Unibest-TC Userguide

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September, 2000

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1 Introduction

1.1 General

Until recently good modelling practice was associated more with having an in-depth knowledge of the implemented theories in a model rather than the art of applying a model as a tool to obtain reliable answers or solutions for a specific question or problem. As numerical modelling has become one of the corner stones of coastal engineering, a wide range of numerical models is applied by users of which many only have a limited understanding of the implemented theories.

This does not have to be a problem as long as users are aware of the limitations of applied the models and have a basic understanding of the implemented theories. However, theoretical knowledge alone is not sufficient for a successful numerical model study. Application of a modelling methodology in which all the necessary phases are explicitly included and can be verified objectively is essential. Too often, studies have resulted in incorrect or incomplete advises due to application of a model which has not been calibrated and validated correctly.

This user guide for Unibest-TC provides a general framework which helps a novice user to apply the Unibest-TC model sensibly. The existing user manual (WL | Delft Hydraulics, 1999) and technical reference manual (Bosboom et al., 1997) are very general and detailed, respectively. From many users there is a need of having a user guide which is on an intermediate level which provides detail where necessary (e.g. model parameters description) and gives general guidelines when setting up the model (e.g. boundary schematisation).

In Figure 1.1 two flow diagrams are given which identify all the phases in the described modelling methodology framework. The left diagram represents the ideal or theoretical set-up of a the different phases in a numerical modelling study. The different phases are distinct and are gone trough only once. However, the right diagram propably gives a more realistic image in which many of the phases are repeated a number of times. The different phases are not well-defined and especially the calibration and validation are often merged into one (often calibration) phase. This may be due to an inexperienced user but also can be caused by poor quality of the available data or unrealistic model outcomes which require reconsideration of some model settings.

A study according to the right plot is not a problem, but it is important the modeller is aware of the different phases. Otherwise, the danger exists that especially the essential calibration and validation phases are not carried out properly. Furthermore, the model's predictive capabilities can not be assessed objectively.

This userguide attempts to give an accurate description of all aspects of a numerical modelling study with Unibest-TC. This involves a description of the phases as they are shown in Figure 1.1. Moreover, all forcing schematisations (e.g. wave height, tide and bottom profile) and all relevant model parameters (e.g. maximum relative wave height, sediment characteristics) are explained in detail.

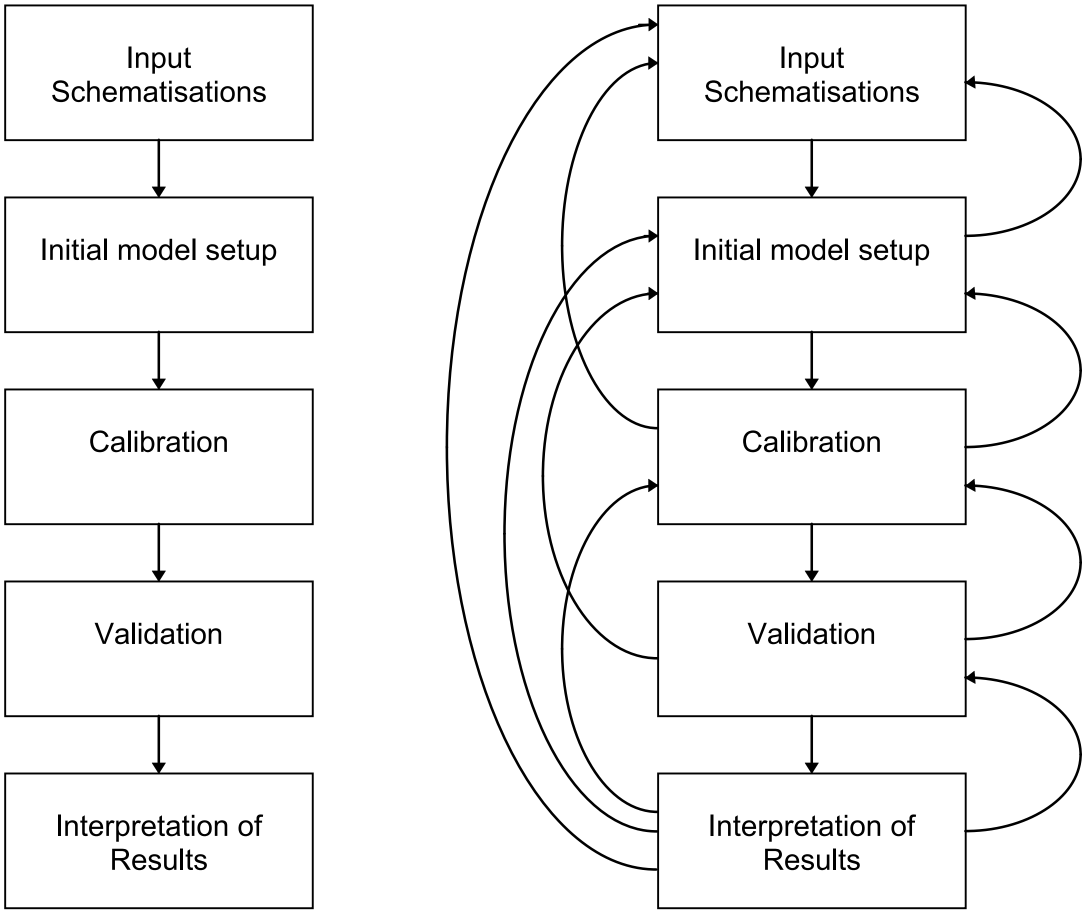


Figure 1.1 Flow diagrams of modelling methodology framework.: left diagram represents idealized case and right diagram the reality for a novice user.

1.2 How to read this user guide

Chapter 1: Introduction
The remaining part of this chapter contains general information on the capabilities of Unibest-TC and an overview of recent morphological studies carried out with the model.

Chapter 2: Input schematisations
Here, a detailed description is given of the various input schematisations. Attention is given to the hydrodynamic forcing and bottom profile schematisation methods.

Chapter 3: Model description
This chapter contains a general description of the Unibest-TC model and a detailed explanation of all the model and run parameters. Examples are given of the sensitivity of the model outcome for variation of relevant parameters. Furthermore, information is given how and if to use parameters during the calibration phase.

Chapter 4: Calibration of Unibest-TC

Chapter 4 is devoted to the calibration of the Unibest-TC model. It contains general information on how to carry out a calibration. Furthermore more specific information is given on how to calibrate the Unibest-TC model. Also, attention is given about some of the numerical aspects and how to interpret the observed deviations in terms of model performance, forcing errors, etc.

Chapter 5: Validation & Determination of predictive capabilities

In Chapter 5 the validation phase is described extensively. It gives some suggestions on how separate the calibration and validation phase. Furthermore, a description is given how to assess the predictive capabilities of a validated Unibest-TC model.

Chapter 6: Interpretation of model results

Finally, information and general guidelines are given on how to interpret the Unibest-TC model outcomes. Here, also reference is made to the benchmarking database for Unibest-TC is made (Roelvink et al., 2000). In this study a tool has been developed to evaluate the model performance objectively by means of a number of statistical parameters (Model Performance Statistics; MPS).

1.3 Capabilities of Unibest-TC

Unibest-TC is a module of the program package UNIBEST, which stands for UNIform BEach Sediment Transport. All modules of this package consider sediment transports along a sandy coast which locally may be considered uniform in alongshore direction.

Unibest-TC (TC: Time-dependent Cross-shore) is the cross-shore sediment transport module of the UNIBEST Coastal Software Package. It is designed to compute cross-shore sediment transports and the resulting profile changes along any coastal profile of arbitrary shape under the combined action of waves, longshore tidal currents and wind. The model allows for constant, periodic and time series of the hydrodynamic boundary conditions to be prescribed.

- The Unibest-TC model can be applied on several coastal problems, e.g.:
- Dynamics of cross-shore profiles.
- Cross-shore development due to seasonal variations of the incident wave field.
- Bar generation.
- To check the stability of beach nourishments and foreshore nourishments.
- To estimate the impact of sand extraction on the cross-shore bottom profile development.

In general Unibest-TC has been designed to study the morphological profile development on the time scale of storms (hours to days) and medium term (months to years). This includes bar migration and generation. However, the model can *not* be applied to explicitly study the morphodynamic behaviour of the shallow surf zone (water depths less then approximately 0.5 to 1 m) and swash zone. Changes in the dry part of the profile are taken into account by extrapolating the transport from the last wet computational point to the top of the dune.

1.4 Overview of recent morphological studies

Since the first release of Unibest-TC the model has been applied successfully on many cases in The Netherlands and abroad. Here an overview is give of recent application of the model which is largely based on Aarninkhof and Roelvink (1998).

Considering the physical model formulations incorporated, Unibest-TC is the most sophisticated morphological model of WL | Delft Hydraulics. Before being implemented in field models like Delft3D-MOR, new model concepts are tested first in the 'relatively simple' 1D Unibest-TC environment. Consequently, recent studies often concern the validation of the Unibest-TC model against extensive flume or field experiments.

The first study to be mentioned is Steetzel and Stive (1986), who used the Unibest-TC' predecessor CROSTRAN to assess model capability to predict morphological changes after closure of the Zeeland' tidal inlets. As long as cross-shore transports dominate the transports alongshore (say the first 10 years after closure), model results are in good correspondence with measurements. In the longer term, when 3D-phenomena become more important, model performance gets less satisfactory. The latter observation is confirmed by Ribberink and De Vroeg (1991), who attempted to apply Unibest-TC at the island of Texel, near a tidal inlet system. Steetzel and Stive (1986) also indicate that model results depend strongly on the availability of good-quality, carefully-selected boundary conditions, as confirmed by Bakker (1995).

The well-known NOURTEC data set acted as the starting point for numerous UNIBEST-TC-based studies. Reniers and Roelvink (1995) have calibrated the model for the Terschelling site, obtaining reasonable results, though the rapid growth of the inner bar in the beach nourishment case was not reproduced correctly. Based on the same case, a validation study by Roelvink (1995) mentioned a good representation of transport rates, despite the occurrence of local errors of the order of a factor 2 to 3. Formulations which take into account wave breaking delay and the slope effect are more effective in reproducing bar generation, though it is doubtful whether the same formulations and settings also hold for profiles with different bar and slope characteristics. In fact, this doubt is justified as demonstrated by Reniers and Roelvink (1996), where opposite settings for the slope effect are needed to obtain reasonable transport rates (local errors of order factor 2). Like Steetzel and Stive (1986), Reniers and Roelvink (1996) found alongshore variability to be a complicating factor in view of the application of Unibest-TC.

Practical, engineering applications of Unibest-TC can be found in Reniers and Roelvink (1997), where the model was used to design a sub-aqueous beach nourishment; moreover, the model was used in studies aiming at saving material by optimising the design of the profile of a land reclamation, either by cutting off the profile at the lower end (Onderwater, 1997) or by steepening the design profile (Bosboom et al, 1996). Steijn (1997) applied the Unibest-TC model to assess the nourishment needs along the Delfland and Goeree coast, caused by different Maasvlakte-2 variants. As a consequence of a not entirely satisfactory model calibration, results could only interpreted in relative sense, but valuable conclusions could still be drawn. Van Rijn et al. (1995) applied Unibest-TC to estimate transport rates across the -20 m and -8 m depth contour lines, concluding (among others) that the bound long wave induced offshore transport is somewhat overestimated, as well as the near-bed undertow. Recently, Unibest-TC was applied in combination with Delft3D in Boers and Walstra (1999) on the foreshore nourishment at Egmond, The Netherlands. In this study an extensive morphodynamic calibration and validation was carried out over periods of years. Followed by the prediction of the morphodynamic development of the foreshore nourishment over a period of five years. During the calibration and validation reasonable agreement with measured profiles (from the Jarkus database) was found. The predictive simulations seemed to indicate that the foreshore nourishment would break up in two bars which migrated shoreward and seaward respectively.

With the Coast3D EU-MaST-project (Soulsby, 1998) an extensive dataset has become available which is also used to validate Unibest-TC. In Kleinhout (2000) an extensive comparison with this field data is carried out. The general conclusion from this work is that agreement is reasonable but that significant deviations can still be observed. Especially the longshore and cross-shore currents show large deviations. However, it should be noted that the a rip was present in the considered transect which hampers a direct comparison and could be one of the main reasons for the observed deviations.

2 Input schematisations

2.1 Introduction

Here the input schematisation is defined as all the schematisations necessary to drive Unibest-TC. Two types of input schematisations can be distinguished:
1. the schematisations related to the hydrodynamic forcing (e.g. waves, tide and wind) and,
2. the schematisations related to the bed profile (e.g. bottom profile schematisation, sediment characteristics, bottom roughness).

In Table 2.1 an overview is given of the input data and its relative importance in practical applications. Note that this is only a general qualification, in specific applications of the software this can change (e.g. if Unibest-TC is used to derive longshore transports, the tidal components and wave direction are essential).

Input data		Remarks	Relative Importance
Hydrodynamic Forcing	Wave height	Time serie or constant	XXXX
	Wave period	Time serie or constant	XXXX
	Wave angle	Time serie or constant	XXX
	Water level (tide)	Time serie or constant	XX
	Longshore velocities (tide)	Time serie or constant	XX
	Wind speed/direction	Time serie or constant	X
Bed Schematisation	Bottom profile	Cross-shore profile	XXXX
	Sediment characteristics	constant or cross-shore varying	XX
	Longshore variability	can have implication on profile schematisation or interpretation of results	XXX

Table 2.1 Overview of input data for Unibest-TC and its relevance in practical applications.

Each type requires its own specific approach. In general, the applied hydrodynamic boundary conditions should result in a reliable description of the residual (e.g. yearly averaged) transports and profile development. Ideally one would like to apply the measured conditions. However, often these are not available or application of these type of boundary conditions can result in unacceptable simulation times. The schematisation of the bed profile is mainly determined by the longshore uniformity of the considered section of coast.

In this chapter both types of input schematisations are discussed. In Section 2.2, the boundary schematisations are treated. In Section 2.3, the available profile schematisations are explained.

2.2 Boundary schematisations

To arrive at a representative prediction of the morphology one would ideally apply the occurring conditions for waves, wind and tide. During the calibration phase (which is essentially a hindcast) this can done. In a forecast, this is obviously not possible. In this section a number of methods are discussed to arrive at a set of representative boundary conditions. The general concept is to make morphological forecasts based on representative forcing obtained from observed conditions. The approach to be followed is often determined by the availability and the quality of measured data. However, the type of coast can also have an influence. The Dutch coast can be classified as an average coast: the complete wave climate has to be considered. Other coasts (e.g. Israeli coast) can be classified as event driven: only (extreme) storm conditions have a significant effect on the coastal morphology. When applying a numerical model, it is essential that the user has some basic understanding of the coastal system in which the modelling exercise takes place.

One of the key elements in medium and long term modelling is reduction of information. This involves essentially four levels, which concern the input, the physical system or its model, the output and the interpretation or generalisation (cf. De Vriend et al., 1993). Here we are focussed on input reduction which is based on the idea that we can describe long term residual effects (e.g. water or sediment transport fields) with models based on the description of small scale processes, if we can find the representative inputs to drive them. In this context we may distinguish between tidal input, wave input and combined wave and tide input filtering approaches.

In the following sub-sections schematisation methods for the following cases are considered:

- sufficient measured data available (Section 2.2.1),
- insufficient measured data available (Section 2.2.2).

It is noted that the described schematisation methods usually refer to the schematisation of the wave conditions. In general the methods can be applied to other types as well. Section 2.4 is devoted to the schematisation of the tide.

2.2.1 Sufficient measured wave data available

Quality assessment of dataset

"When is the available data sufficient?", is the first question to be answered. It first of all depends on the period which is simulated. If a forecast of one or two years has to be made, measured data over a one year period might be insufficient because it might have been an exceptional calm or rough year. So the actual question to be answered is: "Is the available data representative for the simulation period?". Furthermore, there can be an incorrect temporal distribution of wave events in the time series which can results in unrealistic predictions which can cause chronology effects (see e.g. Aarninkhof et al., 1998).

