i \rangle = \frac{1}{T} \int_0^T u_i dt = \frac{1}{T} \int_0^T (u_{f,i} + u_{w,i}) dt = u_{f,i} + \langle u_{w,i} \rangle \]  \hspace{1cm} (4.28)

Subsequent integrating over the depth gives

\[ \hat{u}_i = \frac{1}{h} \int_{-d}^{d} u_i dz = \frac{1}{h} \int_{-d}^{d} (u_{f,i} + u_{w,i}) dz = U_i + \bar{u}_{w,i} \]  \hspace{1cm} (4.29)

In words, the mean total local, or equivalent, velocity is the sum of the depth averaged velocity from the ambient flow and the local mean wave contribution to the mean velocity. See also Figure 4-1 for a graphical explanation.

![Velocity profiles](image)

Figure 4-1 Velocity profiles

The mass flux per unit width of the mean flow is

\[ M_i = \rho U_i h \]  \hspace{1cm} (4.30)

while the mass transport of the wave motion is
\[
\vec{M}_i = \rho h \vec{u}_{w,d} \tag{4.31}
\]

Here \(\vec{M}_i\) is the mass flux of the wave motion so that \(\vec{u}_i\) is the mean equivalent velocity of a vertical column of fluid. The sum of the two, in (4.30) and (4.31), gives the total mass flux \(m\).

From linear (Airy) wave theory follows for the horizontal velocity of a particle due to waves alone:

\[
u_{\text{waves},j} = \frac{\pi H}{T} \frac{\cosh(kz)}{\sinh(kd)} \cos(\omega t - kx_j) \tag{4.32}
\]

Linear wave theory may still be applied with respect to a co-ordinate system moving with the current velocity \(U\). Simply applying the integration over the short wave period mentioned above would give a zero mean velocity. Therefore the Lagrangian analysis of the wave drift is used to approximate the drift velocity. A derivation is given in section 9.2

### 4.4.2 Mass flux

Propagating surface waves cause a motion of water particles in horizontal and vertical direction. The net vertical displacement is zero. It is a symmetrical movement around an equilibrium. The net horizontal displacement for a fluid particle is not equal to zero. This results in a mass flux; the mean of the integrated velocity of all fluid particles in a vertical averaged over a wave period.

In the next section an expression for the mass flux is derived. Fredsoe and Deigaard (1992) give a derivation as well. This derivation is summarised first. (The wave drift velocity can be evaluated by Lagrangian considerations or an Eulerian approach. Both methods give the same result.)

For two reasons the wave induced drift velocity \(<u_w>\) is always positive relative to the wave propagation direction:

1. A fluid particle will stay longer below the wave crest than below the wave trough, because
   a) the fluid velocity is positive below the crest and negative below the trough;
   b) both trough and crest propagate at the same speed \(c\).
2. The particle path is elliptic in shape, with the particle travelling forward at the upper part of the orbit and backward at the bottom of the orbit. The horizontal orbit velocity increases with the height above the bed. At the top of the orbit, the velocity is slightly higher than at the bottom, resulting in a small positive contribution to the drift.

**Lagrangian analysis of the wave drift**

By definition the Lagrangian drift cannot be detected by taking measurements at a fixed point, one has to follow the water particle during its orbital motion.

In case of wave propagation in x direction \(v_w = 0\), consider the points P and Q in Figure 9.2, where P is a point on the orbit of a particle, the mean position of which is Q.
Figure 4-2 Drift velocity as seen from a Lagrangian point of view

The difference between the instantaneous velocity at P and at Q is approximated by Fredsoe et al. (1992):

\[ \Delta u = \frac{\partial u_{\text{waves}}}{\partial x} \Delta x + \frac{\partial u_{\text{waves}}}{\partial z} \Delta z \]  

(4.33)

where \( \Delta x \) and \( \Delta z \) are the horizontal and vertical displacements of P from Q. \( \Delta x \) and \( \Delta z \) are given by

\[ \Delta x = \int u_{\text{waves}} \, dt \]  

\[ \Delta z = \int w_{\text{waves}} \, dt \]  

(4.34)

from which follows at Q

\[ \bar{u}_w = \frac{\partial u_{\text{waves}}}{\partial x} \int u_{\text{waves}} \, dt + \frac{\partial u_{\text{waves}}}{\partial z} \int w \, dt \]  

(4.35)

The right hand side terms in (4.35) can be evaluated by applying the linear (Airy) wave theory expressions for the orbital velocity

\[ u_{\text{waves}} = \frac{\pi H}{T} \frac{\cosh(kz)}{\sinh(kh)} \cos(\omega t - kx) \]  

(4.36)

\[ w_{\text{waves}} = -\frac{\pi H}{T} \frac{\sinh(kz)}{\sinh(kh)} \sin(\omega t - kx) \]  

(4.37)

where:

- \( H \) is the wave height (from trough to crest),
- \( k \) the wave number,
- \( \omega \) the cyclic frequency
- and \( c \) the phase speed,
- \( k \) and \( \omega \) being defined as

\[ k = \frac{2\pi}{L}, \quad \omega = \frac{2\pi}{T}, \quad c = \frac{\omega}{k} \]  

(4.38)

finally resulting in:
\[ \bar{u}_{w,j} = \frac{1}{2c} \left( \frac{\pi H}{T} \right)^2 \left( \frac{\cosh^2(kz) + \sinh^2(kz)}{\sinh^2(kD)} \right) \] (4.39)

It can be seen that the mean drift velocity is a non-zero, second order quantity.

**Mass flux**

Propagating surface waves thus cause a net motion of water particles in horizontal direction. This mass flux associated with the wave drift velocity is found by integration of Eq. (4.39) over the water depth:

\[
\bar{M}_j = \int_{-d}^{0} \rho \bar{u}_j dz
\]
\[
= \rho \int_{-d}^{0} \frac{1}{2c} \left( \frac{\pi H}{T} \right)^2 \left( \frac{\cosh^2(kz) + \sinh^2(kz)}{\sinh^2(kh)} \right) dz
\]
\[
= \frac{\rho}{2c} \left( \frac{\pi H}{T} \right)^2 \frac{1}{\sinh^2(kh)} \frac{\sinh^2(kh)}{2k}
\]
\[
= \frac{\rho \pi H^2}{4T} \frac{1}{\tanh^2(kh)} = \frac{\rho g H^2}{4T \omega^2} = \frac{\rho g H^2}{8c_i} \]
\[
= \frac{E}{c_i}
\] (4.40)

using the dispersion relation (4.9) and where

\[
c = \frac{\omega}{k} \]
(4.41)

In section 9.4 results of simulations with mass flux included are discussed. Beforehand it can not be said whether the mass flux is nullified by a return flow in the vertical or an extra propulsion of the horizontal circulation currents.

**4.4.3 Equations with mass flux formulation**

The equations of mean motion (mass and momentum) can be adapted on the basis of the wave-current separation in the preceding section. Slightly different expressions are derived.

**Mass conservation equation**

The expression for the mass conservation balance reads (see appendix B.2):

\[
\rho \frac{\partial}{\partial x_i} \int_{-d}^{\xi} u_i dz + \rho \frac{\partial \xi}{\partial t} = 0
\]
(4.42)
To bring the equation 4.42 into a more useful form, the total horizontal velocity \( u_i \) is separated into a mean flow and a wave motion component according to Eqn. (4.29), both depth and wave-period averaged

\[
\hat{u}_i = U_i + \bar{u}_{i,j}
\]  

(4.43)

This gives for the conservation of total mass per unit area, expressed in fluxes

\[
\rho \frac{\partial h}{\partial t} + \frac{\partial}{\partial x_i} \left\{ M_i + \bar{M}_i \right\} = 0
\]

(4.44)

**Momentum equations**

The momentum equation for horizontal flow, as derived in appendix B.3 reads:

\[
\rho \int_{-d}^{\xi} \frac{\partial u_i}{\partial t} dz + \frac{\partial}{\partial x_j} \int_{-d}^{\xi} \left( \rho u_i u_j + p \delta_{ij} \right) dz + \frac{\partial h}{\partial x_i} = 0
\]

(4.45)

Suppose \( \hat{u}_i = U_i + \bar{u}_{i,j} \), where \( \hat{u}_i \) is again the total mean velocity. The mean value of the first integral is again \( m_i = M_i + \bar{M}_i \), which is interpreted here as the total horizontal momentum per unit area, i.e. the sum of the current momentum and the wave momentum. Then even so the second integral of Eq 4.45 averaged in time yields:

\[
\int_{-d}^{\xi} \left( \frac{u_i u_j + p \delta_{ij}}{\rho} \right) dz = \int_{-d}^{\xi} \left( [U_i + \bar{u}_{i,j}] [U_j + \bar{u}_{i,j}] \right) dz + \int_{-d}^{\xi} \frac{p \delta_{ij}}{\rho} dz
\]

\[
= \int_{-d}^{\xi} \left[ U_i U_j + U_i \bar{u}_{i,j} + U_j \bar{u}_{i,j} + \bar{u}_{i,j} \bar{u}_{i,j} \right] dz + \int_{-d}^{\xi} \frac{p \delta_{ij}}{\rho} dz
\]

\[
= U_i U_j (d + h) + U_i \bar{M}_i + U_j \bar{M}_j + \int_{-d}^{\xi} \left( \bar{u}_{i,j} \bar{u}_{i,j} + \frac{p \delta_{ij}}{\rho} \right) dz
\]

(4.46)

This can be expressed conveniently in terms of the equivalent momentum flux of a steady stream having the same mass flux \( m_i = M_i + \bar{M}_i \) and the equivalent mean transport velocity

\[
\hat{u}_i = \frac{m_i}{\rho h} = \frac{M_i + \bar{M}_i}{\rho h} = U_i + \bar{M}_i
\]

(4.47)

as the actual mean flow \( U_i \) with an excess momentum flux arising from the superimposed wave motion.

Just substituting these equivalents in the left hand side terms of (4.46) gives

\[
\rho U_i U_j h + U_i \bar{M}_j + U_j \bar{M}_i = \rho \hat{u}_i \hat{u}_j h - \frac{\bar{M}_i \bar{M}_j}{\rho h}
\]

(4.48)

So the mean value in time of the depth integrated momentum equation reads
\[
\frac{\partial m_i}{\partial t} + \frac{\partial}{\partial x_i} \left\{ \rho \hat{u}_i h + S_{ij} \right\} = -\rho gh \frac{\partial h}{\partial x_i} \tag{4.49}
\]

This is the balance equation of total momentum per unit area of the motion, where

\[
S_{ij} = \rho \int_{-d}^{\zeta} \left( u_{w,j} u_{w,j} + \frac{p \delta_{ij}}{\rho} \right) dz - \frac{1}{2} \rho gh^2 \delta_{ij} - \frac{\bar{M}_i \bar{M}_j}{\rho h} \tag{4.50}
\]

represents the excess momentum flux that results from the unsteady wave motion. The rhs term of Eq 4.49 is the net horizontal force per unit area arising from the slope of the free surface (a pressure term).

This resembles the expression of the radiation stress form section 4.1.2. It is slightly different because of the difference in approach of the wave and current separation.

Treatment in (the mass-flux version of) TRISULA

TRISULA deals with the equations for fluid motion in a different form. Two main differences can be observed

1. Instead of using a separate equation describing the energy balance of a combined wave and current field (like in section 4.2), in TRISULA the energy losses are incorporated in the momentum equation, by adding some source terms.

2. The equations describing the momentum balances are solved in a frame, moving with the wave group velocity. The velocities yielded are those, indicated in this study by \( \hat{u}_j \). By post-processing the flow field according to \( \bar{U}_i = \hat{u}_i - \bar{U}_i \), the local velocities are obtained. If measuring near shore currents, one would read the velocity represented in this model by \( U \).

TRISULA momentum equations

In the TRISULA mass flux version, the momentum equation reads:

\[
\frac{\partial m_i}{\partial t} + \frac{\partial}{\partial x_i} \left\{ \rho \hat{u}_i h + S_{ij} \right\} = -\rho gh \frac{\partial h}{\partial x_i} - \tau_{\text{sh.stress},i} \tag{4.51}
\]

instead of (4.49). With as difference an added term for momentum losses due to the bottom friction.

HISWA does not compute the radiation stress for de forcing but uses the expression based on a gradient in dissipation.

\[
F_i = \frac{\partial}{\partial x_i} \left( \frac{D}{\omega} \right) \kappa \tag{4.52}
\]

It can easily be seen that this is essentially the same as (4.51). Still, attention should be paid to the fact that in the last reproduction of the equations of motion, the velocities on the basis of which the bottom friction is calculated should be adapted as well! (see 4.3.2)

Boundaries taking wave mass flux into account
At lateral velocity boundaries, the total flux velocities must be prescribed. When calculating while neglecting the wave mass flux, the total depth averaged velocity $\bar{\hat{u}}_l$ must be taken. If taking wave mass flux in account, the Eulerian velocity $U_l$ plus the wave mass flux velocity $<u_{u_l}>$ must be imposed. Concerning the seaward water level boundary, no adaptations are made. The extra mass that enters the domain by the waves, is taken into the calculation in the normal way.

4.4.4 Hydrodynamics with mass flux

Including wave mass fluxes in the calculations of the hydrodynamic modules essentially means a distinction between several types of velocities. In the derivation of the equations in the previous section they were named $U_i$ and $\hat{u}_i$. According to the provenance of the data from a DELFT2D file they are called respectively $U_{euler}$ (because it is in fact a velocity resulting from an Eulerian approach) and $U_{com}$ because it is read from the communication file. $u_{u_l}$ as such does not appear in DELFT2D, the mass flux expression is used: $M = u_{euler} \rho h = E/c$, which holds the same.

In a certain point the velocities can be considered as vectors:

\[\text{shore}\]

\[\begin{array}{ccc}
\text{no mass flux} & \text{mass flux} & \text{no mass flux} \\
U_{com} & U_{com} & U_{com} \\
U_{euler} & U_{euler} & U_{euler}
\end{array}\]

\[\text{normal transect}\]

\[\text{sea}\]

Figure 4-3 Constitutive velocity vectors

The wave flux velocity is always pointed perpendicular to the shore in case of waves approaching normal to the shore. Differences in flow pattern because of the implementation of mass fluxes will only evolve if the solution of the momentum equations for the combination $U_{euler} + u_w$ is significantly different from the solution of $U_{com}$(without mass flux). This can only be the case if either the mass flux is very sensitive for the ambient flow, or if $U_{euler}$ differs a lot of $U_{com \ no \ mass \ flux}$ because of experiencing a different friction.
5 Simulating Rip Currents using DELFT2D-MOR

This chapter treats the aim and scheme of the computations for the simulation of the evolving rip currents. Choices for the initial topography are explained, just as the physical processes DELFT2D can simulate. Steady state hydrodynamics are conditional for reliable morphodynamic simulations on the basis of the computed flow field. As this situations is not reached anyhow, attention is paid to the way steady state hydrodynamics can be reached most efficiently.