The Unibest-TC model simulates the hydrodynamics and morphodynamics in cross-shore direction of the of the nearshore region. More specific, the model has been designed to simulate morphodynamic cross-shore behaviour of the in the surf zone. The model has not been designed to transform deep water waves over great distances to the nearshore region. In general, the length of model domain should not exceed 5 to 10 km. If the measurement location is too far offshore, the wave climate should be transformed to seaward boundary location of the model domain with the appropriate tool (e.g. with WL | Delft Hydraulics' Watron and Scatter models).

Based on the considerations above, the following steps/checks should be made when compiling boundary conditions for Unibest-TC:

- Is the measurement period representative for a long-term morphodynamic simulation?
- Compare measurement data with other climates in the same region.
- How is the temporal distribution of wave conditions in the dataset (e.g. seasonality effects or time series ordered on increasing wave height).
- At which location and at what depth has the data been collected. Transform wave climate to appropriate location if necessary with other specialised tools (e.g. Watron and Scatter).

Application of a binned wave climate

In most cases the wave climates is described by two tables in which the probability is given as a function of the wave height versus wave direction and of the wave height versus wave period respectively (see Table 2.2, for an example).

As waves in a certain area (in deep water) have a relative strong correlation (e.g. wave measurements along the Dutch coast), it is often a good procedure to check the representativeness of used boundary conditions with wave climates from other buoys in the same region. To make a reliable comparison of wave climates it should ideally be based on tables as shown here (Table 2.2 and Table 2.3).

As Unibest-TC uses time series as input, a binned wave climate has to be converted. To avoid any chronology effects it is essential not to simply build a time series on increasing wave height. It is strongly advised to randomise the sequence of the wave conditions in the time series. Furthermore, to avoid chronology effects the cycle length of the time series should be limited to one year (when simulating the morphology over a ten year period it is better to run through the time series 10 times in a loop rather then scaling the time up to ten years).

Reduction of number of wave conditions

Often Unibest-TC is applied to derive a strongly schematised wave climate (e.g. less then 10 wave conditions) which can be used as forcing for morphological area models (e.g. Delft3D-MOR). Such a reduced wave climate can also be an efficient way to run long term Unibest-TC simulations for two reasons:

- as the time of occurrence of each wave condition is relatively large, the morphological time step is not limited by the duration of the wave conditions,

- as there is a morphological scaling factor between the probability of occurrence of the wave conditions and the actual morphological time (e.g. by simulating 200 days of wave forcing, the actual time progress in the morphodynamic simulation may be one year).

	Wave direction (averaged value of considered class)																				
H_s	215	225	235	245	255	265	275	285	295	305	315	325	335	345	355	365	375	385	395	405	
0.25	0.54	0.20	0.29	0.36	0.35	0.45	0.43	0.55	0.85	0.79	0.69	0.83	0.66	0.81	3.15	1.15	0.72	0.60	0.38	1.00	14.8
0.75	1.69	1.00	1.41	1.33	1.25	1.38	1.16	1.10	1.20	1.16	1.11	1.14	1.31	1.65	3.69	2.22	1.72	1.45	0.94	2.45	30.3
1.25	1.54	1.62	2.13	1.82	1.00	0.79	0.64	0.55	0.61	0.61	0.65	0.73	0.97	1.14	1.92	1.63	1.16	1.00	0.64	1.66	22.8
1.75	1.18	1.46	1.84	1.38	0.57	0.49	0.47	0.42	0.40	0.41	0.45	0.39	0.68	0.75	0.78	0.52	0.43	0.54	0.38	0.79	14.3
2.25	0.64	1.13	1.38	0.74	0.35	0.28	0.25	0.27	0.33	0.26	0.22	0.25	0.46	0.41	0.33	0.23	0.18	0.19	0.15	0.37	8.43
2.75	0.30	0.72	0.76	0.41	0.23	0.18	0.20	0.17	0.16	0.12	0.16	0.18	0.27	0.18	0.16	0.12	0.05	0.09	0.07	0.11	4.63
3.25	0.12	0.35	0.44	0.21	0.12	0.11	0.12	0.13	0.11	0.08	0.08	0.06	0.17	0.10	0.06	0.07	0.04	0.05	0.07	0.04	2.53
3.75	0.06	0.15	0.17	0.12	0.07	0.08	0.07	0.09	0.06	0.04	0.04	0.06	0.06	0.08	0.04	0.02	0.02	0.02	0.02	0.00	1.28
4.25	0.01	0.04	0.04	0.02	0.02	0.03	0.05	0.06	0.06	0.03	0.02	0.03	0.02	0.02	0.01	0.01	0.00	0.01	0.00	0.00	0.47
4.75	0.00	0.01	0.02	0.02	0.01	0.01	0.02	0.03	0.03	0.02	0.01	0.01	0.04	0.01	0.00	0.00	0.00	0.02	0.00	0.00	0.28
5.25	0	0.00	0.01	0.00	0.01	0.00	0.01	0.01	0.01	0.00	0.00	0.00	0.01	0.01	0.01	0.00	0.00	0.00	0.00	0.00	0.07
5.75	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.01	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.02
6.25	0	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.01
6.75	0	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
TOT				31.6				10.6			10.7			13.4			33.4				100.

Table 2.2 Probability of occurrence of wave heights per direction at the Europlatform (1979-1993).

Below a method is given which enables the derivation of such a strongly schematised wave climate:

- To derive a representative set of wave boundary conditions the wave climate is discretisized into wave height and directional classes (the appropriate wave periods are derived from empirical relationships). This results in a table as shown in Table 2.2, in which for each combination of wave height and direction the probability of occurrence, *P(i)*, is listed:
- For each combination of wave height and direction the sediment transports (calculated with an appropriate sediment transport formula) are determined. This results in a table that looks like the table above but is filled with the resulting transports, *T(i)*.
- Next, the weighed transports are calculated by multiplying the transport from the previous step with the probability of occurrence: *S(i)=T(i)*P(i)*
- For each of the five applied directional sections (as indicated in the table above) the total representative transport is determined by summing up the weighed transports in each selected section: $S_{section} = \sum_{section} \left(T(i) * P(i)\right) = \sum_{section} S(i)$
- Determine the ratio, *R(i)*, between $S_{section}$ and the scaled transports (assuming the transport, *T(i),* has an occurrence equal to the probability of the total section: $\sum_{section} P(i)$) for each combination of wave height and direction: $R(i) = \dfrac{T(i) \sum_{Section} P(i)}{S_{section}}$.

- The procedure should be repeated on all the selected transport mechanisms/locations. If three transport mechanisms are considered this will result in three tables with the ratios, *R(i)*, for each combination of wave height and direction.
- By averaging the results of these (three) tables, a table results from which easily the combination of wave height and direction can be selected which has a ratio, $R_{avg}(i)$, closest to one.

By applying the described method on the listed wave climate in Table 2.2 the following schematised (morphological) wave climate results:

Condition	Direction (°N)	Hs (m)	Direction (°N)	wind (m/s)	Tp (s)	Weight
ZW	< 260	1,75	240	7,3	6,0	30,0
W	260-290	2,25	275	9,8	6,6	6,3
WNW	290-320	2,25	305	9,4	6,8	5,5
NNW	320-350	2,25	335	9,5	6,8	6,9
N	> 350	1,25	355	4,3	5,6	25,7
					Total	74,4

Table 2.3 Morphological wave climate.

In the described example only a sub-division in the wave directions is made, this can easily be extended by also selecting a number of wave classes per directional sector. This can be especially useful when explicitly calm and stormy conditions have to be taken into account. Note that the given example, in which a sub-division in wave direction is made, is aimed at describing a representative longshore transport. When cross-shore processes are the main interest, a sub-division in wave periods and wave heights is more appropriate.

When applying such a schematised wave climate it is essential to compare the schematisation against the complete climate.

2.2.2 Insufficient measured wave data available

When there is insufficient wave data available at the modelled location it is often an option to obtain wave data from other sources:

- wave data at another location but in the same coastal region,
- offshore wave data,
- hindcast wave data (e.g. WAM-models).

If a deep water wave climate is available in the same coastal region, a simple transformation to the sea boundary of Unibest-TC will suffice. If more than one climate is available, it is advised to perform a linear interpolation and transform the interpolated wave climate nearshore.

If only a local wave climates at another location are available, the standard procedure is to refract the local climates to deep water and interpolate the deep water climates and transform the interpolated wave climate nearshore.

In case there is no measured data available, an option may be to purchase a wave climate based on a hindcast study.

2.3 Area schematisations

As Unibest-TC is a so-called profile model it assumes that the modelled coastal section is longshore uniform. Any effects of resulting from longshore non-uniformities (e.g. rip currents) are not included in the model. Recent studies of the closed Dutch coast near Egmond have shown that coastal sections that appear to be uniform on larger spatial scales can have a strongly longshore non-uniform behaviour on smaller spatial scales (see Figure 2.1, where opposite bar migration was observed within 300 m longshore distance).

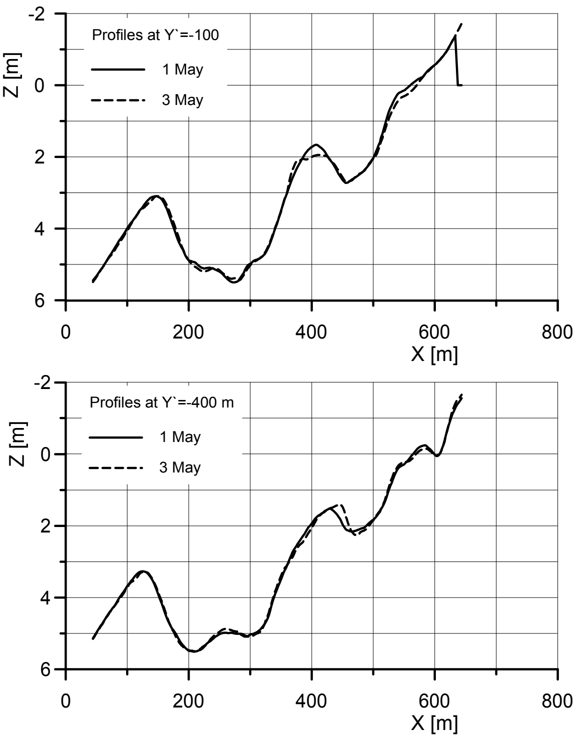


Figure 2.1 Measured profiles near Egmond (Walstra et al., 1999)

If the bathymetric data has a relative high spatial and temporal resolution effects as shown in Figure 2.1 can often be found. The application of only one cross-section in such a case is insufficient as it is not necessarily representative for the behaviour of the considered coastal section. In order to come to an representative profile simulation two approaches can be followed:

- perform simulations for a number of cross-sections or,
- construct a representative bottom profile by performing a longshore averaging of the bathymetry.

The second method is used in Kleinhout (2000) for the Coast3D Egmond site. The profiles at the different transects have to be shifted to eliminate the coastal curvature. For this shift a reference point is required which can be identified in all the considered profiles. In

Kleinhout (2000) different reference points were evaluated. It was found that for this specific case the water line gave the best results.

It is not possible to give general guidelines which method to apply. Morphodynamic simulations which use different starting bottom profiles can be used to get an indication of the sensitivity of the model outcomes to different starting profiles. The application of an averaged profile can give representative results. It is advised always to determine variance around the averaged profile and use this to construct “maximum” and “minimum” profiles by adding and subtracting the standard deviation from the averaged profile (see Figure 2.2 and Figure 2.3).

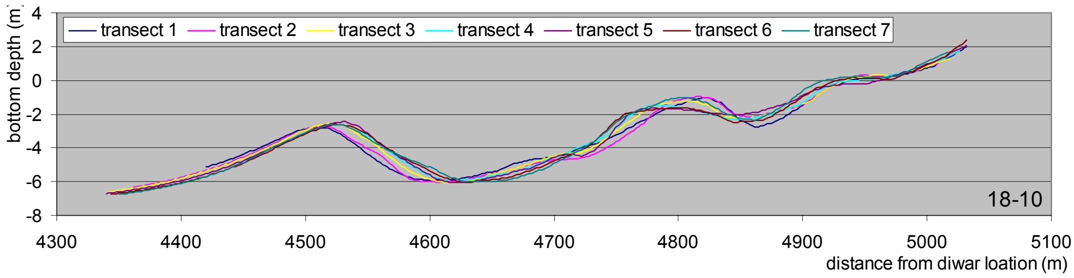


Figure 2.2 Profiles of seven transects with an longshore distance of 100m for 18-10-1998 (Kleinhout, 2000).

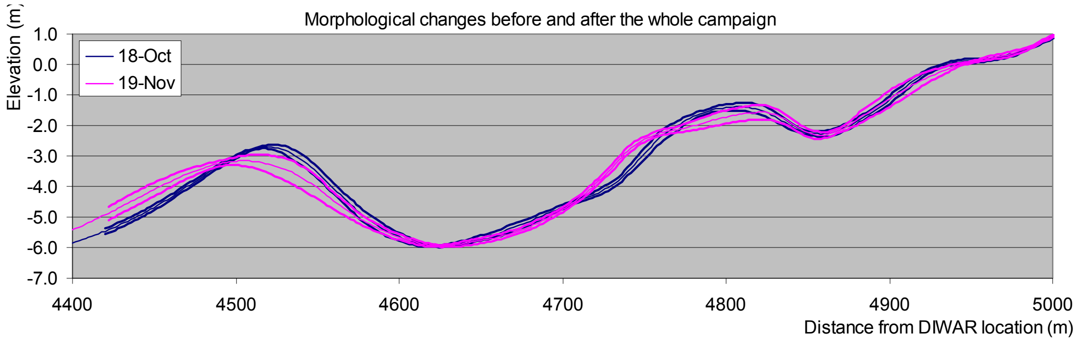


Figure 2.3 Morphological changes of the averaged beach profiles with variation ranges (Kleinhout, 2000).

Often, the bathymetric data consists of only one cross-section at a limited number of times. It is therefore not possible to investigate the sensitivity of the model to various starting bottom profiles based on measurements. However, the user always has to be aware of the effects these types of uncertainties can have on the model outcomes. An alternative may be to artificially modify the starting profile by e.g. increasing or decreasing the bar height.

2.4 Tidal schematisation

The accuracy of the tidal schematisation mainly depends on how Unibest-TC is applied. There is no need for a detailed description of the longshore tidal currents if the cross-shore profile development is the main interest. However, if an accurate description of the (residual) longshore transports is required the tidal schematisation is important.

2.4.1 Tidal schematisation for cross-shore modelling

Usually an average tidal water level variation can be applied. The tidal schematisation is of secondary importance if mainly the cross-shore behaviour is of interest. In these cases one can suffice by applying a tide (i.e. water levels) which is approximately 10% larger than the average. To simulate the cross-shore behaviour, it is important to prescribe a vertical tide. This is will reduce the errors in the shallow inner surf zone as the shoreward boundary of Unibest-TC varies.

2.4.2 Tidal schematisation for longshore modelling

If the longshore transports are of interest, an accurate description of the longshore tidal current is essential. Here, only a general description is given of the available schematisation methods, a more detailed can be found in Appendix A.

If time series of waves and tide are used no tide schematisation is necessary as the complete tidal cycle is included (providing the sufficient length of the time series). However, when randomised wave conditions are used (see Section 2.2.1) this becomes increasingly difficult. Usually each wave condition is simulated a number of times with different tidal conditions to obtain a representative transport. Ideally one would like to simulate a complete tidal cycle (e.g. neap-spring tidal cycle). However this would lead an unacceptable high computational effort.

Ideally, a period (i.e. morphological tidal cycle) has to be selected which gives an optimal representation of the residual transports if a complete (neap-spring) tidal cycle would be simulated. Only when detailed longshore transports are required, a more detailed schematisation is necessary. In Appendix A, two methods are described to derive such a morphological tidal cycle. The first method is described in Van Rijn (1993), the second method was developed by Latteux (1995).

Note that the longshore tidal velocities should be prescribed well outside breaker zone.

3 Model description

3.1 Introduction

This chapter is focussed on a comprehensive description of the many model input parameters of Unibest-TC. To that end, a conceptual description is given of the model, viz.: overall model consists of several sub-modules which describe the main constituents that govern the cross-shore morphology in the surf zone. To give the user a good insight in the effects and impacts of the input parameters, these will be grouped according to the physics that they aim to describe.

The detailed description of the model parameters comprises:

- default values and valid ranges,
- relation to physical process,
- how well is it established,
- sensitivity of the model predictions for variations in input parameter,
- relevance of parameter for calibration purposes.

Where applicable reference will be made to the Technical Reference Manual (Bosboom et al., 1997). Careful study of the Technical Reference Manual is strongly recommended in order to obtain good insight into the theoretical aspects regarding the Unibest-TC model.

3.2 Model concept of Unibest-TC

Unibest-TC fully integrates the effects of waves, tidal and wind driven currents and sediment transport on the morphological profile development. It has been designed to simulate the morphodynamic behaviour of the nearshore coastal regions including the surf zone due to the complex interactions between waves, currents, sediment transport and bathymetry. Each of these processes are dealt with in separate modules. In Figure 3.1 the model concept of Unibest-TC is shown. Four modules (Waves, Flow, Transport and Bottom) are executed subsequently. Each module uses the outcomes of the higher modules as forcing conditions (e.g. Flow module uses wave dissipation due to breaking as a driving force). This is only a general overview, some of the modules actually consist of sub-modules, see Bosboom et al. (1997) where a more detailed overview is given.

Each module makes a forecast of the appropriate constituent(s) based on the applied forcing and user specified model and run parameters. The number of model and run parameters is relatively high as Unibest-TC has many physical processes included. The following sections contain a detailed description of these parameters.

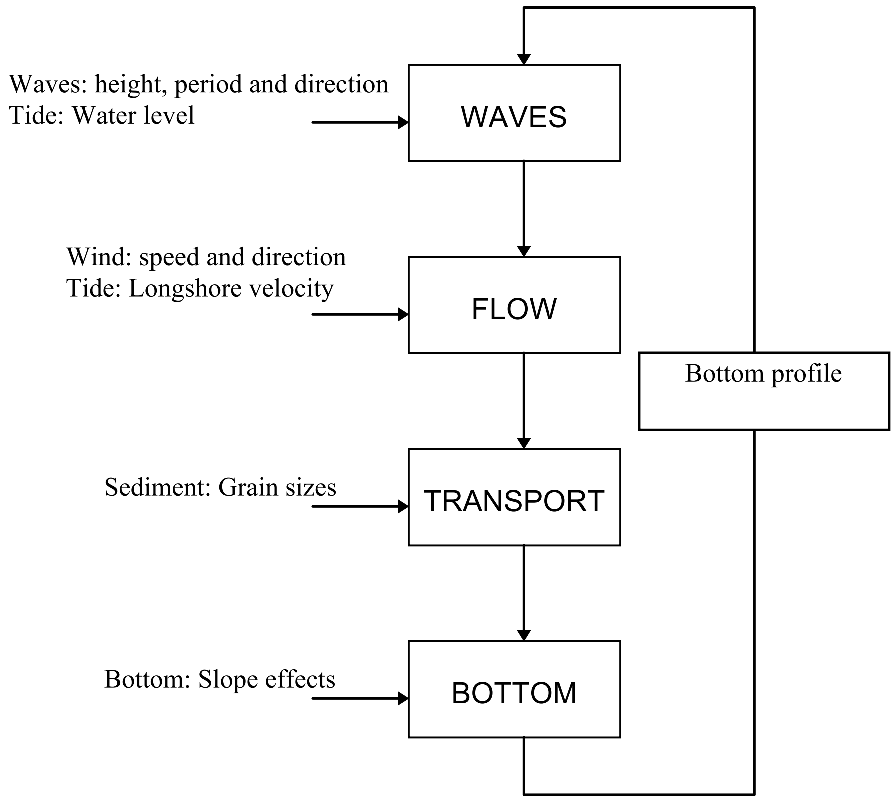


Figure 3.1 Overview of Unibest-TC modules and boundary conditions.

3.3 Description of Run and Model parameters

3.3.1 General

In this sub-section the model and run parameters are described. The parameters are grouped by the modules. The grouping is identical as used in the Unibest-TC user manual:

- General parameters
- Wave module related hydrodynamic parameters
- Flow module related hydrodynamic parameters
- Transport and Bottom module related parameters

Each sub-section has a similar set-up: first a detailed overview is given of the parameters, followed by a table in which the important aspects of all parameters are summarised. However, first some suggestions are given how to construct the computational grid.

3.3.2 Grid schematisation

In Unibest-TC a variable grid size can be used. The first criterion is that the bottom profile must be represented with sufficient accuracy (e.g. bars, shoreline, etc.). The second criterion is that the grid resolution must be able to capture the rapid change of wave characteristics in the surf zone. Both criteria are complementary as waves break in the surf zone were usually also bar systems occur. A general guideline is to use a grid size in the order of 5 to 10 m in the surf zone, in the more deeper region a grid size in the order of 50 to 100 m is acceptable.

The computational stepsize *[dx]* has to be chosen such that:

- the spatial resolution, in particular around sand bars, is sufficiently accurate,
- numerical stability is guaranteed (see Section 4.3),
- the maximum number of grid points of Unibest-TC (i.e. 399) is not exceeded.

Unibest-TC extrapolates the transports in the last wet cell linearly to zero to the last computational point. As a result, the extend of the computational domain can have a significant effect on the model outcomes. It is advised to extend the grid to the top of the dune or just above the maximum runup line (including tide and wave setup). If unacceptable erosion of the dry beach occurs, it may be an option to fix the dry beach artificially by prescribing a non-erodable layer in that region.

3.3.3 General parameters

The general parameters do not relate to a specific module but influence most of the modules.

Time step: [DT] in Days

- Default value: 2.0

The time step has to be specified in days. It is used in both the hydrodynamic and morphodynamic modules. *[DT]* has to be chosen such that the boundary condition time series are well represented and that no instabilities occur in morphological profile development. In Section 4.3 more information can be found regarding the numerical stability of Unibest-TC.

Number of time steps: [NT]

- Default value: 5.0

Determines the total simulation time. Has no influence on the numerical stability of the simulation. However, the user should be aware of the fact that the predictive capabilities decrease with increasing simulation interval. In Chapter 5 more information can be found on the predictive capabilities of Unibest-TC.

Transport at last computational grid point: [USTRA] in m^3/hr

- Default value: 0.0

As Unibest-TC does not have an appropriate description of the swash zone related processes the user can specify the transport rate at the last computational grid point. The model will interpolate the transports between the last considered wet cell and the user specified transport at the last computational grid point to obtain transport values for the dry part of the profile.

This option should be used with care, unrealistic values can have a significant effect on the profile development. Without sufficient profile data on which this parameter can be calibrated it is advised to set its value to zero (i.e. default value).

Maximum relative wave period: [TDRY]

- Default value: 40.0 [10 to 50]

This parameter determines at which water depth the Unibest-TC calculations are stopped. The maximum relative wave period is a dimensionless parameter which essentially indicated the non-linearity of the wave field and is defined as:

$$T_{DRY} = \frac{T_p}{\sqrt{g/h}} \tag{3.1}$$

Higher values for [TDRY] indicate that the simulations is continued into shallower water and vice versa (based on Eq. 3.1 the minimum water depth ,h_{min}, that is considered is: $h_{\min} = g\left(\frac{T_p}{T_{DRY}}\right)^2$). For the Rienecker and Fenton theory *[TDRY]* should be limited to 25, in proto-type conditions values of up to 40 can be used.

Especially during lengthy morphodynamic simulations a relative high value of [TDRY] can result in unstable wave or bottom update results. It is therefore advised to use values in the range of 30 to 40 for long term morphodynamic simulations.

Temperature of the water: [TEMP] in °C

- Default value: 10.0 [4.0 to 20.0]

Is used to determine the water density according to expression from Van Rijn (1993). Has little influence on model results.

Salinity of the water: [SALIN] in promilles

- Default value: 0.0 [0.0 to 40]

Is used to determine the water density according to expression from Van Rijn (1993). Has little influence on model results.

Summary of general parameters:

Variable	*Explanation*	*Ranges*
DT	Time step in days	minutes to days
NT	Number of time steps	N.A.
USTRA	User defined transport rate at the last computational gridpoint (m^3/hr)	N.A.
JFR	Frequency of output to *.daf file (JFR = 1 means every time step)	greater or equal then 1
TDRY	Relative wave period is a dimensionless parameter indicating the non-linearity of the wave field. Stokes theory is valid up to *TDRY* of order 10; Rienecker and Fenton theory for *TDRY* up to order 25. Unibest-TC computations are stopped once *TDRY* exceeds the user-defined value.	10 to 40
TEMP	Temperature of the water in °C (Celsius)	4 to 20
SALIN	Salinity of the water in promille 0/00	0 to 40

Table 3.1 Overview of general parameters.

3.3.4 Wave related hydrodynamic parameters

As waves are usually the dominant forcing in the nearshore coastal regions an accurate description of the wave characteristics is of vital importance. Modifications of these parameters affects the wave model but also all other modules which use wave forcing as input.

Factor for wave dissipation: [ALFAC]

- Default value: 1.0 [0.6 to 1.2]
- Battjes and Janssen (1978)

Factor which is used in the wave dissipation expression of Battjes and Janssen (1978). In combination with the wave breaking parameter *[GAMMA]* this parameter can be used to calibrate the wave model (see also Chapter 4).

Variation of this parameter influences the wave height prediction over the complete profile, as can be seen in Figure 3.2. It is advised to use the default value, *[ALFAC]*=1. In the benchmarking database this parameter is set to its default value in all experiments.

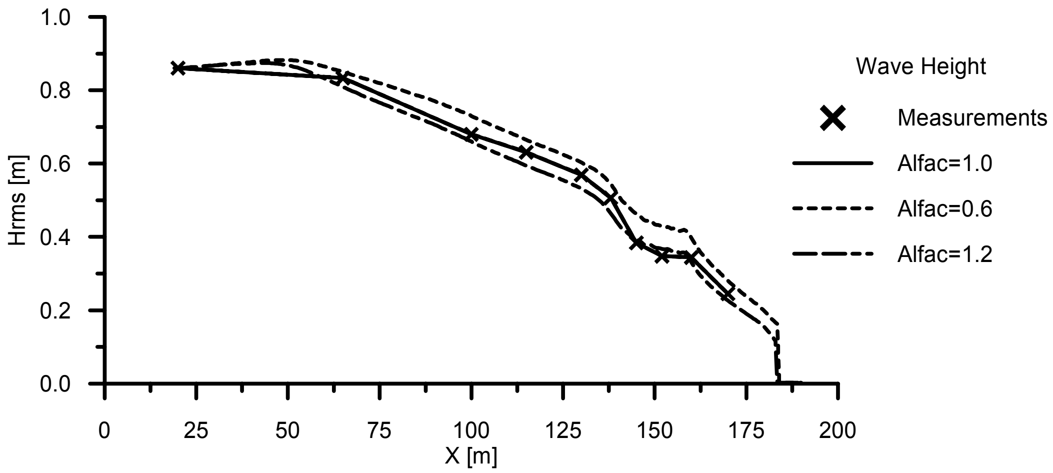


Figure 3.2 Influence of various settings of *[ALFAC]* on wave height distribution for LIP11D: Test 1B.

Wave breaking parameter: [GAMMA]

- Default value: 0.0 [0.5 to 0.8] (default is relation of Battjes and Stive, 1985)
- Battjes and Stive (1985)

In combination with the wave dissipation parameter *[ALFAC]* this parameter can be used to calibrate the wave model (see also Chapter 4). In absence of reliable wave data to calibrate the model it is advised to use the Battjes and Stive (1985) relation, which is used by default: *[GAMMA]*=0.

This parameter can have a significant influence on the predicted wave height as can be seen in Figure 3.3. Especially the initiation of breaking is influenced. In the benchmarking database the Battjes and Stive relation is mainly applied.

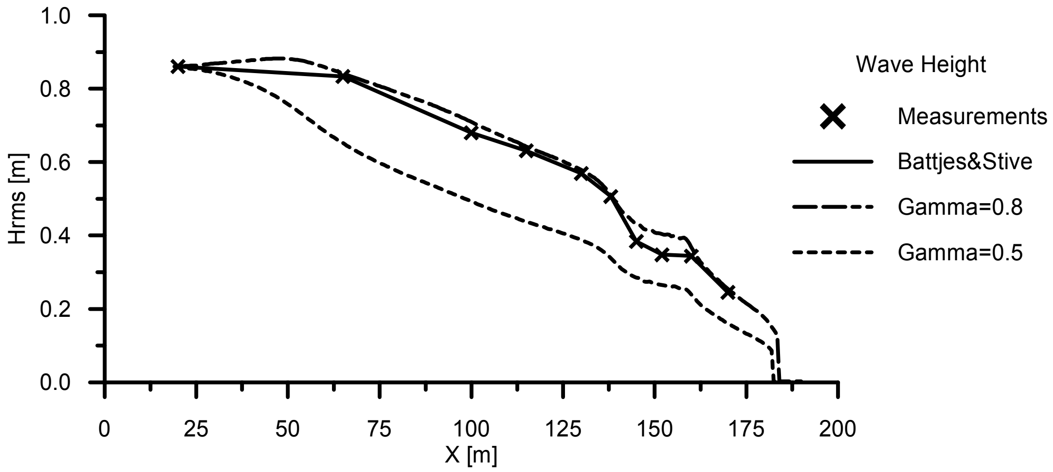


Figure 3.3 Influence of various settings of *[GAMMA]* on wave height distribution for LIP11D: Test 1B.

Slope of wave front: [BETD]

- Default value: 0.10 [0.03 to 0.2]
- Nairn et al. (1990) and Stive and De Vriend (1994)

With this parameter the roller model can be influenced. Higher values reduce the lag effect (lower persistence of wave breaking behind a bar), lower values increase the lag of energy transfer from waves to the underlying water (decreased breaking on top of the bar and increased breaking behind the bar). This parameter only has a small effect on the wave height predictions. However, the wave set-up can be influenced significantly. This parameter can be used to calibrate the water levels and the flow-module as it determines the cross-shore distribution of the surface shear stress due to wave breaking.

In Figure 3.4 the roller dissipation for various settings of *[BETD]* is investigated. As the surface shear stress due to wave breaking is linearly dependent on the roller dissipation it gives insight how the forcing of the water is distributed. High values of *[BETD]* result in smaller lag and vice versa. This is especially clear in the trough region shoreward of the breaker bar (around x=150m), if the roller dissipation is compared to the wave dissipation from the Battjes&Janssen model (from which the roller dissipation is derived). In Figure 3.4 it can also be seen that roller dissipation of the low *[BETD]* value results in an unrealistic lag. Too much energy is being dissipated shoreward of the inner bar.

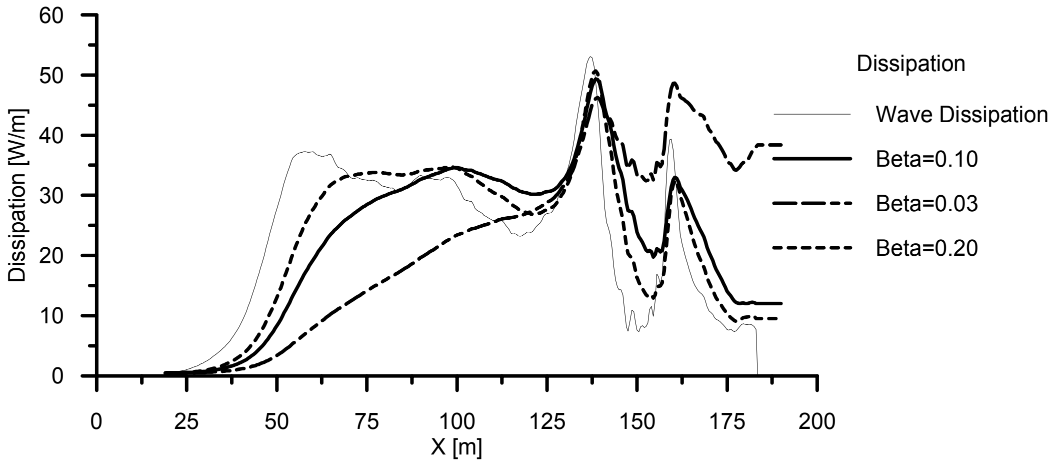


Figure 3.4 Roller dissipation for various setting of *[BETD]* for LIP11D: Test 1B.