5.1 Aim of the simulations

Rip generating mechanism

In chapter 3 several mechanisms of rip current generation are described. The question that remains is which mechanism is most likely to generate rips. The answer is sought by simulating a wave induced flow in DELFT2D-MOR. Two main approaches can be distinguished, viz. simulations of rip development by:

- hydrodynamic causes, or by
- morphologic causes

Beforehand there is no real preference for one or the other approach because our knowledge is incomplete.

Concerning simulation according to the first category, the following can be observed:
If it was exactly known what triggers the generation of rip currents (waves, wave-current interaction, instabilities, etc.) on a perfectly longshore uniform beach, it would be possible to model that mechanism and its subsequent changes in morphology. But because there is no conclusive theory on rip current generation, this is no possible way of modelling rip current dynamics. Therefore, another approach could be used:
It could be possible that the mathematical model (in this case DELFT2D-MOR) gives a conclusive description of all relevant physical surfzone processes and their interactions, although no comprehensive rip current generation theory is available. Assuming the model has this quality, an initial longshore uniform bottom would display the development of a rip current channel under special (boundary)conditions, as also observed in reality.
To apply this approach, one starts with a longshore uniform beach and simulates the hydrodynamics. In 6.2 is given account of the attempts with this approach.

Simulating on the basis of the second category theories (morphological causes) supposes the following reasoning: Although it is not possible yet to indicate which mechanism triggers a rip on a perfectly longshore uniform beach, it can be said that whenever a rip current occurs, morphology is closely involved. Physical reality is to complex to be able to model all processes and interactions which play a part in coastal dynamics. Though it is not known how the generation of a rip current is triggered exactly, once a rip is developing, an irregularity in longshore morphology will grow. Most efforts in this study are resulting from
this approach with an irregularity in longshore direction. In section 6.2.2 the choice of the initial disturbance is explained.

**Maximising the rip current velocity**

From the beginning it was clear that it was of crucial importance to maximise the velocities in the rip channel when trying to simulate the development of a rip current. With respect to this the following considerations play a part:

- If the results of the simulations are valid and corresponding with observations in reality, then they show a fast evolution of the rip; Fast morphological changes are only possible with the strong velocities as observed in physical reality.
- If the longshore, locally curved wave dissipation pattern (as observed on e.g. ARGUS images) results from the wave-current interaction, then this effect would be most clear when the opposing current is large.

Therefore, the choices regarding the simulations are mainly aiming at maximum velocities in the rip channel. This explains why so many tests were done concerning hydrodynamic parameters, as described in chapter 7.

**Angle of wave incidence**

In chapter 3 it was argued that longshore differences in the radiation stress can be the driving force behind the flow in a rip current system. The most simple case is that of perpendicular incident waves because, in that case, the flow through the rip is not influenced by any background longshore current.

In this study a first choice is made to simulate this simple case. This choice can be justified by the fairly symmetrical path of floaters of De Graaff (1995), released in the feeder channel on both sides of the fresh rip, which can not exist with a strong longshore current. Although these measurements were done in a quite well developed rip, and not exactly at the initiation of the rip, the simulation results make clear that assuming perpendicular incoming waves is certainly not absolutely mistaken. Thus the simulations are at first aimed at simulating a rip, solely driven by longshore gradients in the radiation stress with normal incident waves.

### 5.2 Physical process

The modelling system describes the development of a physical process in time, starting from a given initial state. For that modelling of the morphodynamic process, physical sub processes are distinguished. The allowable physical processes account for one or more of the following physical effects and their interactions:

- **hydrodynamics**
  - waves
  - flow
- **morphodynamics**
  - sediment transport
  - bed level variations
Waves and flow together form the hydrodynamic subprocess. Sediment transport and bedlevel variations draw up the morphological part.

For the simulation and prediction of morphological changes in a rip current system, it is required that waves, currents and bathymetry are modelled in an integrated way. All processes regarding these different quantities are closely correlated.

Given an initial bottom, the hydrodynamics cause a change of the morphology. In this study a rip current is a wave induced flow. Tidal currents and flows of other origin are left out of consideration. The only active external forces are the raised by the waves. Therefore first the wave action field, given a bathymetry, is computed. As mentioned, the waves induce a flow. So subsequently, given a bathymetry and a matching wave action field, the flow field is worked out.

In consequence of wave-current interaction, the flow field acts on the wave action field. Because of this mutual influence, iterations between waves and flow must be executed. Assuming no changing boundary conditions during the iterations and assuming convergence of the iteration process, one may expect a stationary outcome of the wave-current interaction. In section 5.4 it is explained in what ways the iterations, for realising stationary hydrodynamics, can be fashioned to realise stationary hydrodynamics.

Once these stationary hydrodynamics are reached after some iteration loops, the morphological sub processes can be started. First the gradients in sediment transport capacity for a period of time have to be computed. Next, bottom changes occurring in this or a multiple of this period can be evaluated and added.

**Continuity corrections**

After a considerable bottom change, the hydrodynamics do not agree anymore with the changed bottom topography. The water movement has to be updated. This can be done in two different ways. One can start again with wave-flow iterations on the changed bathymetry, but one can also assume that local discharges are constant when bottom changes are relatively limited. This enables a quick adjustment of the flow field and confines the computational effort. The adjustment of the flow field on the assumption of constant local discharges is called 'the continuity correction'. This continuity correction is used to update the water movement in this study. The effect of this continuity correction on the morphological changes is discussed in section 8.4.

At the moment the simulation time is reached, the computation ends.

**5.3 DELFT2D-MOR**

In this study the DELFT2D-MOR modelling system is used. This system is developed by DELFT HYDRAULICS. DELFT2D-MOR stands for the part of the DELFT2D modelling system, suitable for morphological applications. It is a system to simulate a (physical) morphodynamic process by a mathematical model. The mathematical model description concerns field equations for a two-dimensional horizontal area.

The modelling system consists of a set of modules which simulate the above-mentioned physical effects, viz.:
- WAVES: computes wave field parameters as wave height, wave period, wave forces etc. for a given set of current fields and a given bed level;
- TRISULA: computes the flow field development (velocities, water level) for a given bed level and wave forces;
- TRSTOT: computes the sediment transport for a given wave and flow field;
- BOTTOM: computes the bed level variation for a given sediment transport.

In this study all these modules are applied. The instructions for DELFT2D-MOR comprehend over 600 records and fields to be allocated. In order to get a realistic simulation and faultless execution these should not contradict too obvious. Data exchange between the modules is managed by using communication files. The different groups on the communication file contain different time frames so correct interchange of particularly time registration is of great importance.

5.3.1 Compound model MAIN

The module MAIN controls the modelling of the physical process. It regulates the sequence of calling the physical subprocess modules and their overall time management. The operating procedure of MAIN can be described using a process tree. It is an hierarchic system build out of branches and nodes.

A node can stands for a physical subprocess module (an elementary subprocess) or a controlling subprocessor (a subprocess). There is one node of the highest order; the root, in fact represents the morphodynamic process. The four elementary subprocesses are always of the lowest order. Subprocesses (hydrodynamics and morphological change) regulate the modelling on levels between root and physical subprocess modules.
Figure 5-1 shows a graphical representation of the process tree. Modelling the development of a rip current system, using DELFT2D-MOR, gives the following stepwise repercussion:

- **HISWA**
  - execution of HISWA. The wave action field is computed. Because HISWA predicts stationary waves, no adaptation of the MAIN time is done;

- **TRISULA**
  - execution of TRISULA for a short period (order of minutes). TRISULA uses the resulting wave forces from the HISWA computation.
  - Time increment only for internal TRISULA purposes;
• TRSTOT
  execution of the sediment transport capacity module. The rate of erosion or
  accretion is averaged over the same period as the TRISULA run. The rate of
  erosion or accretion is multiplied a number of times as specified by the bottom time
  step. This time step can be chosen either fixed or automatic (based on a constant
  courant number).
  The process updates the MAIN time;
  BOTTOM
  The previously stored bottom is updated with the sedimentation or erosion
  quantities;

• Hydrodynamic controller
  Regulates HISWA-TRISULA iteration.
  This controller operates with a stop condition. The process executes an accuracy
  test. It compares the value of the maximum deviation of the flow magnitude of the
  previous and current flow field. If this accuracy is not met within a given number of
  iteration loops (approx. 20.), the process stops;

• Morphodynamic controller
  Regulates the number of executions of the bottom adaptations (approx. 15
  depending on the magnitude of the TRSTOT (automatic time) step). After each run
  of TRSTOT and BOTTOM the flow properties are adjusted using the continuity
  correction. In most runs no increment test of bottom changes is used because that
  complicates comparison of different runs with different simulation times;

• MAIN controller
  Regulates the execution of both the morphodynamic and hydrodynamic process. A
  stop time for the simulation or a number of executions can be specified.

5.4 Wave-Current Interaction

5.4.1 Convergence and steady state

In case of wave induced flows, like rip currents, modelling the interaction between waves
and flow is of main importance. In this project thorough research is done to achieve
stationary hydrodynamics in DELFT2D output. Because of the cyclic writing to the
communication files in DELFT2D-MOR, it is not so obvious to get an insight in the
behaviour of the process while iterating.
When talking about the iteration process a distinction can be made between: stabilising of the iteration process (convergence) and stabilising of the resulting flow field (steady state). Both aspects are treated below. The course of the iteration process can be made visible, like in Figure 5-2.

Stabilising of the iteration process (convergence):
- It should be made acceptable that the equilibrium state (a completely settled combination of wave and flow field) that is reached for the hydrodynamics is a realistic state and not one of the many equilibrium states. When, for instance both the deviations of wave force and current velocity between the last and last but one loop keep up, the process is converging. In such situation, it is assumed that the one and only equilibrium is reached;
- Not converging of the computation results in contradictory outcome. This was also examined by De Graaff (1996). Not converging of the process can originate from:
  - the iteration method;
  - the TRISULA period within the iteration loop;
  - too small eddy viscosity;
  - unacceptable bottom topography.

Stabilising of the resulting flow field (steady state):
- Not reaching the steady state hydrodynamics can result from resonance in the model or from the unstable or pulsating character of rip currents. Reaching the steady state (yes or no) can be read from the TRISULA history file. When plotting time against velocity, a horizontal line indicates a steady state. Another way of getting insight in the computation process is by extracting EPSA numbers (stop condition controller) from the DELFT2D output file:
  - As will be demonstrated in chapter 7, the weakly reflective boundary conditions are of great value damping out resonance of numerical origin;
As all boundary conditions are steady, it is expected that sooner or later all oscillations will dim. Thus oscillations resulting from the character of the current itself, for instance observed by Sonu (1972), can not be predicted by the model. Variations in the current that keep on appearing must be erroneous of origin. If they are limited, the outcome of the simulation is acceptable. They can be prevented by taking another period for TRISULA or another value for the viscosity or friction.

For all these effects, time is needed for ‘warming up’, either the length of the TRISULA period, the number of HISWA time steps or the number of iterations, can be used for knocking up the model.

5.4.2 Structure of the hydrodynamic computation

Various methods to compute the wave-current field are worked out. They differ in sequence of calling the hydrodynamic modules. Each method can be run with different periods for computing and writing. This results in countless possibilities for the hydrodynamic part of the simulation. All these variants are put into order by giving them a code: ‘H. T_{min}.m.n’

- First position: number of HISWA time steps;
- Second position: TRISULA period in minutes;
- Third position: number of max. TRISULA executions following each other;
- Fourth position: number of executions of the hydrodynamic controller.

In the following of this section, a number of variants will be treated. A @ in the code means a non-zero value, while * means a value larger than one.

**1.43.1.1**

![Diagram](image)

Figure 5-3 Structure 1.43.1.1

In fact in this variant no real wave-current interaction takes place because there is no iteration. This variant is used in order to determine qualities of the computational domain, notably the frequencies of oscillations of numerical origin. In appendix C 2.2, it indicated
by id ‘is8’. In 7.2.1 is described how these frequencies are used for deriving the boundary reflection coefficients.

The period is taken 43 minutes because that is sufficient to spin up the flow.

This variant is also used as reference for comparing influence of the wave-current interaction on the final result. Although, refraining from computing the interaction, provides higher velocities in the rip, yet it should be applied in order to get more realistic results. In appendix C 2.2 is illustrated that computing wave-current interaction is required.

There are several advantages of computing wave-current interaction, compared with not taking it into account. Computing wave current interaction:

- enables studying wave induced flow at an longshore uniform beach (see section 6.2). For instance the vector velocity field has a different look with a strong eddy at the waterline in case of no wave current interaction
- results in deviation of current magnitudes;
- results in differences in set-up while the wave dissipation is nearly equal. This is only explicable because $\sigma$ in the wave forcing expression is changeable;
- prevents at first stopping of the morphological development of the rip. When a sill is deposited and the beach slope is locally less steep, then the effect of the currents pushing the wave dissipation peak offshore is present. This is in favour of the extension of the sill;
- also thwarts the rip current. Because of the local high energy conditions at the downstream end of the rip neck, the stream towards the rip head weakens and the extension of sill is resisted. (See also LeBlond and Tang (1974) and section 3.1.2);
- shows the local increase of the wave height at the rip head (current shoaling);
- makes that contradictory dissipation-current fields can be avoided by the iterating process;
- enables the studying the influence of current refraction;
- enables answering the question what can be seen on ARGUS images, because the origin of the wave dissipation bands on the images is not known yet.
- gives the conviction that the outcome of the simulation is more realistic.

1.@.1.20

This method works the best. When the TRISULA period is taken small enough, the iteration process is converging. The determination of the TRISULA period is discussed in section 5.4.3.
1.@.30.20

This method, used in the validation DELFT2D with the HISWA basin, was not converging in the situation and domain of this study.

@.@.0.1 = 1.@.0.1

When no iteration loops are called, it makes no sense to calculate a wave field at a number of times.
15.90.1.10

This method works out a kind of under relaxation. This method only works if the TRISULA period is large and writing is not cyclic. This would be a nice method because the starting up of the flow could be viewed. When a converging solution would be found, one would have an adapted wave-current interaction field at every moment and would be certain about converging of the process. Unfortunately it was not managed to achieve a converging outcome.

An other advantage of this method would be, that it would enable deducing the character of oscillations that remain in the solution even after the iteration becomes stable.

*.@.*.@

Brings no new results, in fact the same as 1.@.30.20

@.@.*.0.1

Does not work out. TRISULA may give a stationary result but will probably keep on rock between two or more current fields. No real wave-current interaction takes place, the wave field is not adapted.