That a value of 0.03 is too low can also be seen in Figure 3.5: unrealistic high return flow velocities are predicted in the region between the inner bar and the shoreline. The roller model also has a significant effect on the magnitude of the undertow: low values of *[BETD]* results in a large roller which is an important contribution to the total mass flux which has to be compensated by the return flow.

This illustrates the delicate balance between the roller contribution to the mass flux and the lag of the wave forcing. The lag effect can be significant, which result in a lower shear stress on top of a bar (x=138, Figure 3.4), but the increased return flow due to the larger roller is dominant and increases the velocities on top of the bar (Figure 3.5).

In the benchmarking database the default value has been applied in all simulations. In most applications a value between 0.05 and 0.10 gives a realistic lag effect.

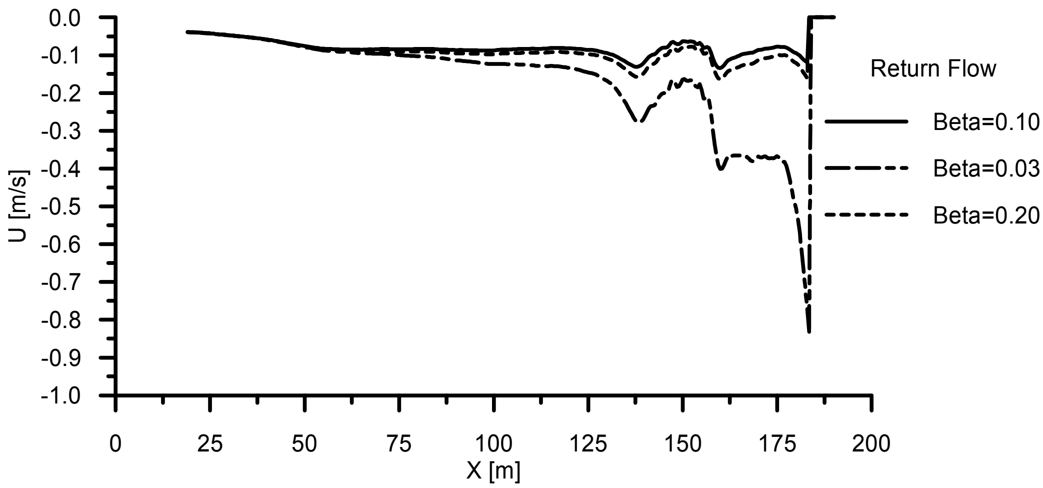


Figure 3.5 Depth averaged return flow for various settings of *[BETD]* for LIP11D: Test 1B.

Bottom friction factor: [FWEE]

- Default value: 0.01 [0.001 to 0.10]

This parameter influences the amount of wave dissipation due to bottom friction. Especially, if wave calculations are made over a relatively long distance (3 to 10 km) this parameter can influence the wave height predictions significantly. Within the surf zone this parameter has little influence as wave breaking is dominant.

Although, the value of *[FWEE]* is influenced by the bed forms (i.e. ripple height) it has been found that the default value gives the best results. In the benchmarking database the default value has been applied in all simulations. Especially an over-estimation of *[FWEE]* can have a significant influence on the wave height predictions (see e.g. Figure 3.6).

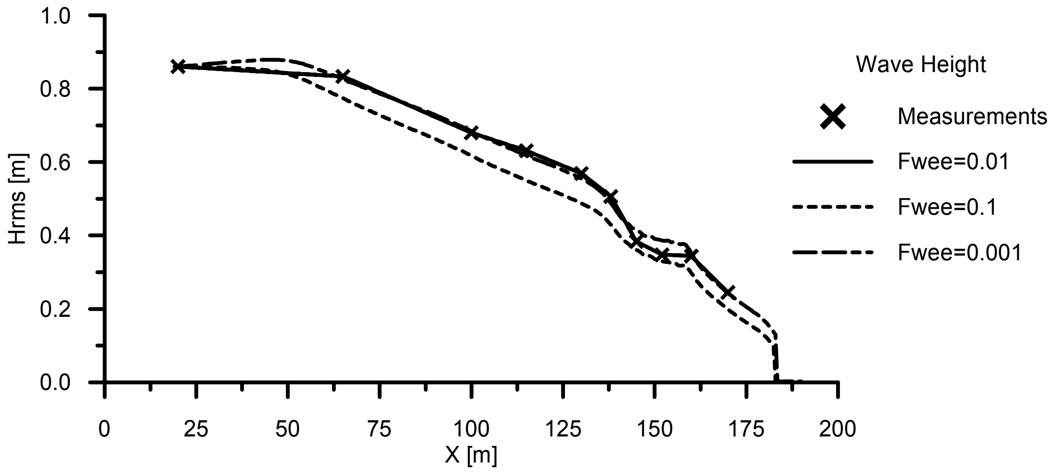


Figure 3.6 Wave height predictions for various settings of [FWEE] for LIP11D: Test 1B.

Correlation coefficient wave envelope and bound long waves: [C_R]

- Default value: 0.25 [0 to 1]
- Roelvink and Stive (1989)

This coefficient arranges the phase shift between the long-wave and short-wave envelope. In case of complete bound long wave the two are exactly out of phase (phase shift is -π). However, in general this does not occur in the nearshore region where slightly negative values are present seaward of the surf zone and slightly positive inside the surf zone as is illustrated in Figure 3.7 below.

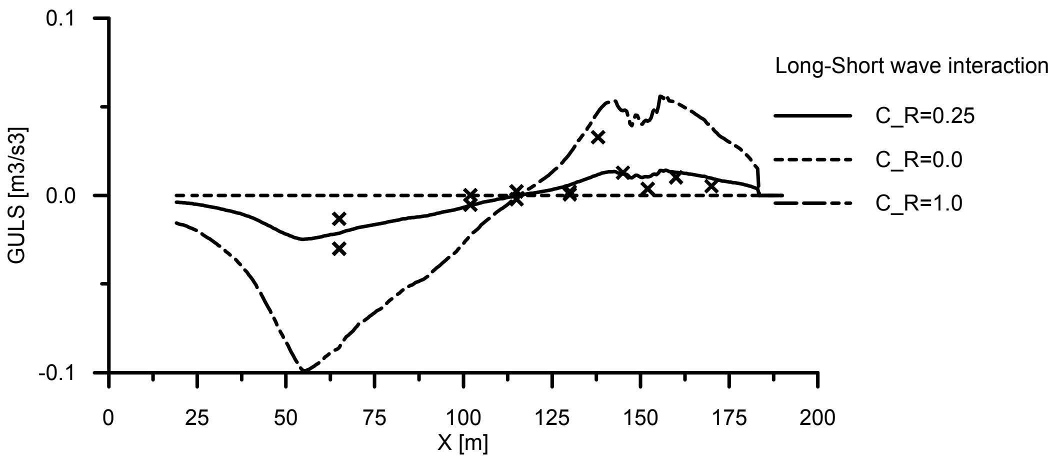


Figure 3.7 Long-shore wave interaction for various settings of [C_R] for LIP11D: Test 1B.

A negative phase shift will result in increased offshore transports as the short waves are the highest in the trough of the long wave (a positive phase shift has the opposite effect). By increasing *[C_R]* the effect of the long wave will be enhanced (i.e. increased offshore transport outside the surfzone and increased onshore transport inside the surfzone). By setting *[C_R]* to zero the long wave effect is eliminated (see e.g. Figure 3.8).

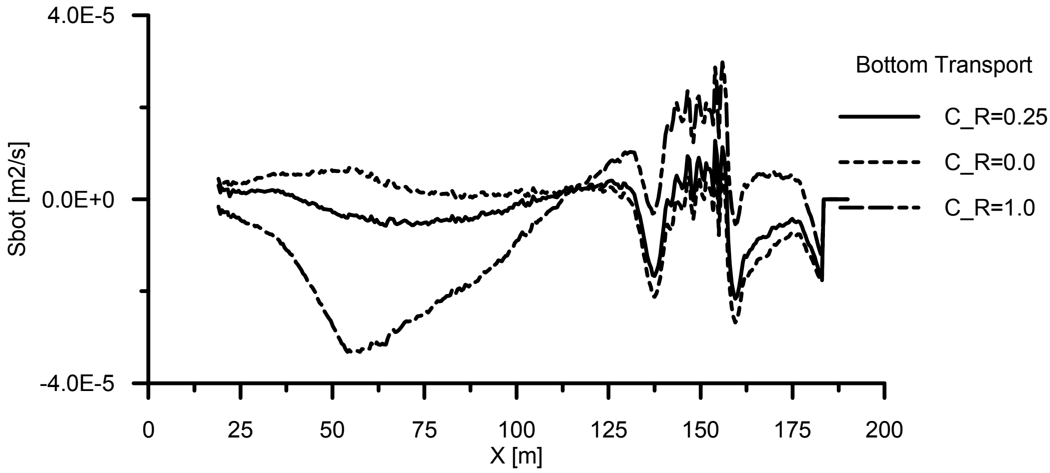


Figure 3.8 Cross-shore bed load transport for various settings of [C_R] for LIP11D: Test 1B.

Breaker Delay: [K_IJL], [F-LAM], [POW]

- Breaker delay switch: *[K_IJL]=1* [0 or 1]
- Number of wave lengths: *[F-LAM]=2.0* [0.5 to 2.5]
- Power in weighting functions: *[POW]=1.0* [0.5 to 2]
- Roelvink et al. (1995)

Breaker delay can be regarded as an extension to the roller model. It modifies the rate of wave breaking via a modification of the reference depth which is used to determine the local maximum possible wave height.

[POW]=1 implies application of a linear weighting function, *[POW]>1* decreases the relative influence of the breaker delay function.

In general it improves results in case of swell-type of conditions. In case of short waves the breaker delay often leads to an over-prediction of the wave heights. This is illustrated in Figure 3.9 and Figure 3.10. In Figure 3.9 the effect of breaker delay is shown for LIP11D Test 1B (H_{rms}=0.90m; T_p=5.0s) and in Figure 3.10 for LIP11D Test 1C (H_{rms}=0.4m; T_p=8.0s). The wave conditions in Test 1B are typical sea waves, whereas the wave conditions for Test 1C can be considered as swell.

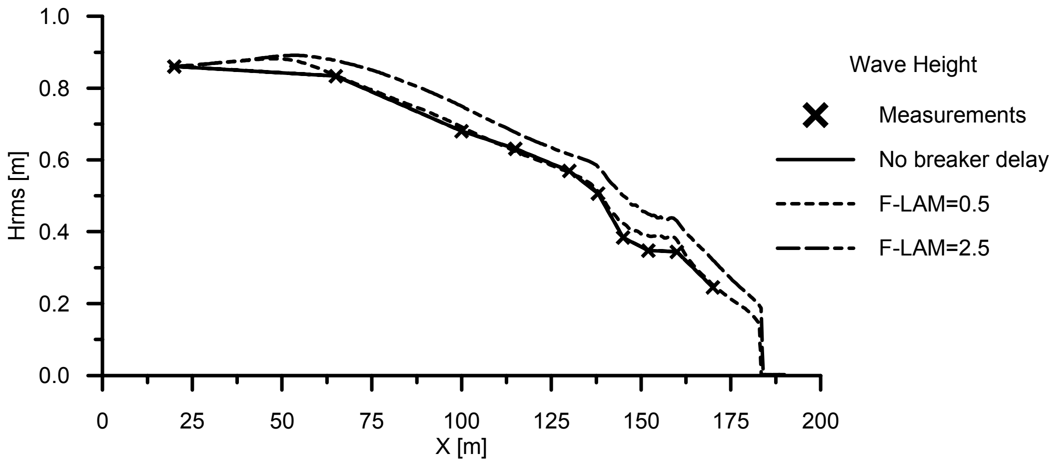


Figure 3.9 Influence of breaker delay on wave height for LIP11D: Test 1B.

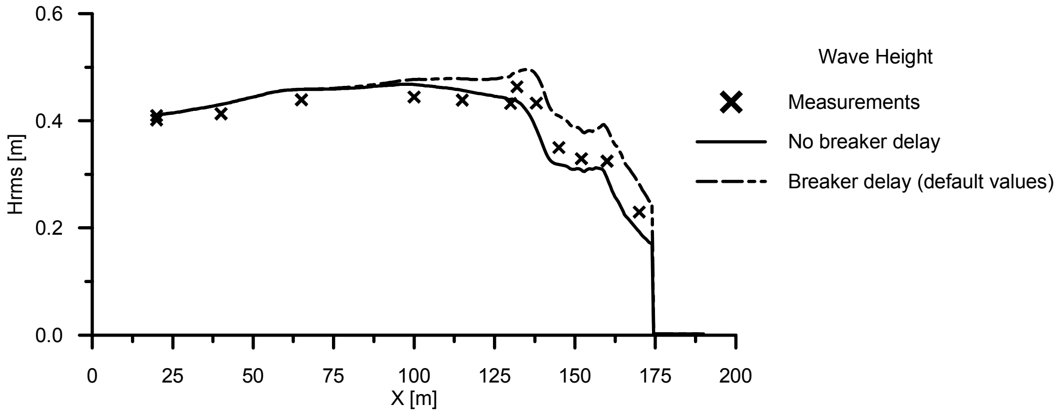


Figure 3.10 Influence of breaker delay on wave height for LIP11D: Test 1C.

It can be seen that the with breaker delay the results of Test 1C improve with breaker delay. However, for Test 1B it results in a significant over-prediction of wave heights. Experience with the breaker delay concept is limited so no general guidelines can be given. Below effects of the two breaker delay parameters is investigated for LIP11D Test 1C.

For Test 1C a value of *[F-LAM]*=1.0 seems to give better results than the default value (Figure 3.11). The power in the weighting function *[POW]* only has a limited influence on the wave height prediction (Figure 3.12).

If breaker delay is used it is advised to apply the default values. Calibration is best done via tuning of [F-LAM] while keeping [POW] constant (see also Section 4.2.1).

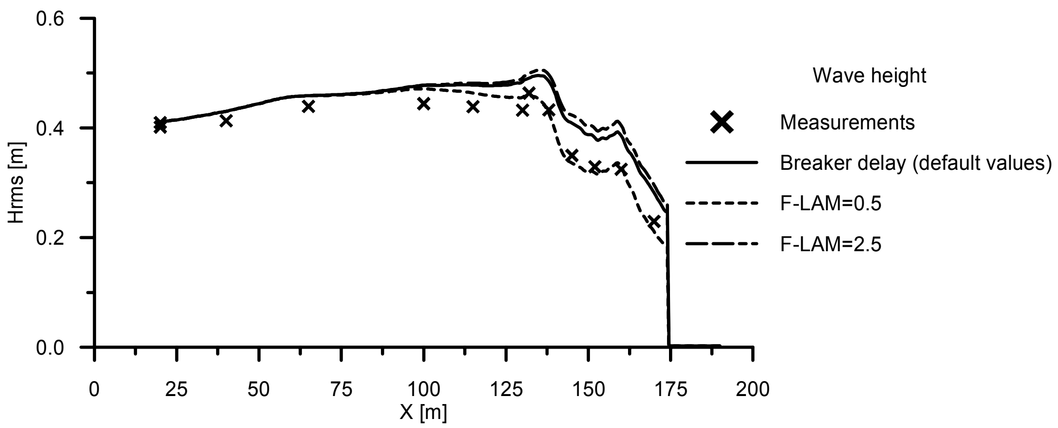


Figure 3.11 Influence of [F-LAM] on wave height predictions for LIP11D: Test 1C.

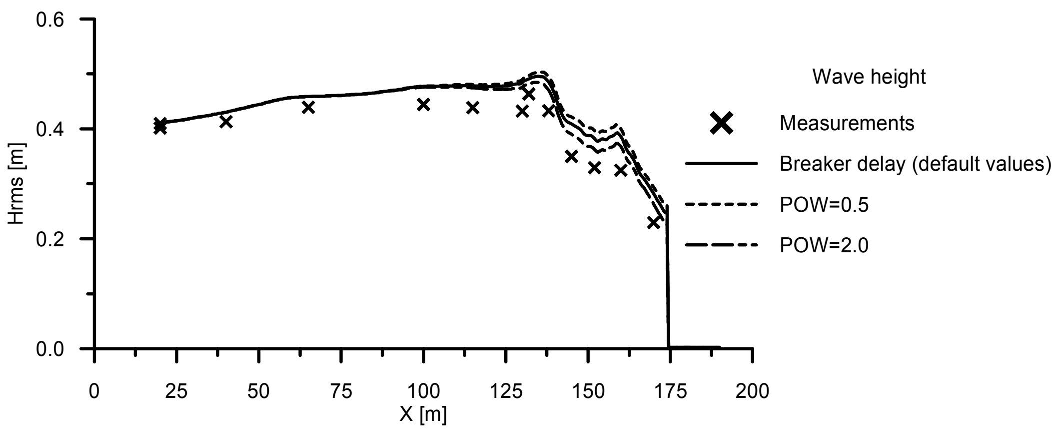


Figure 3.12 Influence of [POW] on wave height predictions for LIP11D: Test 1C.

Summary of wave related hydrodynamic parameters:

Variable	*Explanation*	*Ranges*
ALFAC	Wave breaking parameter for use in dissipation formulation.	0.6 to 1.2
GAMMA	Wave breaking parameter to determine maximum local wave height. Set to '0' for default value according to Battjes&Stive (1985).	0.5 to 0.8
BETD	Roller parameter, expressing the steepness of the wave front.	0.03 to 0.2
FWEE	Friction factor for wave dissipation due to bottom friction.	0.001 to 0.10
C_R	Correlation coefficient between wave envelope and bound long waves.	0 to 1
K_IJL	Breaker delay switch	0 or 1
F-LAM	Number of wave lengths over which weighted depth is integrated.	0.5 to 2.5
POW	Power in weighting function.	0.5 to 2

Table 3.2 Description of wave related hydrodynamic parameters.

3.3.5 Current related hydrodynamic parameters

The forcing for the wave model can be a combination of waves, tide and wind. Especially the cross-shore distribution of the cross-shore currents (undertow) is dominated by the wave forcing. The flow model itself only has three model parameters which have to be specified by the user.

Viscosity coefficient: [FCVISC]

- Default value: 0.10 [0.05 to 0.15]
- Battjes (1975)

A higher value results in a higher (breaking wave induced) viscosity. A higher viscosity results in lower internal velocity gradients and hence in flatter vertical velocity profiles and vice versa. In general a value of 0.10 gives reasonable results. Variation of this parameters should mainly be used to tune the vertical velocity profiles.

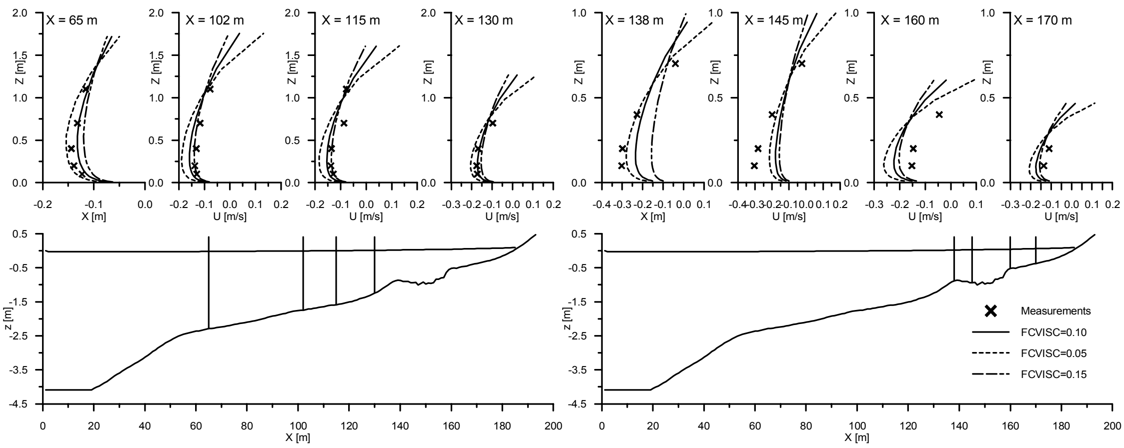
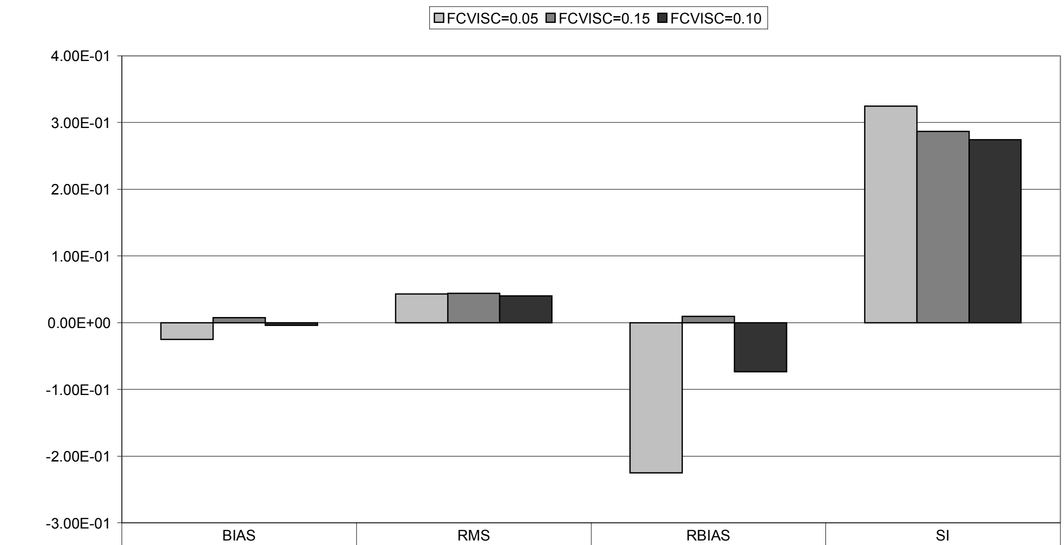


Figure 3.13 Comparison of cross-shore velocity profiles at various locations for variation of *[FCVISC]* for LIP11D: Test 1B.

This is illustrated in Figure 3.13 where the vertical (cross-shore) velocity profiles are shown. On average the default value gives the best approximation, but depending on the location the other values give better agreements with the measurements. The predicted undertow profiles can be evaluated with the Statistical Analysis Tool (see Chapter 6) with which the model performance is evaluated with statistical parameters. The result of some of the appropriate statistical parameters is shown in Figure 3.14 (for a detailed description of the parameters is referred to Section 6.4.2). It can be seen that for most of the MPS's the default value gives the best overall agreement with the measurements (zero value indicates perfect agreement).



	BIAS	RMS	RBIAS	SI
FCVISC=0.05	-2.51E-02	4.32E-02	-2.25E-01	3.25E-01
FCVISC=0.15	7.46E-03	4.42E-02	9.58E-03	2.87E-01
FCVISC=0.10	-4.04E-03	4.03E-02	-7.33E-02	2.74E-01

Statistical parameters

Figure 3.14 Example of Model Performance Statistics (MPS) for LIP11D: Test 1B.

Friction factor for mean current: [RKVAL] in m

- Default value: 0.01 [0.0005 to 0.20]
- Soulsby et al. (1993)

The friction factor is in fact a roughness height, which related to the grain size of the bed material or to the ripple height (if present). This parameter can be used to optimise velocity results, higher values results in large bed shear stresses and hence lower velocities (see e.g. Figure 3.15).

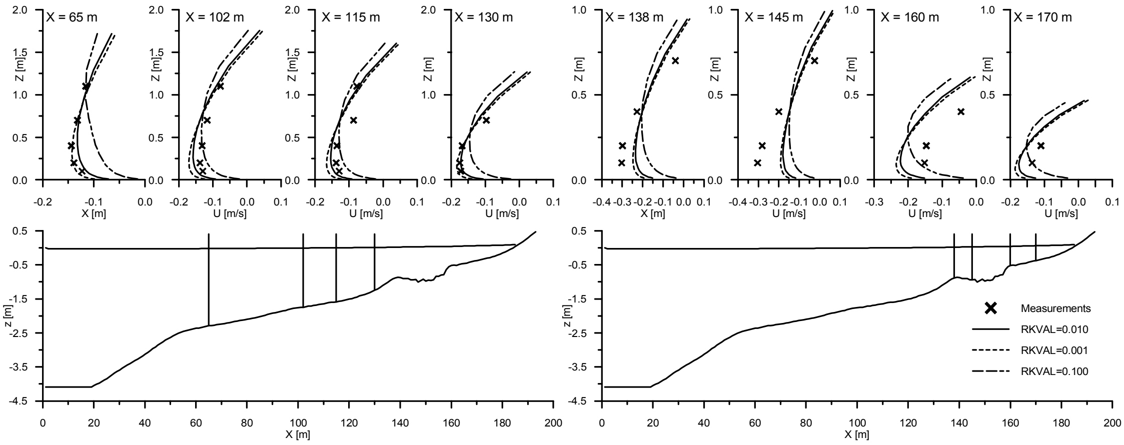


Figure 3.15 Comparison of cross-shore velocity profiles at various locations for variation of [RKVAL] for LIP11D: Test 1B.

In the benchmarking database usually the default value is used (for the Egmond cases a value of 0.05 is used and for the Reniers experiment 0.0005).

Reference depth for tidal velocity: [DIEPV] in m

- Default value: 5.0

The longshore tidal velocities are calculated by determining the alongshore slope of the water surface (this slope is assumed to be constant over the cross-shore profile). The slope is determined by using the Chezy formulation:

$$\bar{v}_{tide} = C\sqrt{h\frac{\partial h}{\partial y}} \tag{3.2}$$

The water depth in Eq. 3.2 is the reference depth.

As no wave effects are accounted for in Eq. 3.2, it is essential to ensure that the reference depth is well outside the surf zone.

Summary of current related hydrodynamic parameters:

Variable	*Explanation*	*Ranges*
FCVISC	Viscosity coefficient of vertical velocity profile.	0.05 to l.10
RKVAL	Friction factor for mean current computation.	0.0005 to 0.20
DIEPV	Reference depth for tidal velocity.	> 0

Table 3.3 Description of current related hydrodynamic parameters.

3.3.6 Grain size parameters

The size of the sediment grain sizes can have a significant influence on the calculated transports. However, it's effect on the profile development can not always be predicted before hand. As it is one of the few parameters which can relatively easy be measured there is usually some data available on the sand characteristics. Furthermore, Unibest-TC has an option to prescribe a cross-shore varying grain size. In general it is not advised to calibrate Unibest-TC by tuning these parameters.

Particle grain size: [D50], [D90] and [DSS] in m

- Median grain size: *[D50]=0.0002m*
- 90% grain size: *[D90]=0.0003m*
- Grain size of suspended sediment: *[DSS]=0.00017m*

The particle grain sizes D_{50} and D_{90} are usually based on a sieve curves. In experiments it was found that the suspended sediment particle grain size is about 60 to 70% of the median grain size. To that end, a particle grain size *[DSS]* for sediment sediment has to be specified.

Cross-shore varying grain size

The particle grain size can be varied cross-shore by means of specifying multiplication factors at three reference depths in the bottom profile (see Table 3.4).

Summary of grain size parameters:

Variable	*Explanation*
D50	D_{50} grain diameter (in meters) of bed material.
D90	D_{90} grain diameter (in meters) of bed material.
DSS	D_{50} grain diameter of suspended sediment.
DVAR	Cross-shore varying grain size switch.
FDIA0	Diameter multiplication factor at HDIA0.
FDIA1	Diameter multiplication factor at HDIA1.
FDIA2	Diameter multiplication factor at HDIA2.
HDIA0	Reference depth for FDIA0 (most shoreward)
HDIA1	Reference depth for FDIA1
HDIA2	Reference depth for FDIA2 (deep water)

Table 3.4 Description of grain size parameters.

3.3.7 Transport parameters (including slope effects)

These parameters are related to the sediment transport formulations.

Morphodynamic switch: [IBOD]

- Default value: 1 [0 or 1]

With this switch the bottom updating module can be switched on or off. If a multiple time steps are executed the hydrodynamic and transport modules will generate output (according to the prescribed boundary conditions) based on the initial bathymetry.

Fixed bottom layer: [REMLG] in m

- Default value: 0.10 m

In combination with the fixed bottom layer this parameter is used to limit sediment transport and bottom changes. *[REMLG]* can be considered as an alluvial layer on top of the fixed bottom layer. If the bottom height is less than the fixed bottom added with *[REMLG]* a reduction factor is applied (if bottom height reaches fixed bottom the reduction factor is zero). In this way a gradual decrease in sediment transports is imposed when the bottom profile reaches the fixed layer which avoids numerical instabilities.

Current and wave related roughness: [RC] and [RW]

- Default value for *[RC]*: 0.01 m [0.005 to 0.10]
- Default value for *[RW]*: 0.002 m [0.001 to 0.10]

In proto-type conditions the values of *[RC]* and *[RW]* should lie between 0.001 and 0.10 m and *[RW]* should. In general, the *[RW]* value should be lower than the value for *[RC]*.

As can be seen in Figure 3.16, the effect of *[RW]* and *[RC]* is significant. Moreover, the way in which these parameters influence the transports is not known beforehand as these parameters not only determine the effective shear stress, but also, among others, the effective grain size diameter. These parameters can be used to tune the transports, but it is advised always to check the resulting transports.

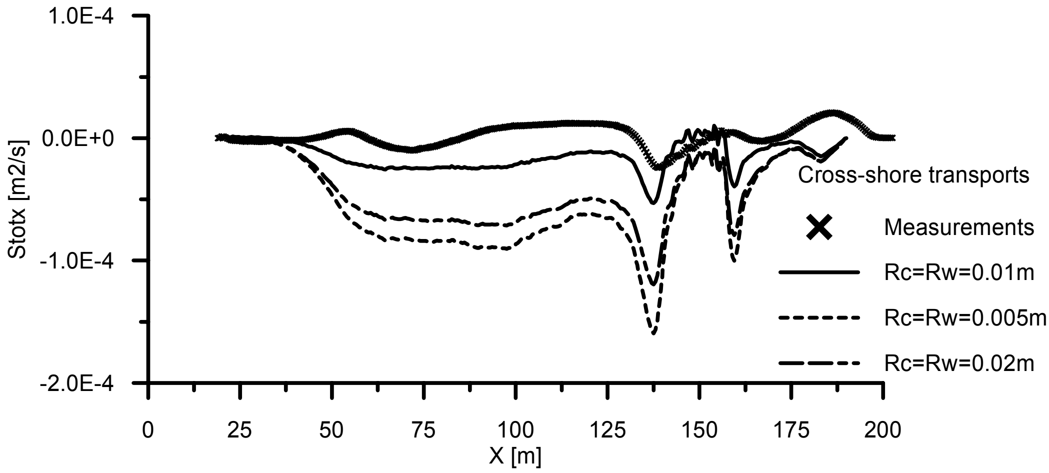


Figure 3.16 Cross-shore transport for various settings of *[RW]* and *[RC]* for LIP11D: Test 1B.

Slope effects: [TANPHI1], [TANPHI2], [XF1] and [XF2]

- Default value for *[TANPHI1]*: 0.03 [0.01 to 0.60]
- Default value for *[TANPHI2]*: 0.10 [0.01 to 0.60]

Modification of the bed load transport as a function of the local bed slope. Local angle of internal friction is specified via two sets of x-coordinates and internal friction angel: *[XF1]* & *[TANPHI1]* and *[XF2]* & *[TANPHI2]* specified in shoreward direction (*[XF1]<[XF2]*). The slope effect is primarily included to damp offshore migrating bars. Without slope effects included, offshore migrating bar will continue to exist which is in contrast with observations (Roelvink et al., 1995).