5.4.3 wave-current interaction TIME TESTS

In order to obtain stationary hydrodynamics with a minimum of computational effort, several tests have been executed. Each test case was a simulation of the wave-current interaction according to ‘1.@.1.*’ with different TRISULA periods.
Striving for stationary hydrodynamics, the length of the period that is actually simulated is not very important. However, what is interesting, is the speed of converging of the process (if it converges at all) and the time needed for the calculation. This 'calculating time' is not unambiguous, it is strongly related to the computer power and network load. Nevertheless, it is clear that writing and reading operations increase the 'calculation time' and that TRISULA executions charge more in general than HISWA computations.

DELFIT2D gives the opportunity to define stop conditions of a controller which is calling a sub process. This stop condition can be an accuracy test. Using the stop condition is very useful in controlling the number of iteration loops executed.

Several fields on the communication file can be used for an accuracy test. In this study the extent of hydrodynamic stationarity is controlled by a stop condition. The quantity used is either the wave force or the current magnitude. Most of the time the accuracy condition is not fulfilled and the calculation stops because the maximum number of executions is reached. And often the accuracy condition, will never be met because the result is though converging not exactly steady. Increasing the value of the maximum number of iteration loops will not help to reach the accuracy condition because some oscillations will stay.

![Graph showing iteration times for TRISULA period](image)

Figure 5-6 EPS time test for TRISULA period

Figure 5-6 shows the result of the time tests. It appeared that TRISULA periods of more than 12 minutes resulted in not converging calculations, for the applied domain. The smaller the TRISULA periods, the stronger the feedback but the longer the calculation time. An optimum for the TRISULA period was found to be about 6 minutes.

### 5.5 Outline of the simulations

The handles that can be adjusted when simulating a rip current can be divided into four categories, which determine the outline of the description in the next two chapters:

- **domain**
  - profile
  - longshore uniform
- initial rip channel

- waves
  - domain
    - boundary
    - grid
  - angle of incidence
  - wave height
  - dissipation pattern

- flow
  - domain
    - boundary
    - grid
  - viscosity
  - bed shear stress
  - water level

- Morphology
  - transport
    - bottom & suspended
    - magnitude
    - direction
  - bottom change
  - time step

The special variations in input for the various simulation runs is explicitly mentioned. If not then ceteris paribus conditions are applied. That is to say:

- cross shore profile with wide trough
- angle of wave incidence = 0 degrees (perpendicular to the shore)
- still water level = 0.0 m
- $H_s$ = 1.5 m
- $f_w$ = 0.05 m (White-Colebrook)
- viscosity = 1.0 m$^2$/s
- transport calculated with Bijker's formulation in module TRSTOT
- no correction for sediment transport magnitude nor direction because of the bed level gradient
- NSTAB = 6 (option determining stabilisation of bedlevel computation)
- fixed morphological timestep of 30 min.
- in case of automatic timestep determination a courant number of 0.7 is applied

Not from every simulation run, the results are taken up into the appendices.
6 Domain

The numerical model for obtaining predictions of waves, currents and morphology uses a given domain with a bathymetry and the offshore wave conditions as input. This domain is bordered by a shoreline along one boundary and an offshore region along the opposite boundary. Modelling a rip current system requires an initial bathymetry. An initial bottom has to be chosen.

The area of the computational grid is enclosed by four boundaries. In physical reality disturbances can travel out of the ‘computational’ area unharmed and without reflections. In a numerical model the interaction with the outside world must be included in the boundary conditions (section 6.1). Apart from vertical boundaries of the domain, a topography has to be specified as well. In the course of this project several alternations of the initial bottom were made. Effects of these are described in section 6.2.

6.1 Boundaries

The distance of the offshore boundary from the shore is determined by the following considerations

- The bottom elevation in the cross shore profile must be sufficiently large so the incident waves field has not been transformed yet. This is a wave effect.
- The rip current may not discharge through the domain boundary. This is a flow effect.

Due to the flow viscosity and gyres beyond the surfzone, the extend of a rip current is confined. So the first criterion is far normative and results in a cross shore length of the grid of 830 m.

Onshore boundary

The onshore limitation of the domain is the beach. This is a natural limit modelled as a closed boundary. The beach consists of inactive dry points.

Cross-shore boundaries

Two cross-shore boundaries complete the enclosure. The boundaries must be sufficiently far from the region of interest (i.e. the central rip channel) to avoid affection of the result unintentionally. The boundaries for the waves and the flow can be considered separately (see 7.1.3 and 7.2.1). A schematic overview of the domain configurations is given in Figure 6-1.
In case of initial rip currents on an open beach, velocities are small for the most part, so disturbances are easily dominant. In order to get no disturbances in the border area’s, it turned out that the size and borders of grids and nestings should be chosen carefully.

### 6.2 Bottom topography

As the consideration behind this study was to use images of the surfzone, made by the ARGUS camera in Noordwijk (NL) for verification of the model results, a representative beach elevation profile at that location was used as input for the bottom-file. A typical profile is shown in Figure 6-2. The profile contains three breaker bars, parallel or nearly parallel to the coast. Coming from offshore, they increase in height. Their spacing is about 200 m. At the offshore border the bottom elevations is about -7 m.

![Figure 6-2 Bottom profiles with widened trough](image)
6.2.1 Longshore uniform beach

One might prefer starting morphodynamic simulations of the development of a rip system with a longshore uniform topography. Results of simulating nearshore dynamics, starting with a longshore uniform barred profile are presented in appendix D.1.3 and Table 6-1. Phenomena which resemble circulation cells occur, although the effects are weak.

Table 6-1 Velocities and spacing on a longshore uniform beach with various wave heights

<table>
<thead>
<tr>
<th>Significant height [m]</th>
<th>Wave Peak Period [s]</th>
<th>friction coefficient</th>
<th>Id</th>
<th>U_{max, offshore} [m/s]</th>
<th>Averaged rip Spacing [m]</th>
</tr>
</thead>
<tbody>
<tr>
<td>1.0</td>
<td>5</td>
<td>0.05</td>
<td>pw1.2</td>
<td>≦0.006</td>
<td>86</td>
</tr>
<tr>
<td>1.0</td>
<td>5</td>
<td>5.0</td>
<td>pw1.1</td>
<td>158</td>
<td></td>
</tr>
<tr>
<td>1.5</td>
<td>6</td>
<td>0.05</td>
<td>ph5.2</td>
<td>≦0.006</td>
<td>159</td>
</tr>
<tr>
<td>1.5</td>
<td>6</td>
<td>5.0</td>
<td>ph5.1</td>
<td>150</td>
<td></td>
</tr>
<tr>
<td>2.0 (no steady state)</td>
<td>7</td>
<td>0.05</td>
<td>pw2.2</td>
<td>≦0.07</td>
<td>105</td>
</tr>
<tr>
<td>2.0</td>
<td>7</td>
<td>5.0</td>
<td>pw2.1</td>
<td>123</td>
<td></td>
</tr>
</tbody>
</table>

Tests point out that perturbations of numerical origin in the model easily dominate the germ of a rip, especially in the uniform initial situation. So it is doubtful whether rip generation is modelled or only the outcome of numerical arbitrariness. Though it is certainly no complete nonsense. The expected increase in spacing when wave height grows is not found. But it should be said that not all the iterations did completely converged.

The runs with a very large friction (White Colebrook fr = 5.0: among others ph5 and pf5) do reach a steady state and moreover, a very regular longshore wave pattern is observed. The overall 3D picture looks like a phenomenon that resembles edge waves (see chapter 3). But in stead of resonant waves, it looks more likely that the longshore variations in set-up and wave height originate from wave-current interaction and instabilities in set-up. Any way, a flow pattern results that resembles a kind of wave driven circulation cell. Zero crossings of the water level which are characteristic for edge waves are not present. These are after all impossible in the model that does not compute set-down.

It is true that the 'spacing' is less than the 350 -600 m Kroon (1997) mentions, but this is about initial rip, without a pronounced bottom irregularity.

On a uniform slope, without bars, these circulation cells occur to a smaller extend.

More research is needed for better simulations and understanding of these phenomena.

The resulting velocities are that small that they are very sensitive for e.g. changes in viscosity, and friction. They are too small as well to use for reliable morphodynamic computations.

6.2.2 Initial rip channel

As argued in section 5.1 an artificial irregularity is put in the longshore uniform profile. Chosen is for a disturbance as small as possible. There are two reasons for this choice, viz:..

Firstly it is not known from measurements what the bathymetry of a well developed rip system looks like. Modelling morphological changes, using an arbitrary large rip channel as starting point, could lead to developments of which the link to reality is uncertain.

Secondly, a larger initial rip channel implies that a substantial part of the available volume of sediment is vanished before only one time step is simulated.
In brief, a small local deepening in the longshore uniform beach bar is applied. The first stages of the morphological development of the rips appeared to be sensitive to sharp edges in the transition of the uniform profile to the local deepened rip profile. For the sake of smooth transitions, several intermediate rip profiles were inserted in longshore direction. A narrow but deep cut in the bar as initial rip with sharp edges resulted in progressive disturbances in morphodynamic computations. All the more a reason to keep irregularities limited and smooth.

**Profiles**

At first, all runs were made with an original representative measured profile of the Noordwijk '96 field campaign. Trying to drive more water through the rip in order to obtain faster morphological changes, later attempts were on the basis of a profile with a broadened trough shoreward of the highest breaker bar.

Figure 6.2 gives the profile for the uniform parts of the input bottom with the rip channel crossing the first bar.

**Results**

Results of hydrodynamic simulations with various topography are given in Table 6-2 and in appendix D.1.2.

<table>
<thead>
<tr>
<th>Profile</th>
<th>Id</th>
<th>$u_{t, \text{max}}$ [m/s]</th>
</tr>
</thead>
<tbody>
<tr>
<td>Original Noordwijk profile (narrow trough)</td>
<td>i5n (in6)</td>
<td>0.22</td>
</tr>
<tr>
<td>widened trough</td>
<td>i6t (nf6)</td>
<td>0.28</td>
</tr>
</tbody>
</table>

Comparison of the flow fields show a much more plausible flow field in case of the widened trough. In case of the profile with narrow trough a pronounced eddy occurs, shoreward of the nearshore bar. In case of a widened trough, the flow towards the rip is much more from behind the bar in contrast to the narrow trough, where the flow is along the bar face. This results in higher velocities in the rip channel in case of the widened trough which proves once more how sensitive the result of the simulation is for initial bottom topography and longshore discontinuity.

In case of oblique incident waves the width of the through turns out to play a very significant role. This is illustrated in the subsection about oblique incident waves.

De Graaff (1996) simulated rips with wide rip channels in DELFT2D as well. As expected, with a wide rip channel resulting velocities in the rip are higher. Simulations for that study show velocities of about 0.35 m/s in wide rip channels. But since this study is restricted to initial rips, these tests are not reported here.

**6.3 Evaluation of the domain related tests**

The following can be remarked concerning domain and topography choices:

- the configuration of grid layouts can introduce small, but clearly present spurious currents
- the flow pattern and velocities of the simulation are strongly dependent on:
— the dimensions of the rip;
— the width of the inner trough;

- morphodynamic computations avail by smooth transitions in topography
- a uniform beach shows regular circulation cells of which the spacing is dependent on the wave height.
7 Hydrodynamic Modules

7.1 Wave Module

HISWA is the wave module of DELFT2D. It predicts the properties of stationary short-crested waves in shallow water with ambient currents. The model HISWA estimates wave parameters in coastal areas from given stationary wind, bottom, and current conditions. The basis of the model is a parameterized version of the action balance of the waves (see Chapter 3).

The action balance approach implies that for each spectral wave component of the wave field the rate of energy change is equal to the net effect per unit time of wind growth, bottom friction and dissipation. The wave spectrum in HISWA is discrete spectral only in the directions; it is parametric in the frequencies. That is to say, two quantities propagate in each spectral direction: a frequency-integrated energy density and a mean frequency. Both quantities vary across the domain. HISWA computes this variation by integrating the local effects of wind, bottom and currents while propagating with these quantities at the group velocity of the mean frequency across a grid. The computations are carried out for each wave component separately for a system of the speed of action and the speed of frequency evolution equations (see HISWA manual for further information).

7.1.1 Grid dimensions and boundary

The longshore width of the HISWA computational area must be larger than that of the area of interest, because along each lateral side of the grid a region exist where the wave field is disturbed by an import of zero energy from the lateral boundaries. The angle of the line dividing the disturbed area from the undisturbed area is an input parameter of HISWA. Chosen is a value of 30° which is typically for waves generated by local wind. This results in a longshore length of the HISWA computational grid of 3500 m, taking into account the enclosure of the TRISULA area. The cross shore width of the HISWA area is taken slightly larger than the TRISULA domain.

In order to limit the computational effort, a nested grid is used. The result of the simulations turned out to be very sensitive to the positioning of the nest. On a plane beach, where velocities are very small, the difference is most obvious. Best results were obtained when the nest is a little larger than the TRISULA grid.

7.1.2 HISWA grid resolution

In this study it turned out to be advisable to use a nested grid for the HISWA computation. While in the largest part of the domain with the central rip, not much longshore irregularities are expected, the longshore size of the meshes can be fairly large, here 50m was taken. In cross shore direction the depth isolines are parallel but close together, so a grid size of 5 m is applied. This does not apply for the area of the rip system. There, the longshore grid distance must be refined so meshes of 5*12.5 m are taken, small enough to be able to distinguish the rip current effect in the wave fields. The grid uses is also shown in Figure 7-1.
7.1.3 HISWA boundaries

The waves enter the domain through the offshore boundary where longshore uniform wave conditions are applied. If not specifically mentioned, it can be supposed that a significant wave height of 1.5 m is applied. Over the lateral boundaries zero energy transfer is imposed.

7.1.4 Wave height variations

Several runs with different wave heights were carried out. An overview of the results is given in Table 7-1.

Table 7-1 Velocities in initial rip with various wave heights

<table>
<thead>
<tr>
<th>Significant Wave height [m]</th>
<th>Peak Period [s]</th>
<th>id</th>
<th>U_rip [m/s]</th>
<th>Set-up [m] (nor in trough)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1.0</td>
<td>5</td>
<td>sw1</td>
<td>0.20</td>
<td>0.04</td>
</tr>
<tr>
<td>1.5</td>
<td>6</td>
<td>sw5</td>
<td>0.24</td>
<td>0.09</td>
</tr>
<tr>
<td>2.0</td>
<td>7</td>
<td>sw2</td>
<td>0.23</td>
<td>0.13</td>
</tr>
</tbody>
</table>

In accordance with expectations the set up on a normal transect increases with increasing wave height. Notably is that the velocity in the rip even decreases with increasing wave height. An explanation can be found in the graphs in appendix D 2.1. For the highest waves applies that the value of maximum dissipation in the rip head is larger than the maximum value on a normal profile. This is the result of the increase wave breaking due to the opposing current. It can not be contributed to the increase in dissipation due to bottom friction, experienced by the waves, since it is compared with a normal cross section with the same wave conditions.

The enlarged value of wave dissipation profile results in more set up in the rip head compared to a normal profile (see graph in appendix D 2.1). And so the rip current is more resisted in case of the highest waves. This looks like a confirmation of observations of e.g. Kroon (1997) that rips develop under moderate wave conditions. Probably an optimum wave height can be found between H_s = 1.5 m and 2 m which induces a maximum flow in the rip. These tests justify the use of H_s = 1.5 m for the morphodynamic simulations.

An additional argument for using H_s = 1.5 m in the following morphodynamic calculations is that wave height is not a local quality, an increase will cause larger transports everywhere because more sediment is brought into suspension. The development of the rip channel would be nullified by the overall change of morphology. When an equilibrium beach profile
would be known for a certain sea state, it would be much easier to use the wave height as an adjustment parameter.

Variation of the wave height does not result in very deviating flow patterns as is illustrated in appendix D 2.1

The wave height is also altered in the morphodynamic computations. Again waves with a significant height of 1.0, 1.5 and 2.0 m where imported in the model. In the long run, no significant deviations in morphological development between the runs were observed (results of these simulations are not recorded in the appendices).

### 7.1.5 Obliquely incident waves

On the basis of field observations it is known that the strongest rip development occurs when waves approach the shore normally or nearly normally. In case of normal incident waves, only a local nearshore circulation cell arises. But under oblique incident waves a strong longshore current appears due to the radiation stress forcing. It was thought that a small longshore current might play an important part in rip development because it might raise the rip velocities. Moreover the longshore current would attack the downstream bank of the rip channel and thus contribute to its widening. Therefore tests where carried out with oblique incident waves, the results of which are shown in Table 7-2.

<table>
<thead>
<tr>
<th>angle of incidence [°]</th>
<th>id</th>
<th>( u_{\text{max}} ) [m/s]</th>
<th>( v_{\text{max}} ) [m/s]</th>
</tr>
</thead>
<tbody>
<tr>
<td>narrow 10</td>
<td>ho1</td>
<td>0.16</td>
<td>0.35</td>
</tr>
<tr>
<td>narrow 5</td>
<td>ho2</td>
<td>0.17</td>
<td>0.25</td>
</tr>
<tr>
<td>wide 10</td>
<td>ho1</td>
<td>0.217</td>
<td>0.31</td>
</tr>
<tr>
<td>wide 5</td>
<td>ho2</td>
<td>0.283</td>
<td>0.24</td>
</tr>
</tbody>
</table>

In contrast with the anticipated result, the rip current velocity did not increase but even decrease at first with the narrow bar. The longshore current was too dominant and smoothed the rip current completely away. Consequently the morphological changes in the model were not spectacular. The direction of the channel changed a little bit with the longshore current, but did not show the expected broadening. Afterwards, when the troughs were widened, the resulting flow fields changed significantly. Velocities in offshore directions increased till more than average values. Simulations show that it is much easier to generate high velocities in a DELFT2D domain with a excess longshore transport of onshore momentum compared with only onshore momentum and no flux longshore. Appendix D 2.2 shows the vector velocity fields with oblique incident waves. In section 8.5, morphodynamic simulations with oblique incident waves are discussed.

### 7.1.6 Wave dissipation and set-up due to an opposing current

In the section 4.3, describing wave energy equations, it was already mentioned that the rip current opposing the waves causes a local accumulation of energy at the rip head. The results of the simulations confirm this statement. The increase of energy results in higher waves, subsequent higher dissipation in the rip head and locally an increase of the water depth due to wave set-up.
For the application of ARGUS video techniques in coastal engineering, it is important to know what causes the more offshore located breaking of waves in a rip, compared to the adjacent beach. When no wave-current interaction is computed, the single cause of wave breaking is the water depth. When calculating with waves influenced by the current, both breaking due to the water depth and the opposing rip current are involved. But differences that occur cannot be attributed to the opposing current in the rip alone. When calculating with wave-current interaction, the water depth increases due to changed wave set-up. This affects the wave breaking pattern as well.

It is not possible to isolate the influence of the set-up by comparing the wave dissipation in a normal cross section for a run including wave-current interaction and a run without wave-current interaction, and rectify the dissipation in the rip with the same difference because the wave set up is not uniform along the beach after all. Wave set-up results in larger water depth and thus causes a shift of the wave dissipation profiles towards the shore. The influence of the current has the following effects in the rip as can be conclude from the graphs in appendix C 2.2:

- shift of the wave dissipation peak downstream;
- increasing wave height (wave blocking, current shoaling);
- rise of the wave dissipation peak.
- increase in set-up at the rip head (see longshore set-up graph in app C 2.2)

The results of the computation show no shift in the wave dissipation profile towards the shore (increased water depth). A slight shift of a few meters downstream is observed. This arises from the interrelated facts that

- the rip reaches its maximum velocity close to the location where the waves break due to the water depth;
- the shore face is steep at the bar. Waves which are about to break are concentrated on a short distance;
- the effect of the larger water depth at the nearshore bar due to set-up is negligible for the wave dissipation.

A rise of the wave dissipation peak is visible. The peak is about 15% higher in case of wave-current interaction, compared with the case without. The increase in wave height is clear. Because this effect does not show up in the normal cross section this can not be caused by shoaling.

Comparing a normal cross section with the initial rip, then turns out that the wave dissipation profiles closely corresponds. The effect of the larger water depth is nullified by the effect of the opposing current.

### 7.1.7 Evaluation of the simulations with varied wave input

Findings following from simulations with various wave input can be summarised as:

- There is a certain wave height which produces maximum velocities the rip. This optimum wave height is in the range of moderate waves. Simulations with $H_r = 1.5$ m produce higher velocities in the rip than with $H_r = 1.0$ m and $H_r = 2.0$ m;
- The wave force is dominated by the dissipation. Only in case of high waves, wave number and frequency effect the wave force noticeable.
- variations in wave height do not alter the characteristics of the flow pattern;
- increase in wave height enhances the peaks in the dissipation pattern, though no shift in location is observed. (outcome of the combination of steepness of the bar face, enlarged water depth in the rip channel, current shoaling and wave energy);
- increase in wave height increases the wave forcing;
- increase in wave height increases the set-up, both in the rip and on a normal profile. The difference in forcing and set-up is about equal in rip and normal;
- none of the tests show a distinct wave dissipation peak at the waterline as observed on the ARGUS images.
- waves approaching under a large angle, (viz. 10° shore normal), generate a too strong longshore current. Effects of the rip are totally smoothed and indistinguishable;
- nearly incident waves (viz. 5° shore normal and perhaps even smaller), do strengthen the rip current. It should be mentioned that the flow pattern is very sensitive for the width of the nearshore trough. A narrow trough does not contribute to a boost of the rip current in contrast to a wider feeder channel;
- Simulations show that current refraction as suggested by Dalrymple et al. (1978) is of minor importance for thrusting water from the sides to the rip. This is substantiated by Appendix D 2.1.

7.2 Flow module

TRISULA is the physical subprocess module of DELFT2D which simulates the flow, in this case induced by wave forcing.

TRISULA uses an alternating direction implicit (ADI) method of solution. The equations, (4.4) and (4.6) mentioned in chapter 4, are solved for the three unknowns. The unknowns are $\zeta$, the mean water surface level, and $U$ en $V$, the two components of the depth averaged mean current. The equations are the continuity equation and the $x$- and $y$- direction momentum equations.

The ADI method first solves, grid row by grid row, for $U$ at the next time level and an intermediate value of $\zeta$, by solving the equation of motion in $x$-direction and the continuity equation using an implicit numerical scheme. This solution is in terms of the known values of $U$, $V$, and $\zeta$ at the present time level. Once all the grid rows are solved in the $x$ direction, the equation of motion and the continuity equation are solved in $y$-direction again by an implicit scheme. Grid column by grid column, for $V$ and $\zeta$ at the next time level in terms of $U$ at the new time level, $\zeta$ at the intermediate level and $V$ at the present level.

7.2.1 Domain

grid and bottom

Using a longshore uniform bottom with a central rip channel as only irregularity makes that not many disturbances of the initial uniform conditions should occur in the border areas. As, calculation time and required disk space are enhanced by an increasing number of grid points, the grid spacing is coarse at the borders and refined (5 m*5 m) at the rip location. Because the velocity magnitudes are relatively small in case of wave induced currents, spurious effects arising from grid discretisations can dominate the physical effects easily.

The computational domain of this investigation consists of a rectilinear co-ordinate system. The conclusion drawn from the simulation runs was that carefully chosen grids are critical
for reliable results on these scales of simulation. It appeared essential that grid size transitions are smooth.

Triangular interpolation methods inhibit generation of bottom-files in pre-processors like RGF and Quickin. Unfortunately the disturbance of the initial rip channel is smoothed and erased by the interpolation. So in aid of producing bottom files a MATLAB routine is written with an adapted interpolation procedure for each direction.

**TRISULA Boundaries**

The offshore boundary is modelled as a water level elevation. A stationary situation is studied so this boundary is unchanged in the simulation time, having a value of $\zeta = 0.0$ m. In case of normal incident waves, it is assumed that the influence of a developing rip current circulation around the initial rip channel is limited locally. So, on a certain distance of the rip channel, no significant flow will be noticeable. There the lateral boundaries of the TRISULA grid can be taken. Because there are cross-shore variations in water level elevation due to set-up, a zero lateral velocity boundary is applied. That is easier to implement than a water level boundary. See Figure 7-2 for the layout.

```
             water level elevation
              ------------------------
                |                          |
                | TRISULA                  |
                |                          |
                |    lateral velocity     |
              ------------------------
            closed boundary
```

Figure 7-2 TRISULA boundary configuration

Of course, when oblique incident waves are simulated, the cross shore boundaries have to be adapted.

**Weakly reflective Boundary Conditions**

A result of the mathematical definition of the boundaries of the model area is that waves which reach a natural or open artificial boundary will be reflected into the area. In case of natural boundaries this is no problem. In physical reality waves can reflect as well after all. But in case of open artificial boundaries, wave reflection is not according to natural circumstances. So it is required that artificial open boundaries let allow waves trough undisturbed. Such boundaries are called non- or weakly reflective boundaries. The advantage of weakly-reflective boundary conditions is that no oscillations by reflections against artificial boundaries heighten the fluid fluctuations.

Stelling (1984) proposed for TRISULA a weakly reflective form of the boundary conditions:
Water level boundary: \[ \zeta + \alpha \frac{\partial}{\partial t} \left( U \pm 2\sqrt{gh} \right) = F_{\zeta}(t) \] (7.1)

Velocity boundary: \[ U + \alpha \frac{\partial}{\partial t} \left( U \pm 2\sqrt{gh} \right) = F_{U}(t) \]

\( F(t) \) is the boundary condition specified for TRISULA. The reflection coefficient \( \alpha \) should be chosen sufficiently small to damp out reflected waves. The TRISULA manual advises the following values:

Water level boundary: \[ \alpha_{\text{wl}} = \frac{T_d}{g} \sqrt{\frac{h}{g}} \] (7.2)

Velocity boundary: \[ \alpha_{\text{vel}} = T_d \]

With \( T_d \) the time it takes for a free surface wave to travel between opposing boundaries.

Because it is difficult to determine \( h \) (for the propagation speed of disturbances) on a sloping profile, in this study an approximation of \( T_d \) (and \( \alpha \)) is obtained in a different way. First a simulation is run in the domain for quite a long TRISULA period with no reflective but stiff boundaries. The outcome is a flow field with periodic fluctuations. The main frequency of these fluctuations is determined by spectral analysis. The result from this analysis is used for the input in TRISULA.

In Figure 7-3 below, it can be seen that the fluctuations in longshore velocity (at a location without rip disturbance) dimmed significantly. There the TRISULA period (in the iteration loop) is 6 min.

![Figure 7-3 Weakly reflective boundary conditions](image-url)
7.2.2 Bed shear stress

For a depth-averaged flow (2D) the shear stress on the bed induced by a turbulent flow is assumed to be given by a quadratic friction law:

\[
\tau_b = \rho \frac{g}{C_{2D}^2} |U|^2
\]  

(7.3)

For the friction coefficient several formulations are available. In this study the Chezy formulation is used with a value of 65 m\(^{1/2}\)/s and tests with White-Colebrook (several values 0.1, 1.0, 5.0 m) are executed.

\(U\) is the depth averaged velocity. This shear stress parameter is important because it determines for a large part

- whether supplied momentum contributes to set-up or is directly consumed by flow,
- in which direction the momentum supplied by the waves is carried away.

**Results**

As expected, an increase in the model bed stress leads to damping of the current speed in the rip system. This is illustrated by the values in Table 7-3 below (especially the last coefficients are a kind of exaggerated in order to investigate the effect of the value of the bed shear stress on the simulations)

<table>
<thead>
<tr>
<th>Formula</th>
<th>for id</th>
<th>Bed stress (coefficient)</th>
<th>(U_{rip \ max.}) [m/s]</th>
<th>length feeder flow [m]</th>
<th>width rip head [m]</th>
<th>rip extension [m]</th>
</tr>
</thead>
<tbody>
<tr>
<td>Chezy</td>
<td>nf6</td>
<td>65</td>
<td>0.28</td>
<td>210</td>
<td>78</td>
<td>195</td>
</tr>
<tr>
<td>White-Colebr.</td>
<td>sn0</td>
<td>0.005</td>
<td>0.255</td>
<td>210</td>
<td>78</td>
<td>166</td>
</tr>
<tr>
<td>White-Colebr.</td>
<td>sn1</td>
<td>0.01</td>
<td>0.253</td>
<td>218</td>
<td>78</td>
<td>166</td>
</tr>
<tr>
<td>White-Colebr.</td>
<td>sn5</td>
<td>0.05</td>
<td>0.242</td>
<td>275</td>
<td>73</td>
<td>163</td>
</tr>
<tr>
<td>White-Colebr.</td>
<td>sh0</td>
<td>0.1</td>
<td>0.237</td>
<td>261</td>
<td>70</td>
<td>161</td>
</tr>
<tr>
<td>White-Colebr.</td>
<td>sh1</td>
<td>1.0</td>
<td>0.160</td>
<td>277</td>
<td>47</td>
<td>134</td>
</tr>
<tr>
<td>White-Colebr.</td>
<td>sh5</td>
<td>5.0</td>
<td>0.093</td>
<td>266</td>
<td>34</td>
<td>116</td>
</tr>
</tbody>
</table>

Furthermore, from the plots in appendix D 3.1 it follows that an increase in bed shear stress results in:

- increase of the differences in set-up between the rip and a normal cross section. The larger shear stresses produce larger gradients in the water level elevation.
- a concentration of the streamlines into the deepest area’s, viz. the trough and the rip channel.
- a reduction of the longshore distance from where water is pulled to the rip.