The slope effect is often used to calibrate the long term behaviour of the bottom profile (predictions longer than 1 year). A general rule is that *[tan(φ)]* should be decreasing in seaward direction to damp possible offshore migrating bars. A calibration for Terschelling, a relatively mild beach with a slope of 1:150, (Roelvink et al., 1995) resulted in 0.03 for *[tan(φ)]* outside the surf zone and 0.10 well inside the surf zone. Aarninkhof et al. (1998) carried out a sensitivity analysis of Noordwijk, with a slope 1:100, in which they used values for *[tan(φ)]* of 0.15 and 0.21 outside and inside the surf zone respectively. Figure 3.17 shows the effect of different subaquous angles of natural repose on the final profile evolution after 50 days in case of a constant wave height H_{rms} = 1.5 m at Noordwijk. The values of *[tan(φ)]* are constant along the beach profile, though generally, they are set to decrease somewhat in off-shore direction, to facilitate the damping of bars at deeper water.

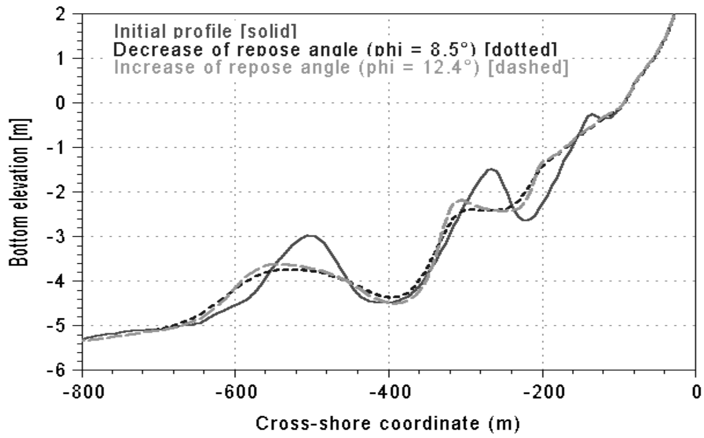


Figure 3.17 The effect of slope effects on final profile evolution (Aarninkhof et al., 1998).

Summary of transport parameters (including slope effect):

Variable	*Explanation*	*Ranges*
IBOD	Morphodynamic switch	0 or 1
REMLG	Layer over which the sediment transport is reduced to zero in case of a fixed bed	greater then 0
RC	Current related roughness for sediment transport computation, the default value RC = 0.01 is obtained from Delta Flume experiments	0.005 to 0.10
RW	Wave related roughness for sediment transport computation, the default value RW = 0.01 equals RC	0.005 to 0.10
TANPHI1	Internal friction angle at location XF1;	0.02 to 0.6
TANPHI2	Internal friction angle at location XF2	0.02 to 0.6
XF1	Reference location for TANPHI1 (most seaward)	*[XF1]<[XF2]*
XF2	Reference location for TANPHI2 (most shoreward)	*[XF1]<[XF2]*

Table 3.5 Description of transport parameters.

4 Calibration of Unibest-TC

4.1 Introduction

A calibration is the next step after having prepared the required schematisations. In this phase the model is tuned to find an optimal agreement with measurements. Here, a methodology framework is given on how to perform a calibration of Unibest-TC. A calibration is here defined as the optimisation of the model performance by tuning the model parameters within realistic ranges.

In this chapter it is assumed that:
- reliable data is available against which the model can be compared (Section 2.2.2 is devoted to cases when there is only a limited amount of data available),
- the user has a basic understanding of the model and its model parameters (see also Chapter 3).

In general two phases can be distinguished during such a calibration: in the first phase the hydrodynamic modules (e.g. wave heights, undertow) are calibrated, followed by the calibration on the morphological profile development (e.g. sand transport).

In Section 4.2 a general calibration procedure is given which gives a novice user insight in the order at which the sub-models should be calibrated and the order in which the appropriate model parameters should be investigated. The numerical stability of Unibest-TC is discussed in Section 4.3. In Section 4.4 an overview is given of the types of deviations that can be expected and what implications these might have.

4.2 Calibration procedure

It is impossible to give a generally applicable calibration method which guarantees success. It is essential that the user has some basic physical understanding and experience with numerical modelling. Here some general guidelines are given which should help the novice user of Unibest-TC.

Calibration of Unibest-TC is in fact a calibration of its sub-modules. The first step is to calibrate the hydrodynamic modules (waves and flow) followed by a calibration of the morphological modules (sand transport and bottom changes).

4.2.1 Calibration of wave model

Calibration of wave height
The wave model should first be calibrated on wave measurements across the surf zone. To that end, the following two model parameters are available:
- *[ALFAC]*, a constant in dissipation equation which should be of order 1,

- *[GAMMA]*, maximum ratio of wave height over water depth.

In general it is advised to use the Battjes and Stive (1985) relation to estimate *[GAMMA]* (set *[GAMMA]* to zero in model input). Especially along the Dutch coast it has been found that the application of the Battjes and Stive relationship can lead to an over-estimation of the wave height in the inner surf zone (see e.g. Kleinhout, 2000). A first step to improve results is to vary *[GAMMA]* and keep *[ALFAC]* constant. A second approach is to use the relationship: $\alpha\gamma^{-5}$ = *constant* (see e.g. Beyer, 1995 and Dingemans, 1997). In practical applications the first approach is suggested.

The wave height predictions are relatively sensitive to variations in these parameter. Consequently, many other model outcomes (including the morphological profile predictions) are also significantly influenced by variation of these parameters.

Calibration of roller module

The roller model describes the rate of energy transfer from the breaking waves to the roller (the white water in front of the wave) and from the roller to the underlying water. It introduces a lag in the energy transfer from the breaking waves to the underlying water. This effect can be important for accurate water level predictions and the undertow and longshore current calculations. There is one model parameter, the slope of the wave front, *[BETD]*, which can be used to tune the rate of energy transfer to and from the roller. This parameter can be used to improve results of:

- water level set-up: lower values decrease rate of energy transfer and result in a shoreward shift of the initial set up and vice versa,
- undertow (wave induced cross-shore currents): lower values increase the lag effect which will result in higher undertow values in troughs behind breaker bars (the velocities on top of the bar will decrease) and vice versa,
- longshore currents: lower values will bring the forcing further shoreward and vice versa

In general a value in the range of 0.05 to 0.10 is suggested. The model is relatively sensitive to variations of this parameter in the region where breaking occurs. Outside the surf zone and in the saturated inner surf zone this parameter has little influence. In long term predictions it can influence the location, migration and size of possible breaker bars.

Calibration of breaker delay module

The breaker delay is controller by two parameters:

- the number of wave lengths used for integration function, *[F-LAM]*,
- the power in the weighting function, *[POW]*.

Experience with this function is limited. In comparison with measurements it has been found that especially during swell conditions (relatively long waves) it can improve the wave height and bottom profile predictions. If it is applied, a value in the range of 1 to 2 should be used. An increasing *[F-LAM]* leads to a shoreward shift of wave height. Furthermore the cross-shore distribution of the roller dissipation, which drives the undertow model, is smoothed significantly which enhances bar growth. In this regard the breaker delay model can be seen as an extension of the roller model. The power function, *[POW]*, should be in the range of 1 to 2. Higher values reduce the effect of the breaker delay.

Compared to *[F-LAM]* it has relative little influence. It is advised set *[POW]* to 1 and use *[F-LAM]* for the calibration.

4.2.2 Calibration of Flow module

The flow module simulates both the cross-shore and longshore currents under the influence of tide and especially waves. The longshore tidal velocity is prescribed by a longshore velocity at a certain water depth. The longshore velocities are computed by using the Chezy relation in which the roughness height is, *[RKVAL]*, can be prescribed by the user. It's value depends on the ripples and bed material. In general its value should lie between 0.005 and 0.10. Increasing values will decrease the tidal longshore velocities (and vice versa).

The flow module itself only has a viscosity coefficient *[FCVISC]* to influence the shape of the vertical velocity profile. Variation of this parameter has limited effect. In general its value should lie in the range of 0.05 to 0.10. Currents induced by (breaking) waves are obviously strongly influenced by the settings of the wave module. If data is available to calibrate the cross-shore distribution of the undertow and longshore currents in the surf zone, it is usually necessary to calibrate the wave and flow model together.

4.2.3 Calibration of morphological modules

The morphological modules consist of a transport module and a bed updating module. Apart from the time step (see also Section 4.3). the bed updating module has no parameters with which it can be modified.

The transport module has a number of parameters which can have a significant influence on the computed transport rates and hence the simulated morphological changes:

- the sediment grain size parameters: [D_{50}], [D_{90}] and [D_{SS}],
- roughness values for waves and currents which are used in the transport formulations: [r_w] and [r_c],
- slope effects via the modification of the internal friction angle: it is possible to have a varying friction angle via the specification of two internal friction angles.

Sediment grain size

The size of the sediment grain sizes has a significant influence on the calculated transports. However, it's effect on the profile development can not always be predicted before hand. As it is one of the few parameters which can relatively easy be measured there is usually some data available on the sand characteristics. In general it is not advised to calibrate Unibest-TC by tuning these parameters.

Typical values for the sediment grain sizes are:

- [D_{50}]: 150 to 400 μm
- [D_{90}] : 200 to 600 μm
- [D_{SS}]: 0.6 to 0.8 of the [D_{50}]- value.

Roughness height for waves and currents
In the present version of Unibest-TC (Version 2.0x) separate roughness height have to be specified for:

- currents: *[rc]*,
- waves: *[rw]*.

These roughness heights are used to determine the bed shear stresses, which are used to determine the transport rates, in the transport module. These parameters are NOT used in the flow model. Increasing the values for both parameters will lead to an increase in the suspended and bed load transport. These parameters can be used to calibrate the sediment transports. In proto type conditions the values of *[rc]* and *[rw]* should lie between 0.001 and 0.10 m and *[rw]* should. In general, the *[rw]* value (default: 0.002) should be lower than the value for *[rc]* (default: 0.01 m).

Slope effects
The subaquous angle of natural repose, *[tan(φ)]*, accounts for slope effects and affects the computed transport rates in two ways:

- The threshold criterion for the initiation of motion is adapted. With increasing *[tan(φ)]*, the non-dimensional critical shear stress θ_{cr} (according to Shields) decreases in case of upslope transport, and increases for downslope transport. In other words, upslope transport is stimulated with increasing *[tan(φ)]*, downslope transport hindered.
- The bed load transport rates are affected by means of a Bagnold multiplication factor β_s, which increases with increasing *[tan(φ)]* in case of upslope transport, and decreases in conditions of downslope transport. Again, upslope transport rates are facilitated by increasing values of *[tan(φ)]*, downslope rates hampered.

Both modifications to the computed transport rates result in the same effect: a higher value of *[tan(φ)]* stimulates accumulation of sediment around the bar crest, hence bar development, a lower value causes the damping of bars.

The profile development is relatively sensitive to variation in this parameter. Unibest-TC allows for (a user specified) linearly varying *[tan(φ)]*. A high value of *[tan(φ)]* (e.g. 0.6) practically removes the slope effect from the transport computations. It will have a significant effect on the profile development when the occurring bottom slopes are of the same order as the prescribed *[tan(φ)]*.

The slope effect is often used to calibrate the long term behaviour of the bottom profile (predictions longer than 1 year). A general rule is that *[tan(φ)]* should be decreasing in seaward direction to damp possible offshore migrating bars. A calibration for Terschelling, a relatively mild beach with a slope of 1:150, (Roelvink et al., 1995) resulted in 0.03 for *[tan(φ)]* outside the surf zone and 0.10 well inside the surf zone. Aarninkhof et al. (1998) carried out a sensitivity analysis of Noordwijk, with a slope 1:100, in which they used values for *[tan(φ)]* of 0.15 and 0.21 outside and inside the surf zone respectively.

Notice furthermore the value of [tan(φ)] is not allowed to exceed the local beach slope.

4.3 Numerical stability

The wave model and bottom updating module contain differential equations that are solved over the horizontal computational domain. For these modules, the numerical stability depends on both the ratio spatial stepsize *[dx]* over time step *[dt]*, and the local bottom slope *dz/dx*. There has been no numerical study to derive stability criteria for the model. Here we can only give some general guidelines.

Grid size
In Unibest-TC a variable grid size can be used. The first criterion is that the bottom profile must be represented with sufficient accuracy (e.g. bars, shoreline, etc.). The second criterion is that the grid resolution must be able to capture the rapid change of wave characteristics in the surf zone. Both criteria are complementary as waves break in the surf zone were usually also bar systems occur. A general guideline is to use a grid size in the order of 5 to 10 m in the surf zone, in the more deeper region a grid size in the order of 50 to 100 m is acceptable.

The computational stepsize *[dx]* has to be chosen such that:
- the spatial resolution, in particular around sand bars, is sufficiently accurate,
- numerical stability is guaranteed,
- the maximum number of grid points of UNIBEST-TC (i.e. 399) is not exceeded.

The first criterion implies an upper limit for *[dx]* (e.g. $dx \leq 10\ m$ in surf zone), the second implies a lower limit (e.g. $2.5 \leq dx$).

Numerical time step
As with any numerical model exercise it is advised to vary the numerical time step to investigate the effects on the final model results. The general procedure is to double the time step until significant errors (or instabilities) in the solution occur.

A representation of the boundary conditions is another criterion: the time step has to be small enough to represent all the wave, wind and tidal conditions. If a time series with a time step of 3 hours is used, this will be the upper limit (even though numerical stability can result in a much higher upper limit).

For further details regarding the calibration of Unibest-TC reference is made to earlier studies in this respect, like Reniers and Roelvink (1995), Reniers et al. (1995) and Reniers and Roelvink (1996).

4.4 Identification of deviations

Two types of deviations are distinguished:
- identification of deviations which can be improved by tuning the model (recoverable deviations),
- identification of deviations which are the result of the process descriptions in the model and can therefore not be improved by tuning the model (unrecoverable deviations).

Here, a pointwise description is given of the deviations that are usually encountered. Note, that again we are not able to give a complete overview.

4.4.1 Recoverable deviations

Wave model

- Over-prediction of wave heights.
  ⇒ Reduce offshore wave boundary condition in case of too high values along the complete profile.
  ⇒ Reduce γ breaker parameter, this will result in lower wave height in the surf zone and seaward shift of the breaker zone.
  ⇒ Check tidal water levels that are prescribed, if there is a mismatch between water level and wave height boundary conditions this can have a significant influence on the wave height predictions.
- Under-prediction of wave heights
  ⇒ Repeat 3 steps listed above.
  ⇒ Especially swell type wave conditions can result in an under-prediction of the wave heights in the breaker zone, this can be significantly improved by applying the breaker delay function.

Flow model

- Longshore velocities
  ⇒ Increase/Decrease roughness value.
  ⇒ Check longshore velocity boundary conditions and water depth at which it is applied. Make sure that the velocity signal is imposed at a water depth at which no wave breaking occurs.
  ⇒ If the longshore velocities in the surf zone show deviations with measurements, the wave model settings have to be modified. Under the assumption that the wave height predictions are already calibrated, the driving forces due to wave breaking have to be optimised. A decrease of the slope of the wave front, *[BETD]*, will results in a shoreward shift of the roller dissipation which drives the flow module. Tuning of the breaker delay parameters may also be an option.
- Cross-shore velocities
  ⇒ The cross-shore mass flux (i.e. depth averaged velocities) is determined by the wave height and the roller model. Especially in the breaker zone, the roller model has a significant influence by introducing a lag in the energy transfer from the wave to the underlying water. If the cross-shore velocities show a seaward shift, the slope of the wave front may be reduced. Tuning of the breaker delay parameters is also possible.
  ⇒ The vertical velocity distribution is also dependent on the cross-shore distribution of the dissipation as it determines the amount of turbulence which is generated by the waves. The viscosity coefficient *[FCVISC]* and the roughness height *[RKVAL]* can also be used to optimise the undertow. However, in general the wave forcing is the dominant effect, calibration of the wave parameters (roller model and breaker delay) is advised to optimise the flow model.