In contrast to what Arthur (1996) predicted, the friction does not broaden the streamlines. The effect of the preference of the flow for the deepest water depth is stronger.

In all the simulations, the shear stress on the bed for combined waves and currents is computed by using the friction model of Fredsoe (1984). No variations were executed.
7.2.3 Viscosity

In the simulations a uniform horizontal eddy viscosity is applied to compute the horizontal turbulent dispersive transport of momentum. It turns out that the resulting rip current is very sensitive for the value given to the viscosity coefficient.

Stability, velocity and flow pattern can be compared. Runs were made with a value of the viscosity coefficient of (amongst others) 0.1, 1.0 and 3.0 m²/s.

The computation with Viscosity = 0.1 m²/s did not converge. Unstable eddies occurred.

Table 7-4 Velocities in rip for various viscosity values

<table>
<thead>
<tr>
<th>id</th>
<th>coefficient of horizontal eddy viscosity [m²/s]</th>
<th>$U_{\text{rip max}}$ [m/s]</th>
</tr>
</thead>
<tbody>
<tr>
<td>vi0</td>
<td>0.1</td>
<td>0.357</td>
</tr>
<tr>
<td>vi0'</td>
<td>0.8</td>
<td>0.250</td>
</tr>
<tr>
<td>nf6</td>
<td>1.0</td>
<td>0.242</td>
</tr>
<tr>
<td>vi3</td>
<td>3.0</td>
<td>0.176</td>
</tr>
</tbody>
</table>

The lower the viscosity the higher the velocities in the rip channel. The lower the viscosity the longer the penetration of the rip trough the breaker zone and beyond. The lower the viscosity the narrower the rip current. Both effects are explainable by smaller shear stresses and velocity transfer.

The flow pattern over the bars is less influenced by the viscosity variation, just some more eddy activity is observed.

7.2.4 Evaluation of the flow simulations

Findings following from simulations with various flow input can be summarised as:

- The extension of the rip current through the surfzone, as observed e.g. by McKenzie (1958), is reproduced by the simulation. In a representative run, at a cross shore distance of 200 m from the waterline still a current of 0.10 m/s is conveyed;
- The velocity in the initial rip is in none of the simulations larger than about 0.30 m/s. This is less than the expected 1.0 m/s order of magnitude. Note that these computations are carried out with depth averaged velocities. The velocities that would be measured in a real rip at a representative depth will be more;
- The flow towards the rip comes from a longshore distance of about 250 m on both sides. This is within the range of 350 m- 600 m which is mentioned by Kroon (1997);
- The pulsation of the current is present in case of calculating with lower values for the viscosity. But this effect is of numerical origin (iteration process of waves and currents) and not similar to the variations observed by Sonu (1972) and Bowman et al. (1987);
- The flow pattern is in correspondence with the observations of Sonu (1972) that the water is approaching more from the side to the rip channel than trough the feeders.
- Friction does dominate the velocity more than viscosity;
- A higher mean water level (+0.30 m) does not produce significant deviating flow patterns.
8 Morphodynamics

On the basis of hydrodynamic calculations, which reached a steady state for the static flow, now the morphology can be simulated with a progressing time. In this chapter successively the transport and bottom level change module of DELFT2D-MOR will be treated, each with its most important variables. Further on, the result of the simulations will be discussed and evaluated.

8.1 Structure of the morphodynamic simulations

Like discussed in chapter 5, after obtaining a flow field the sediment transports are calculated on the basis of the current velocities, subsequently followed by time steps of bottom level change. In their turn morphological calculations are succeeded by either a continuity correction or a new hydrodynamic iteration process.

For the benefit of comparison of the resulting bottoms after an equal period of simulating, the computations are ended in this study after two (in a few cases more) callings of the hydrodynamic controller.

Computations of bottom changes in this project are inclined to get unstable, e.g. after feedback of the water motion. The length of the automatic time step (if applied) drops quickly and becomes very small (order of magnitude of minutes). This complicates comparing at the same output time. Comparison of quantified bottom changes is difficult any how because of:

- the strong three dimensional character of the bottom changes;
- the effect of the rips can not be isolated from the overall changes in bathymetry

8.2 Transport module

DELFT2D offers the choice out of different modules for the computation of the sediment transport rate. Each module, TRSTOT and TRSSUS, determines the sediment transport components \( S_s, S_n \) at the grid points in a specified time interval, using the flow and wave fields and a bed level available on the communication file. More exact, the discharge components from the flow field group CURTIM are used (instead of the velocity records). This enables the application of the continuity correction by which the velocity (resulting from an adapted bottom) can be updated without calling on the flow module for a new run.

It is very complicated, if not impossible, to get a comprehensive expression for all effects of sediment transport in a fluid in motion. Simplifications are made by distinguishing the nature of the transport; suspended, bedload or total (combination of both). A difficulty when simulating rip currents with all sediment transport relations, especially the ones for suspended transport, is that they can not easily handle zero velocities in combination with high rates of wave activity.

Attempts of calculating the sediment transport rates with TRSSUS, the suspended transport module of DELFT2D, were not successful. The sediment transport magnitude exploded locally in gyres where zero velocities were present, and a kind of volcano's grew in the bed. Therefore in this study the sediment transport is calculated using TRSTOT for total transport. The magnitude of the sediment transport due to the combined effects of waves...
and current is computed using the relation of Bijker. The transport rates are computed for
the time interval indicated by the module MAIN. In fact, because stationary hydrodynamics
are assumed, the interval can be a single moment. On any account the interval can be taken
small, while this reduces the times the flow field should be written to the communication
file.
In this study, spin-up effects will not occur in the time interval of the computation of the
flow field. These effects are prevented by the iterations of the hydrodynamic modules. So,
transport components are averaged over only a small time interval and no initial time
interval is specified.

8.2.1 Transport formulation of Bijker

The combination of waves and currents can give a high rate of sediment transport
(compared with only currents and only waves). In that case, the waves cause a large amount
of suspended sediment which the current transports.

The approach of Bijker was to introduce the wave influence via a modification of the
bottom shear stress in an existing sediment transport formula for currents. He used the
Kalinske-Frijlink formula for the bed load transport coupled to the Einstein expression for
suspended sediment transport. This resulted in an expression showing that the suspended
load is directly proportional to the bed load. For further details is referred to Van der Velden

8.2.2 Sediment Transport Magnitude

The magnitude of the sediment transport $S'$ computed by the sediment transport relations
mentioned above, can be corrected in DELFT2D-MOR for different effects:

- physical slope effect: $\alpha_s$
- numerical stability/accuracy: $\alpha_n$

The magnitude of the sediment transport can be expressed as:

$$ s = s' \alpha_s \alpha_n $$

(8.1)

Bed level gradient

Like in other transport relations, also Bijkers' transport formula does not contain the effect
of a bed level gradient. The effect of the bed level gradient can be simulated according to a
correction term $\alpha_s$:

$$ \alpha_s = 1 + \alpha_{bs} \frac{\partial z_b}{\partial S} $$

(8.2)

The derivative of the bed level has been taken in the direction of the transport. The
coefficient $\alpha_{bs}$ has to be specified in the transport input file by the parameter ALFABD.
It was varied with values 1.0, 5.0 and 10. No changes in morphology of the rip were
perceptible after several hours of simulating. This is explicable considering the slopes of
the bar face and the banks of the rip channel. The slope of the bar face is generally much
steeper than the banks of the initial rip channel. Therefore the correction will in particular act upon the bar face. But over there the sediment transport rates are rather small, so the effect of the coefficient is not pronounced. (These results are not recorded in the appendices).

Several formulations for numerical stability are implemented in DELFT2D-MOR. This feature brings about that the transport declines when the current is going upwards and increases, going down. Most of the runs in this project are executed with option number 6. (See the DELFT2D-MOR manual).

Besides variations in sediment transport magnitude coefficients, the effect of variation in the parameters, governing the direction of sediment transport is also examined. This resulted in even less fortunate simulations. By entering coefficients into the formula computing the direction of the sediment transport, a kind of bed level slope effect can be introduced. Because of the same reason as stated above, no improvements (speeding up of the morphological process) were observed. Rather spurious bottom levels where the outcome of these variations in transport direction.

### 8.3 Bottom Module

The determination of the bottom level changes is based on the conservation of (sediment) mass. The maximum time step $\Delta t$ for the execution of bottom updating follows from the Courant condition in the mean flow direction:

$$\sigma = \frac{c \Delta t}{\Delta x}$$

in which:
- $\sigma$ = Courant number (-)
- $c$ = bed celerity
- $\Delta t$ = time step (s)
- $\Delta x$ = mesh size (m)

The bed celerity $c$ can be seen as the propagation speed of bed-level disturbances. One can chose to use an automatic time step which satisfies a condition for this propagation speed. Applying a small value for the courant number increases the stability of the computations. Exceptional disturbances are prevented in this way, outcome is however a slowing down of the simulation time progress.

### 8.4 Time scale of bottom changes in simulations

The course of the rip development is not easily to describe by looking at a sedimentation/erosion rate at a certain grid point. For instance the development of the sill is mainly in horizontal direction where 'vertical' laminates are disposed. A location where heavy sedimentation occurs in the beginning can have zero bed level change at a later stage while the morphologic process is still in full swing. There is no function available to evaluate the morphodynamic change per time unit over the whole area.

It is expected that, given a flow field, the morphology will be adapted to a large extend to that flow after a period of time. Yet actually, there should be a feed back to the hydrodynamic computations in a much earlier stages. That necessity of calling the hydrodynamic modules once in a while causes that:
the course of the bottom changes is not gradually in time. After a new flow field is read from the communication file, first a rapid bottom change will occur after which the speed of the processes probably slows down until the next execution of the flow computations, and so on;

- the way of computing the morphodynamics will in any case introduce a kind of inertia to the system.

Although the continuity corrections do good, they will not prevent the given disadvantages totally. For instance, when the rip channel would erode and get deeper, velocities would decrease due to the continuity correction. These lower velocities do not benefit ongoing erosion.

There are, even at the longest simulations no clear signs that the rip development stops and whether an equilibrium is reached or not. Instead, there is ongoing accretion and erosion, yet on a slow rate.

### 8.5 Evaluation of the morphodynamic simulations

Morphodynamic simulations are those in which bottom changes are computed, followed by an up-dated water motion field. Findings as a result of the simulations are reported in this section, divided in the categories: model behaviour, hydrodynamic changes and morphological development.

#### Model behaviour

It turned out to be difficult to obtain an unconditional stable morphodynamic computation. The outcome of the computations is strongly dependent on:

- the structure of the computation;
- hydrodynamic feedback; the time elapsed after which the wave and flow modules are called again;
- number of continuity corrections;
- the question whether automatic bottom time steps are used and the resulting simulation period;
- the parameters of the transport module (slope effects, sediment transport magnitude, direction).

Frequent hydrodynamic feed back and/or small bottom time steps cause a more irregular sedimentation erosion pattern. Disturbances are exaggerated. Large time steps therefore give a smoother topography after some hours of simulating. This can be illustrated on the basis of the plots in appendix D 4 (comparison fbs and fps).

It is questioned whether it is legitimate to expect accurate results, using a number of continuity corrections. In fast developing circumstances, for instance, with short term morphological adaptations and relatively shallow waters, bottom changes are comparatively large. An optimum should be found, when using continuity corrections, between on the one hand large time steps and large morphological changes and on the other hand a small time step and irregularities.

### Hydrodynamic changes
Changes in hydrodynamic characteristics after morphodynamic computations are listed below:

- **Velocity.** The velocity in the rip channel increases after the morphological changes but drops abruptly in the rip head. This comes from the tendency of the current to split and flow on both sides around the top of the sill. This is confirmed by the discharge graph.(App D 4.2.4)
- **Wave Height.** The wave height in the rip profile remains larger because fewer waves break on the flattened second bar. The increase of the wave height due to the opposing current is less pronounced.
- **Wave dissipation** in the rip is less concentrated due to the milder slope at the sill. (see below for comparison with ARGUS video images)
- **Wave force.** The difference in wave force between rip and normal profile is less clear. This is due to a decreasing influence of the opposing current on breaking. The decrease of water depth over the sill, after the bottom changes, dominates the wave breaking even more compared to the initial situation;
- **Wave Set-up.** There is less set-up behind the offshore bar because fewer waves break. The water level slope on the rip axis is milder because of the shift in breaker location due to the sill. The differences in set-up in the trough remain about equal.
- **oblique incident waves.** Simulations with oblique incident waves look promising. Because the sill is to a smaller extend an obstruction to the rip current, the flow strengthens and turns more and more perpendicular to the shore. (compare appendix D 4.3.3 and D 2.2)

Explaining graphs can be found in appendix D 4.2

**Morphologic developments**

About the erosion/sedimentation in the domain the following remarks can be made:

- In all runs a large deposit arises at the rip head;
- Substantial sediment transport takes place at the second bar. The rip current, combined with the high sediment transport capacity of the waves over there, causes a flattening of that bar;
- Near the rip, the face of the nearshore bar erodes. This is caused by the current, flowing over the bar face to the rip;
- The shore side edge of the bar near the rip channel erodes;
- The rip channel deepens slightly;
- Erosion appears at the waterline close to (in) the rip channel;
- The widening of the rip channel is limited. The expected development from v-shape to deeper u-shape is not obvious. Probably this is caused by the flow contraction due to the increased depth;
- Long simulation periods (over 3 days) show that rip current obstructs itself through the sill in front. Two different scenarios can be distinguished:
Figure 8.1 Schematisation of morphological development for less frequent and frequent feedback

- frequent feedback of hydrodynamics: the current tends to split and flow around the sill (see run fp6 in appendix D 4.3.2)
- restricted feedback of hydrodynamics: the current digs a new channel through the sediment deposited in front of the bar cutting in an earlier stage (see run fb5 in appendix D4.3.3)

- Descriptions in literature (see chapter 2) mentioned a migration of rips. A strong development is absent, not to mention migration.
- Oblique approaching waves have a large impact on the initial bathymetry.
  - the rip channel becomes wider and deeper
  - because the nearshore trough silts up down-stream of the rip channel, the longshore current is obstructed and bends more offshore
  - the rip current experiences less obstruction of the sill. The sill is deposited down stream of the rip channel, so the flow strengthens.
  - like in the hydrodynamic simulations the width of the trough is decisive, a narrow trough does not result in a larger rip channel in contrast to a wide trough.

Graphs in appendix D 4.3 ground these observations

**Wave dissipation patterns compared with ARGUS images of rips**

The maximum of the wave dissipation above the nearshore bar shifts offshore due to the growth of the sill (23 m offshore). In the ARGUS intensity profile of Figure 2-7 the shift is about 31 m. The peak in wave dissipation at the water line as observed on the ARGUS images is not clearly present. In case of a deeper rip channel this peak on the water line will probably arise. The overall picture of the ARGUS intensity profile resembles any way, the dissipation pattern after morphodynamic simulation (see Appendix D 4.3.2).

Although the results resemble the expectations of rip development (like widening of the channel), the speed at which the rip develops is far less than expected. On the ARGUS images and from other observations it is known that fully developed rips can be generated within the period of time of half a day. The simulations with maximum accretion/sedimentation rates of 20 cm/day therefore lag behind this generation time.
9 Morphodynamics with addition of wave mass flux

The simulation attempts in the preceding chapters show the morphological development of a rip but not with the speed we anticipated. So further investigations were made. Considering the model of Deigaard (1990), with the imposed discharge over the bar, it was expected that processing wave mass fluxes in the computation would strengthen the rip current and accelerate its development.

Section 4.4 gives an description of the most important equations involved in computing hydrodynamics with addition of wave mass fluxes. This chapter gives a survey of results of simulations in DELFT2D-MOR, including wave mass flux. First the effects on the hydrodynamic properties are discussed in section 9.1, next in section 9.2 the deviations of the morphodynamic calculations are treated.

9.1 Hydrodynamics with wave mass flux

The outcome of computations including mass flux are printed in appendix E 2. For the hydrodynamics the following can be observed in the charts and explained:

- In normal transects (or on a longshore uniform beach), mass flux addition results in undertow. Additional mass transported to the shore is returned in the vertical. $u_{rul}$ is directed offshore while $u_{com}$ (the equivalent velocity of both component parts: wave drift and flow) still has a value of about zero;
- The effect of mass flux on wave height is not worth mentioning;
- The mass flux it self is larger in the rip than on a normal profile. Because the wave heights (and consequently $E$) are the same. This is the result of a smaller phase velocity, which is caused by the opposing current as explained in section 4.3;
- The water level elevation increases in the mass flux version. The additional mass, transported to the shore by the waves, 'wants' to return to the sea, either as undertow or as horizontal circulation. It experiences bottom friction and because of this an extra gradient in the water level appears. This is the cause of the increase of the water level elevation. This effect arises both in the rip and in the normal profiles, so the set-up increases all over the surfzone, differences in set-up do not increase ergo the forcing remains the same;
- The flow pattern does not change a lot. The friction seems to play a dominant role here. It is true that the momentum equations are solved for $u_{rul}$ but the friction tensor is now computed with $u_{rul}$ as well. The overall result is that $U_{com mass flux}$ and $U_{com no mass flux}$ are about the same so the $U_{com}$ velocities in the rip do not increase, they even decrease;
- Velocity in the rip channel. $U_{com mass flux}$ in the rip channel is smaller than $U_{com no mass flux}$. In both cases, the forcing is the same as explained above, so the balancing force of the friction has to be the same as well. The only free quantity that can satisfy this requirement when the mass flux is added is $U_{com}$. And that is why $U_{com mass flux}$ is smaller than $U_{com no mass flux}$ in the rip channel;
- The wave dissipation at the nearshore bar is slightly lower for the simulation including mass flux. Because the wave height is not affected by the addition of the mass flux, this is the result of the current. Apparently, the wave velocity in the Doppler relation is not $U_{euler}$ but $U_{com}$. In fact $U_{euler}$ would be more appropriate to use.

### 9.2 Morphodynamics with mass flux

It turns out, as explained in the previous section, that the effect of including wave mass fluxes in the computation does not have a decisive effect on the flow through the initial rip. Like the bottom shear stresses are computed on the basis of $U_{euler}$, so are the transports. This, however, does give clearly different results. Velocities are larger and so are the transports. Since this is not a local effect, restricted to the rip channel, but taking place in the whole domain, the transports increase everywhere. Consequently, the initial profile, which is not a real equilibrium profile starts to resettle all along the shore. Comparing morphodynamic calculations with (fp5) and without (fp6) mass flux (and overlooking the overall longshore change due to Eulerian transports), indicates that adding mass fluxes does not give a more pronounced rip channel development. In appendix E.2 the simulation results are taken up.

However, morphodynamic calculations including the wave mass flux formulation will give more realistic results, provided that the major part of the initial topography is in equilibrium with the water movement. Besides it would be appropriate to use a suspended transport based formulation for the transports in order to compensate the additional transport of $U_{euler}$ by the sediment transportation capacity of the waves.
10 Conclusions and recommendations

10.1 Conclusions

This report shows that it is possible to simulate the morphological development of a rip current on an open beach, using a numerical model. Results of the computational simulations agree qualitatively with ARGUS images of a rip current. Rip currents are important for nearshore morphodynamics because of their ability to rearrange sediment in the nearshore zone.

The following conclusions can be drawn, as an answer on the questions stated in the introduction.

10.1.1 Wave-current interaction

A well settled wave-current field can be obtained in DELFT2D. The most straightforward feedback between wave and current field gives the best steady state hydrodynamics. This steady state can be reached, fairly fast, provided that small flow periods (6 min) are taken.

10.1.2 Mechanism of Rip current generation

In chapter 3, several theories about the generation of rip currents were described. This study does not add a new theory but on the basis of the executed simulations and ARGUS observations, some new insights are obtained.

Physical process

Rip currents are a phenomenon of hydrodynamic origin. This is suggested by strong regularity in spacing, as reported in literature, and the regular distance of intensity peaks on ARGUS images (Figure 2-6). Although the results of the model should be considered with some reservations, simulations show also regular longshore undulation of waves and current conditions (App D 1.3). On a longshore uniform barred beach, ‘circulation cells’ occur which are regular in spacing and dependent on the wave height and bottom friction.

The simulations confirm the observations that rip currents develop under moderate wave heights. The forcing of small waves is too small to get the flow going. High waves are accompanied by too much dissipation in the rip head. High energy conditions at that location weaken the current in the rip channel.

Rips do only evolve when waves approach under a small angle to the shore normal. Longshore currents easily wipe out the effect of a rip.

The initial wave-current field of a rip system has its repercussions on the morphology. In front of the rip channel a sill is deposited. The development of a sill is essential in rip morphodynamics. Changes in morphology at first stimulate rip development but slow it down in at a later stage.
A sill, downstream of the rip channel, first causes a strengthening of the rip current, because it increases differences in wave set-up. Later the strongly developed sill becomes an obstruction of the flow and brings the development of the current (under subsiding energy conditions) to an end.

Considerable erosion takes place at the second bar from the shore. This will assure the persistence of the rip.

A gentle longshore current stimulates the development of a rip system. The sill is to a lesser extend an obstruction. Siltation of the nearshore trough down stream of the rip channel strengthens the flow that breaks out.

**Model results**

The velocities in the initial rip, and after some time of morphodynamic simulations, are less than expected.

A wider nearshore trough amplifies the development of a rip system. In case of a widened trough, velocities in the rip channel increase. Moreover, in case of morphodynamical simulations with waves approaching under a small angle, the width of the trough turns out to be decisive for the development.

The importance of the value of the viscosity is on the area of stable numerical outcome, rather than rip development. Although a smaller viscosity strengthens the flow, it confines it as well. This is not for the benefit of widening the rip channel in morphological simulations.

The longshore length of what could be called the ‘circulation cell’ is largely determined by the friction. A high value for the friction enlarges confinement of streamlines because the current flows through the deepest area’s in the rip and feeder channels.

The effect of friction is important for the difference in outcome in case of applying wave mass fluxes.

The details of the changed bed level are very sensitive to the value of the morphological time step and the extend of hydrodynamic feedback.

According to the frequency of feedback from the morphological modules, different morphological developments can be observed. Fewer feedback causes the rip current to dig through the sill. Frequent hydrodynamic feed back results in splitting of the rip current and flowing on both sides around the rip.
Figure 10-1 Effect of way of modelling and stage of development on dissipation and set up in a rip system
The results of the computational simulations give insight in what sense the way of modelling a rip current and the stage of its development affect the dissipation pattern and the set-up. This is summarised in Figure 10-1.

10.1.3 ARGUS images

The simulated rip current, opposing the waves, causes an enhancement of the dissipation peaks rather than a shift of the dissipation maxima. The wave dissipation pattern as observed on ARGUS images of steep beach profiles, possibly with bars and/or sill, shows depth induced breaking rather then current induced breaking. The dissipation pattern resulting from morphodynamic simulations resembles white breaker bands on the ARGUS images in shape.

10.1.4 Addition of wave mass flux

Addition of wave mass fluxes is not essential for rip current simulations. Only in case of exaggerated friction, a difference in flow pattern is observed. Addition of wave mass fluxes does not result in a shift of the flow towards the rip channel from the bar face to the nearshore trough. Though, addition of wave mass fluxes gives more realistic results of the transport capacity in the simulated domain, provided that a transport relation for suspended sediment transport is applied.

10.2 Expectations

Considering that rips are likely generated by a hydrodynamic phenomena, (morphology is closely involved but does not trigger the generation of rip currents), I expect that the most valid theory for the generation of rip currents is that of instabilities in wave set-up.

The velocity in the rip is significantly determined by the value of the wave force in the rip head, that is to say, the local slope of the water level.

When the rip current is able to deepen the cannal to a larger extend, the development of the rip channel will be boosted. Using continuity corrections does not stimulate deepening of the rip channel while in a deeper channel the velocities decrease when the discharge is the same.

Introduction of a roller model will transport energy over the bar into the feeder channel. It is expected that such a formulation enhance the flow through feeder channels towards the rip.

Rip currents will probably be clearly recognisable on coasts with a mild slope of the shore face and a wide wave spectrum. Than there will always be wave which is breaking due to the opposing current.

Under obliquely approaching waves, there will be more dissipation at the waterline. The approaching waves experience less effect of the current.

Rips are likely to develop after a quiet wave period where bars have been growing, because pronounced bars are favourable for rip generation. With pronounced bars, the differences in wave set-up can arise.
10.3 Recommendations

Concluding this study, the following items are recommended (subdivided into four categories):

**Physical process of rip currents**

Advances in studying rip currents are possible in the field of:
- Deriving a ‘rip number’ comprehending the following parameters:
  - wave height
  - beach face slope
  - rip extension through the surfzone
  - hydraulic radius of the rip channel
  - longshore length of the distance from where the flow moves water towards the rip (in other words the spacing)
- Improve theories of instability mechanisms in the surfzone, both for the longshore current and for the set-up.

**Computational simulations of rip currents**

Advised is to direct further research on:
- The ‘circulation cells’ that occur when simulation with a longshore uniform barred profile is carried out. Are these a kind of edge waves or instabilities in set-up? What is their relationship with incident wave height? What happens when a morphodynamic calculation is made starting with a uniform beach.
- Optimising the angle of wave incidence. Find an optimum between absence of a longshore current and a longshore current that fades a rip channel away;
- The introduction of a roller model;
- Monitoring the bed level adaptations, in order to optimise the structure, the time steps and periods of the morphological computations;
- Using the suspended transport module, in stead of the bedload approximation;
- Involving a tide in the simulations. Velocities at falling tide could be important for morphological development.

**DELFT2D-MOR**

The following adaptations would enlarge the applicability of DELFT2D-MOR:
- Option for simulating with a wave forcing computed on the basis of radiation stresses instead of only dissipation gradients. This would strengthen the forcing, because firstly the gradients of the water level elevation increase (addition of set-down) and secondly the forcing due to transformation of waves that do not break is taken into account.
- Smoothing of transport capacities. Now, transport vectors explode, especially when computing with suspended transports, when a lot of wave activity is present at a certain location, while local ambient currents have zero velocities.
- A tool that quantifies overall morphological change, so that can be decided whether a new hydrodynamic computation is required yet or not.

**ARGUS program**
For the research in the framework of the ARGUS program it is interesting to study the two dimensional variations in intensity of the breaker zones. With special filters for intensity and contrast, differences can be made visible and can contribute to a better understanding of rip currents of other surfzone phenomena. Besides a quantification of the pixel intensity related to the hydrodynamics is required for further applicability of the video techniques. ARGUS video techniques will be useful for monitoring the effects of e.g. supplections on a coast. Conclusions of this study will be helpful, when interpreting the images in such a case.
Acknowledgements

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The cooperation with the following persons contributed greatly to the pleasure with which I worked on this thesis:
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References


A  Series of rectified ARGUS Images (Sep - Dec 1995)

During the Noordwijk '95 field campaign a rip developed at about x = -700 m. The rip was formed between September 27 and September 29. It persisted on the same location until the end of the year. The rectified images below show the rip during high wave conditions, which occurred during these months.

Rectified ARGUS image Sep.27 1995, GMT 15:15 hr

Rectified ARGUS image Sep.29 1995, GMT 10:14 hr
B Derivation of the Short-wave depth averaged Equations for free surface motion

In this section the wave induced terms from the equations describing the motion of a fluid will be derived. A Cartesian co-ordinate system will be used. As a starting point the full Navier Stokes equations are taken.

The Navier Stokes equations will be averaged first over the short wave period and subsequently over the water depth. This requires some integrations and definitions (section B 1) of the distinguishable velocities, before averaging the equations itself (section B 2 for mass conservation and B-3 for momentum conservation).

B.1 general fluid motion

The motion of a fluid is governed by the conservation laws of mass and momentum. The first of these is the conservation of mass,

\[ \frac{dp}{dt} + \rho \nabla \cdot \mathbf{u} = 0 \]  \hspace{1cm} (B.1)

If the density of a fluid \( \rho \) does not change, (B 2) simplifies to

\[ \rho \nabla \cdot \mathbf{u} = 0 \quad \text{or} \quad \nabla \cdot \mathbf{u} = 0 \]  \hspace{1cm} (B.2)

where \( \mathbf{u}(x,t) \) is the velocity vector \((U,V,W)\) and \( x = (x,y,z) \) with the z-axis pointing vertically upward. For convenience of vertical integration, the vertical and horizontal directions are distinguished. Specifically, the vertical velocity is denoted by \( w \) and the vertical co-ordinate by \( z \), the horizontal velocity components by \( u_i \) \((i = 1,2; u_1 = u, u_2 = v)\), and the horizontal co-ordinates by \( x_i \) \((i = 1,2; x_1 = x, x_2 = y)\).