Morphological modules

- Sand transport
  - ⇒ see Section 4.2.3, in which the calibration of sand transport module is described.
- Bar migration and bar growth
  - ⇒ Especially the breaker delay function can be used to influence the bar generation and bar migration. With the breaker delay function enabled, the roller contribution to the mass flux and surface shear is shifted from the seaward flank/top of the bar to the bar trough. This can result in stable bars which would otherwise migrate offshore. Bar growth is also enhanced if the breaker delay function is activated due to a reduction of the transport capacity on top of the bar.
  - ⇒ The slope effect should mainly be used to dampen offshore migrating bars. However, it can also be applied in the surf zone to stabilise breaker bars.

4.4.2 Unrecoverable deviations

Unrecoverable deviations are often due to longshore non-uniformities such as:

- A longshore non-uniform bathymetry which can induce 3D currents patterns (e.g. rip currents).
- Headlands or breakwaters which can reduce the wave action and/or tidal currents.
- Offshore shoal which can reduce the wave action, or possibly increase wave action due to focussing of wave energy.

In practical applications it is often not clear what is the cause of deviations. It is advised always to assess the characteristics of the coast under investigation. Any of the items listed above can have an influence on the hydrodynamics and morphology. It is however important to realise that no model is perfect and that Unibest-TC is no exception to this rule. Many perfectly 2DV flume experiments have been used to validate Unibest-TC and in none of these studies perfect agreement between measurements and model results was obtained!

Focus on the model outcome which is relevant for the project and try and calibrate the model in such a way that all model parameters still have sensible values and all sub-modules show realistic results. A sensitivity analysis is an intrinsic part of any modelling exercise as it gives insight in the robustness of the model outcomes with respect to varying forcing conditions. If the model is relative insensitive to variation in forcing (and model parameters) of which there is the most uncertainty (i.e. least data available) it is an indication that the model results are consistent and relatively reliable. If the opposite occurs, it is essential to include prediction ranges which give an indication of the uncertainties in the model outcomes.

5 Validation & Determination of predictive capabilities

5.1 Introduction

After the calibration in which the model has been tuned to give an optimal representation of the investigated area, a validation is required to investigate the applicability and predictive capability of the model for the problem under investigation. During this phase the model settings as derived in the calibration are used to make model predictions which can be verified with measurements. In general there will be a temporal division between the calibration, validation and predictive simulations (e.g. calibration of model on profiles from 1990 to 1995, validation on profiles from 1995 to 2000 and forecast from 2000 to 2010). However, sometimes a spatial division is made due to lack of data (e.g. calibrate and validate the model for Noordwijk and apply it in Egmond). Both approaches will be discussed.

Determination of the predictive capabilities is one of the key aspects of a validation. This predictive capability can be derived based on the implemented physical processes. However, it is not always clear what the validity ranges of the implemented theories are. Furthermore, the effects of (non-linear) interactions between the various included processes is even more difficult to assess. Usually, the predictive capability for a certain application is derived from an interpretation of the results of the present validation or from a previous study in the same or a comparable application.

In summary, the following items will be treated:

- division between calibration/validation and prediction (temporal vs. spatial),
- predictive capability based on validation (e.g. agreement with measurements, sensitivity of predictions to small modifications in input),
- predictive capability based on implemented theory (which type of forcing is applied, which phenomena have to be predicted),

5.2 Validation vs. Calibration

The main difference between a validation and a calibrations is that during the calibration phase an optimisation of the model performance is obtained by tuning the model input parameters and the forcing conditions. However, during the validation phase no optimisation of the model performance is allowed. The only allowable difference between the calibrated model input and the input used in the validation are the boundary conditions. This enables a relative objective check of the model performance and it gives an indication of the model’s predictive capabilities for the problem under consideration. In other words, it is a quality assessment of the model forecast which is usually the aim of a consultancy project.

In the following two sub-sections the separation (temporal and spatial) between the calibration and validation versus prediction are discussed.

5.2.1 Temporal separation

A temporal separation is the most common and preferred way of setting up a calibration/validation/prediction (CVP) procedure. The model is calibrated with data of a certain period after which the model is validated with a subsequent period: e.g. the model is calibrated with hydrodynamic and bathymetric data from 1990 to 1995, after which the model is validated over 1996 to 2000. Finally, predictive simulations are made at the same location.

This is usually the most preferred CVP procedure because it requires no additional information or interpretation of hydrodynamic and morphological data.

5.2.2 Spatial separation

If it is impossible to apply a temporal separation (usually because there is a lack of data in the area of interest), a spatial separation can be applied. In a spatial separation of the CVP procedure the calibration and validation are carried out at a different location to where the prediction is made. Before such a spatial separation can be applied, the user has to check if the location which is used for the calibration and validation has the same characteristics as the location of the predictive simulations.

In the following cases, a spatial separation could be considered:

- No or not sufficient hydrodynamic data available in the area of interest.
  ⇒ Use hydrodynamic data from other sources in the same region, interpolate and/or refract the data to the correct location if necessary (see also Chapter 2).
- No or not sufficient bathymetric data available for calibration and validation purposes.
  ⇒ Use hydrodynamic forcing of location of interest and construct characteristic bottom profile or use bathymetric data from nearby locations.
- No or not sufficient data is available at all (i.e. no hydrodynamic and bathymetric data).
  ⇒ In this case a so-called analogy model could be applied. This implies execution of the CVP procedure on a nearby location with similar hydrodynamic and morphological characteristics and translate these results to the area of interest based on expert judgement.

The hydrodynamic forcing should be approximately identical for both locations. Furthermore, the observed morphological developments in both locations should have a high correlation. These checks are essential as the implicit assumption in a spatial separation is that the calibrated and validated model may be applied with the same model parameters on the location of the predictive simulations. Note that hydrodynamic forcing or bottom profile for the predictive simulations do not necessarily have to be identical.

5.3 Predictive capabilities

The ultimate goal of the calibration and validation exercises is to obtain a model with maximum predictive capabilities. This section gives some guidelines in how to interpret and evaluate the model performance based on the validation and based on the implemented theory (i.e. what are strong and weak points in the implemented theories).

5.3.1 Predictive capabilities based on implemented theory

Wave module
The wave module consists of the implemented theory of the wave model of Battjes & Janssen (1978) and the roller model of Nairn et al. (1992) modified according to De Vriend & Stive (1994). These models have been well-established and have been verified extensively against laboratory and field experiments. The cross-shore wave and water level predictions are reliable in the case of relatively short waves (e.g. wind generated, typical peak wave period of approximately 5 to 7 s). However, when swell conditions are simulated (peak wave periods in the range from 8 to 12 s) wave height predictions are less accurate. Typically, the breaking is occurring too far seaward. The breaker delay function (Roelvink et al., 1995) can improve wave height conditions considerably for these kind of wave conditions. There has been relatively little experience with the breaker delay function.

Flow module
The flow module is a Quasi-3D model which takes effects of wind, breaking waves, surface slopes (tidal forcing) and streaming in the wave boundary layer into account (Roelvink and Reniers, 1994). A parabolic distribution of the eddy viscosity is used which takes turbulent energy from different sources into account in a consistent manner. The vertical and horizontal distribution of the cross-shore velocities have been evaluated extensively in laboratory experiments. In general the agreement with measurements is reasonable. However, deviations are often present on top of the bar and the trough behind the bar. It is thought that the major cause for these deviations is due to the driving forces from the wave model. Undertow predictions can usually be optimised by tuning the wave model.

The simulated longshore velocities have not been verified to the extend as the cross-shore velocities. Comparison with field measurements at the Coast3D site near Egmond have shown significant deviations with measurements (Walstra et al., 1999). These deviations are mainly due to the fact that the Egmond site can not be considered longshore uniform. Furthermore, the longshore velocities are determined locally, the effects lateral distribution of momentum are not taken into account. This effect which induced longshore shear is thought to be another reason for the relative large deviations that have been observed.

Morphological modules
The sediment transport model was developed by Van Rijn (1995), it has only been verified against a limited number of cases. Comparison is hampered by the limited availability of high quality suspended sediment measurements in laboratories and in the field. The morphological profile development predicted by Unibest-TC has been extensively verified against numerous laboratory and field experiments. A weakspot in the model is the very shallow surf and swash zone. Inaccurate predictions in this area can influence the profile

development in deeper water. Accurate wave height and flow predictions are necessary but are no guarantee that the morphological predictions are realistic. This always has to be verified, if this is impossible a prediction range should be given based on sensitivity simulations.

5.3.2 Predictive capabilities based on validation

The predictive capabilities of a model are often implicitly based on the model performance in a hindcast or is based on results obtained from previous studies. However, to assess the predictive capabilities of a model, the sensitivity of the model results to small variations in the input should also be determined.

If very good agreement with measurements is obtained in the calibration and validation phases. But, the sensitivity of the model results to small variations in the input is large. The predictive capability is still low.

It is emphasised that both checks have to be made to assess the predictive capabilities of the model.

Another important aspect in this regard are the time interval over which the calibration and validation are carried out. If these are relatively short compared to the time interval over which a prediction is made the predictive capabilities of the model decrease (the opposite also applies).

A general rule is to calibrate and validate over approximately the same time interval as over which the prediction is required. If this is not possible, the user has to be aware of a decrease in the predictive capabilities which explicitly should be addressed by performing sensitivity simulations with which error ranges of the predictions can be obtained.

6 Interpretation of model results

6.1 Introduction

This chapter contains a description of the various methods to interpret the Unibest-TC model results. Often, the interpretation of model results is only based on a visual inspection. This may be sufficient during some phases of a project (e.g. parts of the calibration), but only general qualifications can be derived which are often subjective as well. It is important to derive quantitative qualifications which can be verified objectively (e.g. volume changes in a certain part of the bottom profile). The specification of model performance statistics (MPS's), which are applied when evaluating the model based on a direct comparison with measurements, are also described in this chapter. Furthermore, an overview is given of specific interpretation methods for various types of model results.

Because the Statistical Analysis Tool (SAT) is designed to operate from with in the newly designed benchmarking database for Unibest-TC (Roelvink et al., 2000) a short introduction about the structure and application of this database is given in the following section.

6.2 Benchmarking database for Unibest-TC

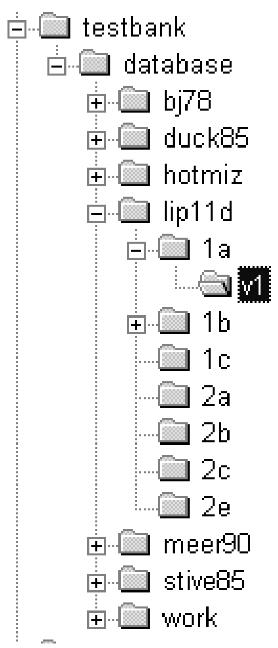


Figure 6.1 Overview if data structure of benchmarking database.

The database is aimed at facilitating a simple and transparent comparison between measurements and model predictions. It has an open structure which enables users to insert specific data sets in order to utilise all the visualisation and error analysis tools. The database is sub-divided in separate data sets, in which a number of cases are defined. The LIP11D experiment is a good example of this approach. Under the LIP11D directory a number of case directories are present (1a, 1b, etc.) in which for each case all the applicable measurement data is collected. Data is stored in ASCII-files according to the very simple and transparent TEKAL-format. Each data file contains only one physical parameter. The file names are standardised:

- <param>.tek for f(x) (not time dependent)
- <param>Xxxx.tek for f(z) or f(t) at location xxx
- <param>Tttt.tek for f(x) at time ttt

The data for the model runs is located under the case directory. In the example shown above the directory "*v1*" contains all the Unibest-TC input files and Unibest-TC will also be executed at this location. As the temporal and spatial coverage of the model output is obviously more dense compared to the measurement data, the model output will be reformatted so that the model data has exactly the same data structure and file name convention as the measurement data.. In the "*v1*"-directory the base runs are located, to perform more runs just simply copy the "*v1*"-directory to a new name and modify input, etc..

6.3 Statistical Analysis Tool

A shell (see Figure 6.2) has been constructed which enables an efficient use of the database. With the shell it is possible to set-up a sequence of Unibest-TC runs and the subsequent statistical analysis and visualisation. Furthermore, the SAT results of various model runs can be compared, saved for later use and visualised. In the model directory the Statistical Analysis Tool (SAT) can be used to investigate the model performance for the required parameters. The SAT interpolates the calculated values to the measurement locations or time points and performs a number of statistical operations and saves the results to file.

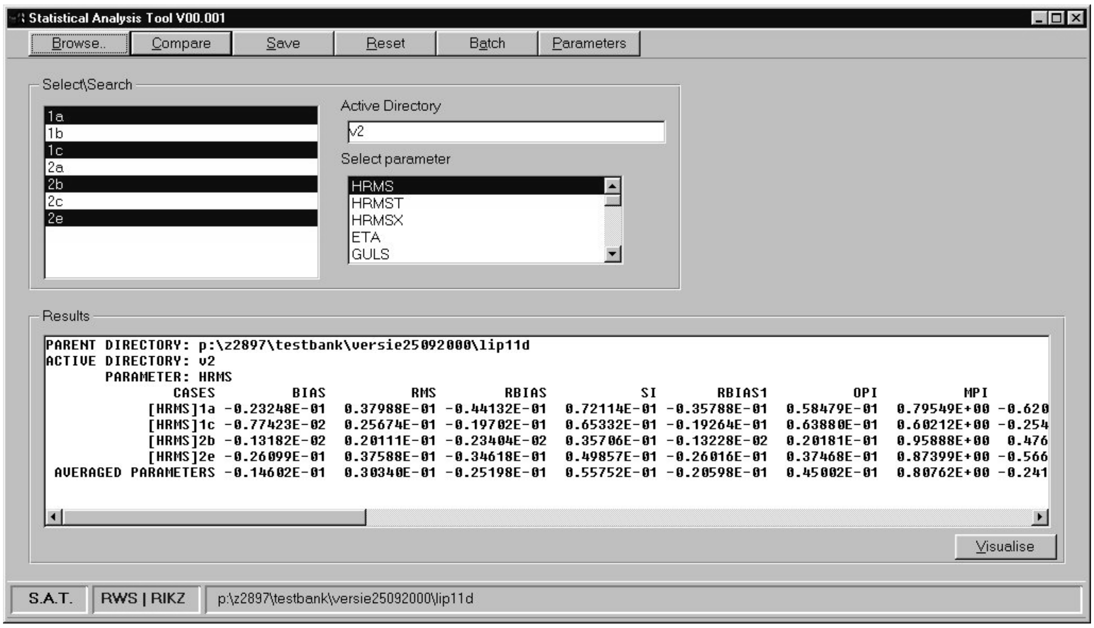


Figure 6.2 SAT-shell of benchmarking database for Unibest-TC

With the [BROWSE] button the user has to select the parent directory which is displayed in the status bar at the bottom of the window. In the top left list box a listing is given of the directories in the parent directory. With the "active directory" the user can specify the working directory. With the "select parameter" list box the user can select an output parameter from which the statistical error ranges are collected.

If the selection is made according to Figure 6.2, pressing [COMPARE] will results in a search of the highlighted directories within the parent directory (*1a, 1c, 2b* and *2e*) for a directory "*v2*" with MPS's concerning the selected parameter (*HRMS*). The results of the search are printed in the "*results*" window. As can be seen in Figure 6.2 the results window also displays an average over all the MPS's that have been found. By pressing [SAVE] these results can be stored in ASCII-file. Note that multiple searches (e.g. for HRMS and ETA) can be stored in one output file. The [RESET] button will empty the search buffer. With the [VISUALISE] button below the results window the MPS's can be visualised in bar charts.

The [BATCH] button creates batch files to run Unibest-TC in the "*v2*" directory residing under the selected directories (*1a, 1c, 2b* and *2e*). The Unibest-TC batch file (extension *btc*) should be loaded into Batch-TC from which Unibest-TC can be run in batch. Also a normal batch file is created which arranges the data extraction, statistical error analysis and visualisation with MATLAB. Note that in this case the "*parameter selection*" is not used.