\( \zeta(x,y,t) \) is the free surface displacement, and \( \zeta \) is its mean,

\[ \overline{\zeta} = \int_0^T \zeta dt \]  \hspace{1cm} (B.3)

\( T = \) short wave period
\( d = \) the still water depth

\[ h = d + \overline{\zeta} \]  \hspace{1cm} (B.4)

\( h = \) the mean water depth
Figure 3-1 definition of variables in the vertical direction

The momentum equations for incompressible flow read, in the horizontal directions:

\[
\rho \frac{\partial u_i}{\partial t} + \frac{\partial}{\partial x_j} \left( \rho u_i u_j + p \delta_{ij} \right) + \rho \frac{\partial}{\partial z} (u_i w) = 0
\]  \hspace{1cm} (B.5)

where i,j=1,2, p the pressure. \( \delta \) is the unit tensor (\( \delta_{ij} = 1 \) if j=i, and vanishes otherwise) and for the vertical momentum:

\[
\frac{\partial w}{\partial t} + \frac{\partial u_i w}{\partial x_i} + \frac{\partial w^2}{\partial z} + \frac{1}{\rho} \frac{\partial p}{\partial z} + g = 0
\]  \hspace{1cm} (B.6)

With the kinematic boundary conditions

\[
w = \frac{\partial \zeta}{\partial t} + u_j \frac{\partial \zeta}{\partial x_j} \quad \text{at} \quad z = \zeta(x,t)
\]

\[
w = -u_j \frac{\partial d}{\partial x_j} \quad \text{at} \quad z = -d(x)
\]

\[
p = 0 \quad \text{at} \quad z = \zeta(x,t)
\]  \hspace{1cm} (B.7)

The mean pressure, can be found by integration of the vertical momentum equation. Neglecting second order terms (hydrostatic approach), this gives:

\[
\bar{p} = g \int_{-d}^{\zeta} \rho dz = \rho g (\zeta + d) + p_{atm}
\]

\[
\frac{\partial \bar{p}}{\partial x_i} = \rho g \left( \frac{\partial \zeta}{\partial x_i} + \frac{\partial d}{\partial x_i} \right)
\]  \hspace{1cm} (B.8)
B.2 Averaged Equation of Mass Conservation

Equation B.2 can be written as:

$$\rho \frac{\partial u_i}{\partial x_i} + \rho \frac{\partial w}{\partial z} = 0$$  \hspace{1cm} (B.9)

Integrating this local continuity equation over the depth using Leibnitz's rule

$$\int_{a(x)}^{b(x)} \frac{\partial}{\partial x} f(x,z) dz = \frac{\partial}{\partial x} \int_{a(x)}^{b(x)} f(x,z) dz - f(x,b) \frac{\partial b}{\partial x} + f(x,a) \frac{\partial a}{\partial x}$$  \hspace{1cm} (B.10)

gives

$$\rho \int_{-d}^{\zeta} \frac{\partial u_i}{\partial x_i} dz + \rho \int_{-d}^{\zeta} \frac{\partial w}{\partial z} dz = 0$$  \hspace{1cm} (B.11)

$$\rho \int_{-d}^{\zeta} \frac{\partial u_i}{\partial x_i} dz + \rho \{[w]_{\zeta}-[w]_{-d}\} = 0$$

where the kinematic conditions at the surface and at z = -d are

$$[w]_{\zeta} = \frac{\partial \zeta}{\partial t} + \left[ u_i \frac{\partial \zeta}{\partial x_i} \right]_{\zeta}$$

$$[w]_{-d} = -\left[ u_i \frac{\partial d}{\partial x_i} \right]_{-d}$$  \hspace{1cm} (B.12)

because on the bottom the vertical velocity is zero. Inserting equation B.12 in B.9 gives

$$\frac{\partial}{\partial x_i} \int_{-d}^{\zeta} u_i dz - \left[ u_i \frac{\partial \zeta}{\partial x_i} \right]_{\zeta} - \left[ u_i \frac{\partial d}{\partial x_i} \right]_{-d} + [w]_{\zeta} - [w]_{-d} = 0$$  \hspace{1cm} (B.13)

resulting in the conservation of mass per unit area

$$\rho \frac{\partial}{\partial x_i} \int_{-d}^{\zeta} u_i dz + \rho \frac{\partial \zeta}{\partial t} = 0$$  \hspace{1cm} (B.14)
B.3 Averaged Equations of Momentum Conservation

Again applying Leibnitz rule on the horizontal momentum equation and integrating from bottom \(-h\) to the instantaneous free surface \(\zeta\), yields

\[
\rho \int_{-h}^{\zeta} \frac{\partial u_i}{\partial t} dz + \int_{-h}^{\zeta} \frac{\partial}{\partial x_j} (\rho u_i u_j + p \delta_{ij}) dz + \rho \int_{-h}^{\zeta} (u_i w) dz = 0
\]  \hspace{1cm} (B.15)

For each separate term the depth integrated form gives

\[
\int_{-d}^{\zeta} \frac{\partial u_i}{\partial t} dz = \frac{\partial}{\partial t} \int_{-d}^{\zeta} u_i dz - \left[ u_i \frac{\partial \zeta}{\partial t} \right]_{\zeta} \hspace{1cm} (B.16)
\]

\[
\int_{-d}^{\zeta} \frac{\partial}{\partial x_j} (u_i u_j) dz = \frac{\partial}{\partial x_j} \int_{-d}^{\zeta} u_i u_j dz - \left[ u_i u_j \frac{\partial \zeta}{\partial x_j} \right]_{\zeta} + \left[ u_i u_j \frac{\partial \delta}{\partial x_j} \right]_{-d} \hspace{1cm} (B.17)
\]

\[
\int_{-h}^{\zeta} \frac{1}{\rho} \frac{\partial}{\partial x_i} (p \delta_{ij}) dz = \frac{1}{\rho} \frac{\partial}{\partial x_i} \int_{-h}^{\zeta} p \delta_{ij} dz - \left[ \frac{p \delta_{ij} \partial \zeta}{\rho \partial x_i} \right]_{\zeta} + \left[ \frac{p \delta_{ij} \partial \delta}{\rho \partial x_i} \right]_{-d} \hspace{1cm} (B.18)
\]

\[
\int_{-d}^{\zeta} \frac{\partial}{\partial x_i} (u_i w) dz = \left[ u_i w \right]_{\zeta} - \left[ u_i w \right]_{-d} \hspace{1cm} (B.19)
\]

\[
= \left[ u_i \left( \frac{\partial \zeta}{\partial t} + u_i \frac{\partial \zeta}{\partial x_i} \right) \right]_{\zeta} - \left[ u_i \left( u_i \frac{\partial \delta}{\partial x_i} \right) \right]_{-d}
\]

\[
= \left[ u_i \frac{\partial \zeta}{\partial t} \right]_{\zeta} + \left[ u_i u_i \frac{\partial \zeta}{\partial x_i} \right]_{\zeta} - \left[ u_i u_i \frac{\partial \delta}{\partial x_i} \right]_{-d}
\]

substituting all these terms again in the depth-averaged momentum equation results in (because a lot of terms are crossed out against each other)

\[
\rho \frac{\partial}{\partial t} \int_{-d}^{\zeta} u_i dz + \frac{\partial}{\partial x_j} \int_{-d}^{\zeta} (\rho u_i u_j + p \delta_{ij}) dz - \rho p \frac{\partial \delta}{\partial x_i} = 0 \hspace{1cm} (B.20)
\]

The mean value of the hydrostatic pressure from the bottom to the mean surface can be substituted in B.20, making use of:

\[
\overline{p}_d = \rho g (d + \overline{\zeta})
\]

\[
\rho g (d + \overline{\zeta}) \frac{\partial \delta}{\partial x_i} = \frac{\partial}{\partial x_i} \left[ \frac{\rho g (d + \overline{\zeta})^2}{2} \right] - \rho g (d + \overline{\zeta}) \frac{\partial \overline{\zeta}}{\partial x_i} \hspace{1cm} (B.21)
\]
The term below the integral mark in B.21 is the radiation stress:

\[ S_y = \int_{-d}^{\zeta} \left( \rho \bar{u}_i \bar{u}_j + p \delta_{ij} \right) dz - \frac{1}{2} \rho gh^2 \delta_y \]  

(B.22)
C Wave-Current interaction

C.1 Overview of possible configurations
C.2 Wave Current interaction
C.1 Overview of possible configurations
DELFT2D

Wave - Current interaction
C.2 Wave Current interaction

C.2.1 Course of wave-current iterations with different configurations
C.2.2 Effect of computation with wave current interaction
C.2.1 Course of wave-current iterations with different configurations

1. direct feedback  1.6.1.30  (if5)
2. HISWA basin  1.6.30.20  (in7)
MORPHODYNAMIC RIP SIMULATION
hydrodynamics
course of wave-current iteration process (l. #: 1.30)

EPS wave force
EPS current magnitude

DELFT HYDRAULICS

if5 C2
MORPHODYNAMIC RIP SIMULATION
hydrodynamics (HISWA basin)
course of wave-current iteration process (1.9.30.20)

DELFT HYDRAULICS

EPS wave force
EPS current magnitude

in7  C2
C.2.2 Effect of computation with wave current interaction

1. effect on flow pattern
2. effect on wave height and wave dissipation and rip velocity
3. effect on wave force and wave set-up
4. effect of the rip current on the wave set-up profile
MORPHODYNAMIC RIP CURRENT SIMULATIONS
hydrodynamics
effect of wave-current interaction

DELFt HYDRAULICS
local increase in wave height due to opposing current

more wave dissipation in rip due to opposing current

wave dissipation only due to depth

small shift in rip compared to normal

deviating velocity

MORPHODYNAMIC RIP SIMULATIONS
hydrodynamics
effect of computation with and without wave-current interaction

DELFT HYDRAULICS

wave height
dissipation
MORPHODYNAMIC RIP SIMULATIONS
hydrodynamics
effect of computation with and without wave-current interaction

DELFT HYDRAULICS

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MORPHODYNAMIC RIP SIMULATIONS
hydrodynamics
effect of the current on the set-up profile in rip and normal transect

DELFT HYDRAULICS

set-up

dissipation

sn5 C 2.2.4
D DELFT2D-MOR Simulations

D.1 Domain related tests
D.2 Wave related tests
D.3 Flow related tests
D.4 Morphodynamic simulations
D.1  Domain related tests

D.1.1 non equidistant TRISULA grid
D.1.2 Topography
D.1.3 longshore uniform beach
MORPHODYNAMIC RIP CURRENT SIMULATIONS
hydrodynamics & morphodynamics
rectangular non-equidistant TRISULA grid

DELFt HYDRAULICS
D.1.4 Topography

1. initial bathymetry with central rip channel and wide trough
2. birds eye view of domain and rip channel
3. profiles with wide and narrow trough
4. effect of variation trough width on flow pattern
birdseye view of entire domain

Figure D - 1 birds eye view of rip channel
MORPHODYNAMIC RIP SIMULATION

cross shore profile

wide and narrow trough

Delft Hydraulics
MORPHODYNAMIC RIP CURRENT SIMULATIONS
hydrodynamics
effect of variation trough width on flow pattern

DELT HYDRAULICS

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D.1.5  longshore uniform beach

1. course of computation on longshore uniform beach (pf5,ph5)
2. longshore uniform beach, flow pattern for wave height variation $H_s = 1.0$ m and $H_s = 1.5$ m
3. longshore uniform beach, flow pattern for wave height variation $H_s = 2.0$ m
4. longshore uniform beach, longshore variation in set-up
5. longshore uniform beach, longshore variation in set-up and wave height
both computations reach an almost steady state

MORPHODYNAMIC RIP CURRENT SIMULATION
hydrodynamics course of the computations
longshore uniform beach, longshore variation in set-up

DELFT HYDRAULICS
COMPUTATIONAL RIP CURRENT SIMULATION
hydrodynamics
longshore uniform beach

DELFT HYDRAULICS

pw1

ph5

wave height variation

Hs = 1.0 m  Hs = 1.5 m
Flow velocity

Hs = 2.0 m

pw2
smaller waterdepth results in larger amplitudes

wave height- and set-up variations have an equal longshore wave length

longshore wavelenght of dissipation and wave force is the same, values are different

MORPHODYNAMIC RIP CURRENT SIMULATION
hydrodynamics
longshore uniform beach, longshore variation in set-up, wave height

ph5 no mass flux
pf5 mass flux

DELFt HYDRAULICS
addition of wave mass flux results in larger set-up, longshore wave length remains the same
D.2 Waves related tests

D.2.1 Incoming Wave Height variations

D.2.2 angle of wave incidence
D.2.1 Incoming Wave Height variations

effect on:
velocity vector field
$U_{rip}(x)$
$V_{normal}(x)$
dissipation
wave forcing
wave set-up
current refraction
MORPHODYNAMIC RIP CURRENT SIMULATIONS
hydrodynamics
vector velocity field for various wave height

DELFT HYDRAULICS

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MORPHODYNAMIC RIP SIMULATION
hydrodynamics
effect of waves height variations on velocity

DELFT HYDRAULICS

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MORPHODYNAMIC RIP SIMULATIONS
hydrodynamics
effect of variations in wave height on wave force and set-up

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DELFt HYDRAULICS
MORPHODYNAMIC RIP SIMULATION
hydrodynamics
effect of waves height variations on wave dissipation and height

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DELFt HYDRAULICS
MORPHODYNAMIC RIP CURRENT SIMULATIONS
hydrodynamics
effect of current refraction (hardly visible)

DELFt HYDRAULICS

nf6

D 2.1
D.2.2 angle of wave incidence

effect on:

velocity vector field with narrow trough
velocity vector field with wide trough
MORPHODYNAMIC RIP CURRENT SIMULATIONS
hydrodynamics
oblique approaching waves with wide trough

DELFt HYDRAULICS
angle 10 dgr
different angles of incidence
angle 5 dgr D 2.2
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<td>oblique approaching wave with narrow trough</td>
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<td>DELFT HYDRAULICS</td>
<td>D 2</td>
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MORPHODYNAMIC RIP CURRENT SIMULATIONS
hydrodynamics
oblique approaching wave with narrow trough

DELFt HYDRAULICS

ho1
angle = 10 degr
D2
D.3  Flow related tests

D.3.1  variation of friction coefficient

D.3.2  variation in viscosity
D.3.1 variation of friction coefficient

effect on:
  velocity vector field
  $U_{ri}(x)$ and shear stress
  $V_{normal}(x)$ and shear stress
  dissipation
  wave set-up
MORPHODYNAMIC RIP CURRENT SIMULATIONS
hydrodynamics
effect of variation bottom friction on velocity field

Delft Hydraulics
MORPHODYNAMIC RIP SIMULATIONS
hydrodynamics
effect of bottom shear stress coefficient variations

DELF T HYDRAULICS

velocity, shearstress

depth

sh* D 3
increase in difference in set-up due to increase in friction

decrease in dissipation due to decrease in velocity, while no clear shift of peak value is visible

MORPHODYNAMIC RIP SIMULATIONS
hydrodynamics
effect of bottom shear stress coefficient variations

Delft Hydraulics
D.3.2 variation in viscosity

effect on:
  vector velocity field $U_{\text{rip}}(x)$
  $V_{\text{normal}}(x)$
  wave dissipation
  wave force
  wave set-up
  wave height
MORPHODYNAMIC RIP MODELLING
hydrodynamics
effect of variation of viscosity on current field

<table>
<thead>
<tr>
<th>v (m/s)</th>
<th>vi*</th>
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<tr>
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<tr>
<td>0.1</td>
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</table>
MORPHODYNAMIC RIP SIMULATION
hydrodynamics
effect of viscosity variations on velocity in rip and normal transect

<table>
<thead>
<tr>
<th>v</th>
<th>3.0 m2/s</th>
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</thead>
<tbody>
<tr>
<td>v</td>
<td>1.0 m2/s</td>
</tr>
<tr>
<td>v</td>
<td>0.8 m2/s</td>
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DELFt HYDRAULICS

D3.2
MORPHODYNAMIC RIP SIMULATION
hydrodynamics
effect of viscosity variations on wave forcing, dissipation and wave height

DELFT HYDRAULICS

<table>
<thead>
<tr>
<th>v</th>
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MORPHODYNAMIC RIP SIMULATION
hydrodynamics
effect of viscosity variations on wave dissipation and wave set-up

\[ v = 3.0 \text{ m}^2/\text{s} \]
\[ v = 1.0 \text{ m}^2/\text{s} \]
\[ v = 0.8 \text{ m}^2/\text{s} \]
D.4 Morphodynamic simulations

D.4.1 Course of a morphodynamic computation

D.4.2 Hydrodynamic changes after morphological development

D.4.3 Changed morphology

D.4.3.1 run with 2 times hydrodynamic feed back in large interval (fb5)
D.4.3.2 run with 2 times hydrodynamic feed back in smaller interval (fp6)
D.4.3.3 morphodynamics with oblique incident waves
D.4.1 Course of a morphodynamic computation
MORPHODYNAMIC RIP SIMULATIONS
morphodynamics
course of the computation

MORPHODYNAMIC RIP SIMULATIONS
morphodynamics
course of the computation

very frequent hydronamic feedback, small morphological timestep
(total simulation time less than 10 hours)

hydrodynamics not able to stabilize, unacceptable bottom. (larger morphodynamic time steps)

fb2: regular process
fb6.2: unstable process

DELFt HYDRAULICS
D.4.2 Hydrodynamic changes after morphological development

effect on:
- dissipation
- wave force
- wave height
- wave set-up
- $U_{ni}(x)$
MORPHODYNAMIC RIP SIMULATION
morphodynamics

shift due to sill
\pm 23 \text{ m}

difference with
dissipation
profile due to
opposing current

wave growth due to
opposing current

due to flattening
second bar
less wave
growth

dissipation
wave height

DELFt HYDRAULICS
MORPHODYNAMIC RIP SIMULATION
morphodynamics
differences in rip after 40 hrs sim.

DELFt HYDRAULICS
MORPHODYNAMIC RIP SIMULATION
morphodynamics
differences in rip after 40 hrs sim.

DELFT HYDRAULICS
D.4.3 Changed morphology

D.4.3.1 run with 2 times hydrodynamic feed back in large interval (fb5)

D.4.3.2 run with 2 times hydrodynamic feed back in smaller interval (fp6)

D.4.3.3 morphodynamics with oblique incident waves
D.4.3.1 run with 2 times hydrodynamic feed back in large interval (fb5)

- velocity vector field
- transport vector field
- longshore depth profiles
- cross shore depth profiles
- bottom change after 147 hrs
- dissipation map
Flow velocity

\[ \rightarrow 0.200 \text{m/s} \]

MORPHODYNAMIC RIP CURRENT SIMULATIONS
morphodynamics
current velocity over bottom after 80 hours

DELFt HYDRAULICS
MORPHODYNAMIC RIP CURRENT SIMULATIONS
morphodynamics
transport over bottom after 147 hours

DELFt HYDRAULICS
MORPHODYNAMIC RIP SIMULATIONS
morphodynamics
longshore transects at x=664 m, x=679 m, x=699 m, x=739 m.

DELFt HYDRAULICS

large interval between feedback
MORPHODYNAMIC RIP SIMULATION
morphodynamics
crossshore transects at y=928 m, y=918 m, y=878 m, y=688 m.

DELFt HYDRAULICS

large interval between feedback
automatic timestep
MORPHODYNAMIC RIP CURRENT SIMULATIONS
morphodynamics
bottom change after 147 hrs

DELFt HYDRAULICS

fb5

large interval between feedback

autunm, timet

D 4.3.1
D.4.3.2 run with 2 times hydrodynamic feedback in smaller interval (fp6)

velocity vector field
transport vector field
longshore depth profiles
cross shore depth profiles
bottom change after 60 hrs
dissipation map
Flow velocity

> 0.500 m/s

MORPHODYNAMIC RIP CURRENT SIMULATIONS
morphodynamics
current velocity over bottom after 40 hrs

DELFt HYDRAULICS

fp6  no mass flux
small interval between feedback
fixed timest.  D 4
MORPHODYNAMIC RIP CURRENT SIMULATIONS
morphodynamics
transport over bottom after 60 hrs

DELFt HYDRAULICS

fp6
no mass flux
small interval between feedback
fixed timestep
D 4
MORPHODYNAMIC RIP SIMULATIONS
morphodynamics
longshore transects at x=664 m, x=679 m, x=699 m, x=739 m.

DELFt HYDRAULICS

fp6
no mass flux

small interval between feedback
fixed time step
MORPHODYNAMIC RIP SIMULATIONS
morphodynamics
crossshore transects at y = 928 m, y = 918 m, y = 878 m, y = 688 m.

DELFt HYDRAULICS
fp6
small interval between feedback
fixed timest. D 4
MORPHODYNAMIC RIP CURRENT SIMULATION
morphodynamics
dissipation with bottom after 40 hrs

DELFt HYDRAULICS

fp6
no mass flux
small interval between feedback
fixed timest.
D 4.3.2
MORPHODYNAMIC RIP CURRENT SIMULATIONS
morphodynamics
bottom change after 60 hrs

DELFT HYDRAULICS
D.4.3.3 morphodynamics with oblique incident waves

velocity vector field
transport vector field
longshore depth profiles
cross shore depth profiles
bottom change after 60 hrs
dissipation map on bottom after 40 hrs
MORPHODYNAMIC RIP CURRENT SIMULATIONS
morphodynamics
waves approaching under angle of 5 dgr.

DELFT HYDRAULICS
MORPHODYNAMIC RIP SIMULATION
morphodynamics
longshore transacts at x=664 m, x=679 m, x=699 m, x=739 m.

DELF Hydraulics
E Wave Mass Flux

E.1 Wave energy equation including mass flux
E.2 Hydrodynamic simulations with mass flux
E.3 Morphodynamic simulations with mass flux
E.1 Wave energy equation including mass flux

An equation that describes the balance of energy is found to be (Phillips 1977):

$$\frac{\partial}{\partial t} \left\{ \frac{1}{2} \rho u_i^2 + \rho gz \right\} + \nabla \cdot \left\{ u_i \left( p + \frac{1}{2} \rho u_i^2 + \rho gz \right) \right\} - p \nabla \cdot u_i = u_i \cdot f \quad (D.1)$$

This is the scalar product of $u$ with the basic momentum equation.

An equation for the balance of vertical mean total energy can be found by integration of (E 1) over the depth of the water. In case of small bottom slopes and overlooking the external forces this yields (Dingemans 1997):

$$\frac{\partial}{\partial t} \int_{-d}^{z} \rho \left( \frac{1}{2} u_i^2 + gz \right) dz + \frac{\partial}{\partial x_i} \int_{-d}^{z} \left( \frac{1}{2} \rho u_i^2 + \rho gz + p \right) dz = 0 \quad (D.2)$$

Now the integrated energy equation is time averaged. To that end the motion is written as the sum of the mean motion and the residual motion: $u_i = \bar{U}_i + \tilde{u}_i$. The average energy $E$ can be split up in a contribution due to the residual motion alone, $\bar{E}$, and one due to the effect of the mean motion.

Phillips shows that the substitution of time averaged finally results in

$$\frac{\partial}{\partial t} \left\{ \frac{1}{2} \bar{U}_i \bar{m}_i + \frac{1}{2} \rho g (h^2 - d^2) - \frac{\bar{M}_i^2}{2 \rho h} + E \right\} + \frac{\partial}{\partial x_i} \left\{ \bar{m}_i \left( \frac{1}{2} \bar{U}_i^2 + gh \right) - \frac{1}{2} \bar{u}_i \bar{M}_i^2 \rho h + U_i E + F_i + U_j S_{ij} \right\} = -\varepsilon + U_i \tau_i \quad (D.3)$$

$F = Ec_g$, and represents the mean energy flux by the fluctuating motion alone. The energy equation includes a number of terms describing the interaction between the short-wave motion and the currents. The first part between braces represents the mean energy density. It is the sum of terms describing:

1. the kinetic potential energy density in an ‘equivalent’ uniform stream with the same depth and mass flux as the actual motion
2. the potential energy density in an ‘equivalent’ uniform stream with the same depth and mass flux as the actual motion
3. the difference of the energy density of the actual mean motion and the ‘equivalent’ one
4. the energy density of the fluctuating motion

The mean energy flux is given as (second part between braces)

1. the energy flux of the ‘equivalent’ uniform stream with the same depth and mass flux as the actual motion
2. the difference of the energy flux of the actual mean motion and the ‘equivalent’ one
3. the convection of fluctuating energy by the mean flow
4. the energy flux by the fluctuating motion
5. the energy flux by the excess momentum flux

The right hand side of this equation represents the loses.

\( \varepsilon = \text{rate of energy dissipation per unit area} \)

\( U_j T_j = \text{rate of dissipation due to turbulence generated by the bottom shear stress} \)

The equations of the energy balance can be split in the energy balance of the fluctuating motion:

\[
\frac{\partial}{\partial t} \left\{ E - \frac{\tilde{M}_j^2}{2 \rho h} \right\} + \frac{\partial}{\partial x_i} \left\{ U_{j,i} E + F_i - \tilde{u}_i \frac{\tilde{M}_j^2}{2 \rho h} \right\} + S_{y,j} \frac{\partial U_j}{\partial x_i} = -\varepsilon \quad (D.4)
\]

\[
\frac{\partial E}{\partial t} + \frac{\partial}{\partial x_i} \left\{ U_{m,i} E + \tilde{F}_i + U_{m,j} S_{y,j} \right\} - S_{y,j} \frac{\partial U_{m,j}}{\partial x_i} = -\varepsilon \quad (D.5)
\]

and the energy balance of the mean stream

\[
\frac{\partial}{\partial t} \left\{ \frac{1}{2} \tilde{u}_i m_i + \frac{1}{2} \rho g (h^2 - d^2) \right\} + \frac{\partial}{\partial x_i} \left\{ m_i \left( \frac{1}{2} \tilde{u}_j^2 + gh \right) + U_j \frac{\partial S_{y,j}}{\partial x_i} \right\} = U_j T_j \quad (D.6)
\]

If \((H/h)^2 \ll 1\) (deep water), all the mass flux related terms \((\tilde{M})\) in the energy balance of the fluctuating motion can be neglected. As the mean energy flux by the fluctuating motion alone, \(F = c_E E\), one gets

\[
\frac{\partial E}{\partial t} + \frac{\partial}{\partial x_i} \left\{ E \left[ U_i + (c_E) \right] \right\} + S_{y,j} \frac{\partial U_j}{\partial x_i} = -\varepsilon \quad (D.7)
\]

Where \(c_E\) is the propagation velocity of the wave energy. But in more shallow waters the contribution of the wave mass flux gets increasingly important.

Dissipation is calculated by means of relations substituted in the source term of the energy equations. Both losses due to bottom shear stresses as models for shallow-water breaking waves are applied to solve the dissipation.
E.2 Hydrodynamic simulations with mass flux

1. velocity vectors, difference between $U_{eler}$ and $U_{com}$
2. effect of addition wave mass flux on cross shore velocity
3. cross shore profile of wave mass flux value
   effect of addition wave mass flux on longshore velocity
4. effect of addition wave mass flux on wave dissipation, wave height and set-up
MORPHODYNAMIC RIP CURRENT SIMULATIONS
hydrodynamics
wave mass flux, difference $U_{euler}$ and $U_{com}$

DELFT HYDRAULICS
MORPHODYNAMIC RIP SIMULATIONS
hydrodynamics
effect of addition wave mass flux on cross shore velocity

DELT HYDRAULICS
with decreasing friction, the current strengthens in the rip, and decreases the phase velocity in the rip head. The energy flux from offshore is the same, so the mass flux increases.

MORPHODYNAMIC RIP SIMULATIONS
hydrodynamics
effect of addition wave mass flux

DELFTHYDRAULICS

mass flux
longshore velocity

sh5,sn5 E 2.3
dissipation a little less, while dissipation on normal profile the same, so due to decreased U com in rip

addition of mass flux has no effect on wave height in rip

addition of mass flux increases wave set-up, in rip and normal profile with the same amount

MORPHODYNAMIC RIP SIMULATIONS
hydrodynamics
effect of addition wave mass flux
E.3 Morphodynamic simulations with mass flux

velocity vector field
transport vector field
longshore depth profiles
cross shore depth profiles
bottom change after 60 hrs
MORPHODYNAMIC RIP CURRENT SIMULATIONS
morphodynamics
current velocity over bottom after 40 hrs

DELFt HYDRAULICS

fp5
wave mass flux included
fixed timst. E 3
MORPHODYNAMIC RIP CURRENT SIMULATIONS
morphodynamics
transport over bottom after 60 hrs

DELFt HYDRAULICS

fp5
wave mass flux included
fixed time
E 3
MORPHODYNAMIC RIP CURRENT SIMULATIONS
morphodynamics
longshore transects at x=664 m, x=679 m, x=599 m, x=739 m.

DELFt HYDRAULICS
MORPHODYNAMIC RIP CURRENT SIMULATIONS
morphodynamics
crossshore transects at y=928 m, y=918 m, y=878 m, y=688 m.

DELFT HYDRAULICS

fp5
wave mass flux included
fixed timest.  E 3
erosion/sedimentation [m]

- ABOVE: .24
- .20: .24
- .16: .20
- .12: .16
- .08: .12
- .04: .08
- - .04: .04
- - .08: .08
- - .12: .12
- - .16: .16
- - .20: .20
- - .24: .24

longshore distance [m]

- 600
- 650
- 700
- 750
- 800
- 850
- 900
- 950
- 1000
- 1050
- 1100
- 1150
- 1200

cross shore distance [m]

MORPHODYNAMIC RIP CURRENT SIMULATIONS
morphodynamics
bottom change after after 60 hrs

DELFT HYDRAULICS

fp5
fix.bmp t= 300
mass flux included
E 3