The [PARAMETER] button creates a batch file to modify the parameter settings in the input for Unibest-TC in the "*v2*" directory residing under the selected directories (*1a, 1c, 2b* and *2e*). In the work directory a file called "key.inp" is present which can be edited. All the settings in the selected Unibest-TC input files will be over-written according to the settings in the "key.inp" file.

6.4 Model Performance Statistics

6.4.1 Introduction

The question of how good a model is should be defined in a more quantitative matter than the usual qualitative ranking (reasonable, good and excellent) that is normally applied. Model Performance Statistics (MPS's) have been used before so the existing statistics are possible solutions to the problem of how to compare model performance to data. Note, however, that it is not possible to make any objective conclusions without a full discussion of the measurements and their inherent errors. Moreover, the degree of model tuning will play a role in determining the performance of the model.

Zambresky (1989) conducted a verification study of the global WAM model, which involved comparing time series of statistics measured at offshore buoys to time series of model parameters. Some of the compared statistics were:

- Mean value and bias (difference of means),
- Standard deviation,
- Root mean square error,
- Scatter Index (SI) which is the RMS-difference (between modelled and measured values) divided by the average measured value.

Three model performance statistics were used in a recent paper (Ris et al., 1999) aimed at verifying the performance of the coastal area wave model, SWAN. They were:

- Scatter Index (SI) from Zambresky,
- Operational performance index (OPI) which is the RMS-difference divided by incident observed value,
- OPI values tend to be low (at least for wave height) as boundary values are high,
- The Model Performance Index, MPI, was defined in terms of RMS-difference and RMS-changes.

6.4.2 Implemented statistics in SAT

Let $f_{meas,i}$ be the measured value and $f_{comp,i}$ be the model output value, with $i=1,....,N$. Where $n=1$ is the offshore boundary value used to drive the model (i.e. $f_{comp,1} = f_{meas,1}$).

The following statistical parameters have been implemented:

The bias, which is the systematic error:

$$BIAS = \frac{1}{N}\sum_{i=1}^{N}\left(f_{comp.,i} - f_{meas.,i}\right) \tag{6.1}$$

The bias should have low values, it can become negative. Large negative or positive values indicate to a poor model performance.

The RMS error:

$$RMS = \sqrt{\frac{1}{N}\sum_{i=1}^{N}\left(f_{comp.,i} - f_{meas.,i}\right)^{2}} \tag{6.2}$$

Similar to the bias, large values for the RMS-error indicate a poor model performance.

As the bias and the RMS-error are absolute statistical parameters, the resulting values should always be interpreted by taking magnitude of the measurements and model results into account. By making both relative to the averaged measured values more objective parameters are obtained.

Relative bias:

$$RBIAS = \frac{\sum_{i=1}^{N}\left(f_{comp.,i} - f_{meas.,i}\right)}{\sum_{i=1}^{N} f_{meas.,i}} \tag{6.3}$$

and the relative RMS-error, which is usually referred to as the Scatter Index (SI):

$$SI = \frac{\sqrt{\frac{1}{N}\sum_{i=1}^{N}\left(f_{comp.,i} - f_{meas.,i}\right)^{2}}}{\left(\frac{1}{N}\sum_{i=1}^{N} f_{meas.,i}\right)} \tag{6.4}$$

Low values of SI indicate a good model performance. However, the SI can become quite high where the average measured value is low, which is quite often the case for coastal engineering, but is less of a problem offshore where the statistic was derived.

Figure 6.3 present relative biases for Guss of the LIP IID data set. *RBIAS* was computed for runs with breaker delay ('v1', light columns) and without breaker delay ('v2', dark columns). In all cases, *RBIAS* is positive, which implies that the model overpredicts the measurements. In all cases, differences between preditions with and without breaker delay are small. On overage, *RBIAS* is about 0.75 for the runs with breaker delay, implying that on average Guss is overpredicted by 75%. Without breaker delay, the average overprediction reduced to about 55% (*RBIAS* approximately 0.55).

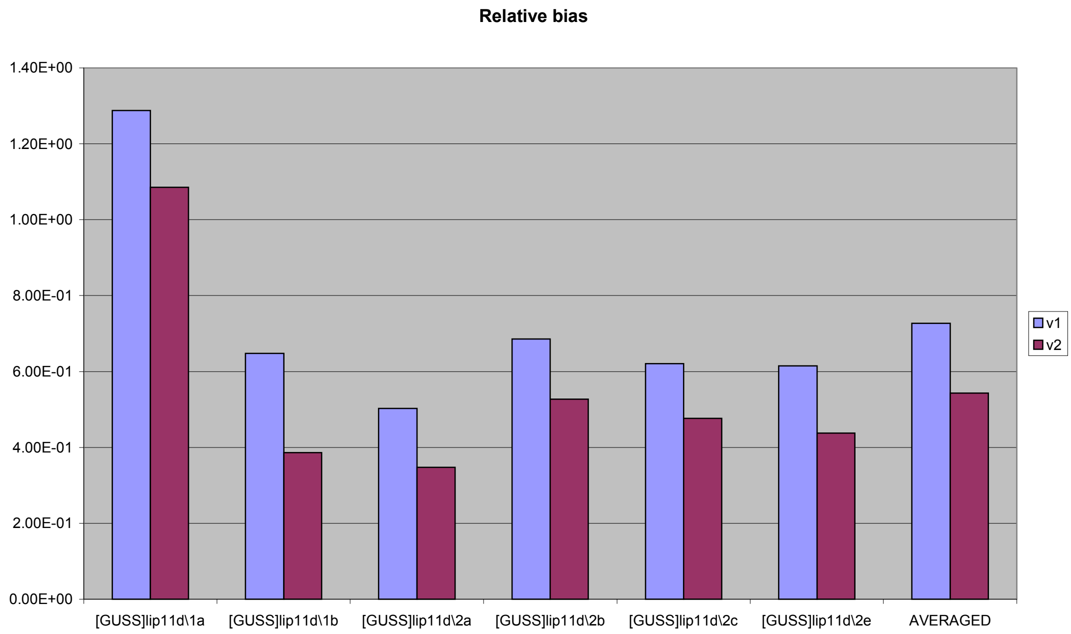


Figure 6.3 Relative bias for Guss in the LIP IID dataset for simulations with (v1) and without (v2) breaker delay (Roelvink et al., 2000).

As some parameters inside the domain have a strong correlation with the observed values at the boundary (e.g. wave characteristics), the statistical parameters are made relative by dividing them with the observed value on the model boundary.

The systematic error relative to the boundary value, *RBIAS1*:

$$RBIAS1 = \frac{\frac{1}{N-1}\sum_{i=2}^{N}\left(f_{comp.,i} - f_{meas.,i}\right)}{f_{meas,1}} \tag{6.5}$$

The RMS-error relative to the boundary is usually referred to as the Optical Performance Index (OPI):

$$OPI = \frac{\sqrt{\frac{1}{N-1}\sum_{i=2}^{N}\left(f_{comp.,i} - f_{meas.,i}\right)^2}}{f_{meas.,1}} \tag{6.6}$$

Figure 6.4 shows the *OPI* values for a total of 39 cases. *OPI* values are shown both for the model runs with breaker delay ('Series 1', light columns) and without breaker delay ('Series 2', dark columns). In several cases inclusion of the breaker delay improves model predictions. For instance, for the Duck85 data OPI reduces between 30 to 50%. In other cases, results with and without breaker delay are almost identical (data Boers) or predictions are better without breaker delay (data Battjes and Janssen). On average, OPI is 0.08, with implies that the standard deviation of the difference between modelled and measured Hrms is about 8% of the offshore Hrms.

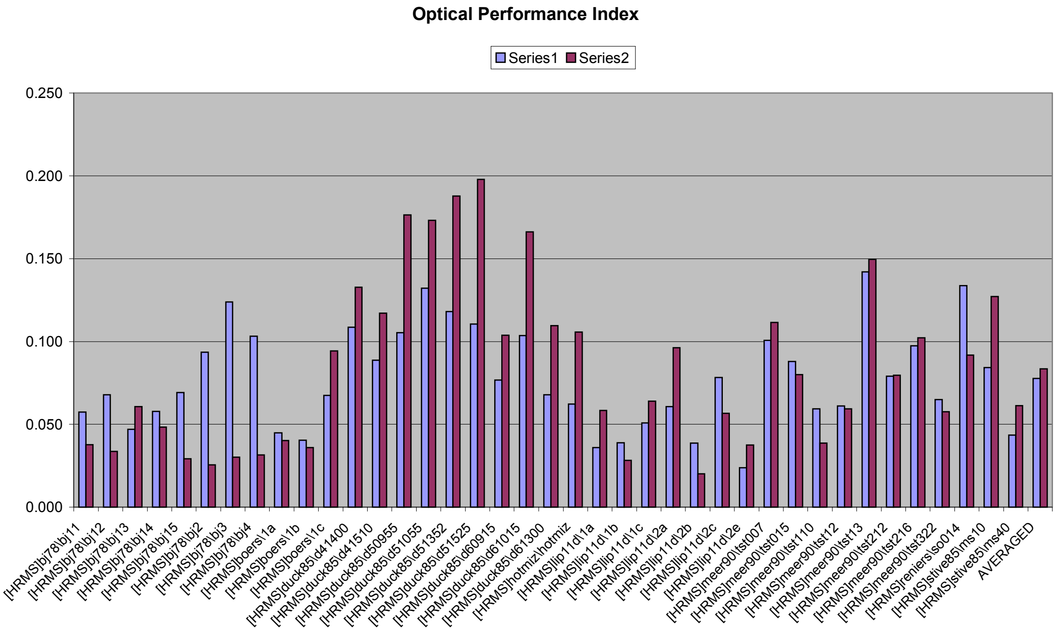


Figure 6.4 Optical Performance Index for several Hrms cases in benchmarking database for simulations with (Series 1) and without (Series 2) breaker delay (Roelvink et al., 2000).

The Model Performance Index (MPI) is defined in terms of RMS-error and RMS-changes:

$$MPI = 1 - \frac{\sqrt{\frac{1}{N}\sum_{i=1}^{N}\left(f_{comp.,i} - f_{meas.,i}\right)^2}}{\sqrt{\frac{1}{N}\sum_{i=1}^{N}\left(f_{meas,i} - f_{meas,1}\right)^2}} = 1 - \frac{RMS_{error}}{RMS_{changes}} \tag{6.7}$$

In the cases considered by Ris et al. (1999) the $f_{meas,n}$ and $f_{comp,i}$ are the measured and modelled significant wave heights or wave periods at a number of buoys at the same time, with $f_{meas,1}$ the offshore wave height (or period) used to drive the model so $f_{meas,1} = f_{comp,1}$. $f_{meas,0}$ is omitted from the averaging of the $f_{meas,i}$ values. Note that the RMS-difference between the modelled and measured values is due in part to the errors in the measurements as well as to the errors in the modelling.

Consider the contrasting cases of a set of wave height measurements made outside the surf zone and another set made over the inner bar of a double-barred beach for the same offshore wave conditions. If the RMS-error in wave height in both cases is 0.25m but the average wave height is 5.25m offshore, 5m just outside the surfzone and 1m over the inner bar then the respective Scatter Index values are 0.05 and 0.25. The second case is much more complicated to model than the first and the RMS-error is no worse than in the first case yet the *SI* gives a much worse value for this case. *OPI* values tend to be low (at least for wave height) as $f_{meas,1}$ values are high. Its failing is therefore the same type as the Scatter Index. In the fictitious example used for the *SI*, the *OPI* values offshore and over the inner bar are both 0.048.

For a perfect model and a perfect dataset, the $MPI = 1$ as $rms_{difference} = 0$. If the model predicts no changes, $f_{comp,1} = f_{meas,1}$ so $rms_{difference} = rms_{change}$ so $MPI = 0$. Alternatively, if $rms_{difference}$ is greater than $rms_{changes}$ then the *MPI* is negative. Note that the value of the *MPI* will decrease if the errors in the measurements increase as well as when the model performance worsens. In the fictitious example above, $MPI = 0$ just outside the surfzone and $MPI = 0.95$ over the inner bar.

The Brier Skill Index (*BSI*) is defined in terms of mean-squared differences between measured values, modelled values and a set of baseline predictions, $f_{base,I}$:

$$BSI = 1 - \frac{\sqrt{\frac{1}{N}\sum_{i=1}^{N}\left(f_{comp.,i} - f_{meas.,i}\right)^2}}{\sqrt{\frac{1}{N}\sum_{i=1}^{N}\left(f_{meas,i} - f_{base,i}\right)^2}} = 1 - \frac{RMS_{error}}{RMS_{base\ error}} \tag{6.8}$$

The baseline prediction can be original measured bathymetry, with $f_{meas,i}$ the measured final bathymetry and $f_{comp,i}$ the predicted final bathymetry. Alternatively the baseline prediction could be a measure of the average climate (so could be the average measured wave height or current at a point). A skill score of 1 implies perfect modelling, a score of 0 implies that you have done no worse than the baseline prediction and a negative score implies that your model results are worse than the baseline prediction. The *BSI* reduces skill for mean, phase and amplitude errors. Summary of implemented model performance statistics:

Parameter	Description	Ranges
Bias	systematic error	can become negative, low values indicate low systematic error
RMS	standard deviation of the error	low values indicate low systematic error
Rel. Bias	systematic error rel. to the mean	low values indicate a good model performance (relative low values for the mean can cause high values)
Scatter Index (SI)	standard deviation rel. to the mean	low values indicate a good model performance (relative low values for the mean can cause high values)
Bias/$f_{meas,}$	systematic error rel. to the start or boundary value	values tend to be low due to high offshore values
Operational Performance Index (OPI)	standard deviation rel. to the start or boundary value	values tend to be low due to high offshore values
Model Performance Index (MPI)	is defined in terms of rms differences and rms changes	for perfect model MPI=1
Brier Skill Index (BSI)	mean squared differences between measured values, modelled values and a set of baseline predictions.	can be used to evaluate improved versions by using predictions of original model as baseline predictions

Table 6.1 Summary of implemented Model Performance Statistics.

6.5 Interpretation methods

In addition to the MPS's that have been given in the previous section (which are also an interpretation method) this section describes a number of interpretation methods which can be used to evaluate the morphological profile predictions made by Unibest-TC.

Cubing methods

Often the stability or "the amount of sand in the profile" are determined by quantifying the volume and volume changes in certain parts of the profile. Two methods can be used to determine the volume changes:

- Horizontal cubing (user has to specify horizontal range, z_{min} and z_{max}, and a vertical reference level, x_{base}; Figure 6.5)

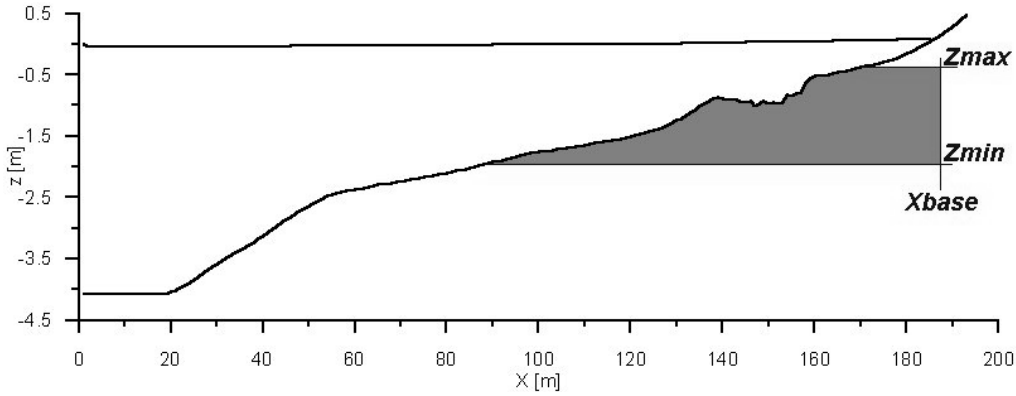


Figure 6.5 Definition of horizontal cubing in Unibest-TC

- Vertical cubing (user has to specify vertical range, x_{min} and x_{max}, and a horizontal reference level, z_{base}; Figure 6.6)

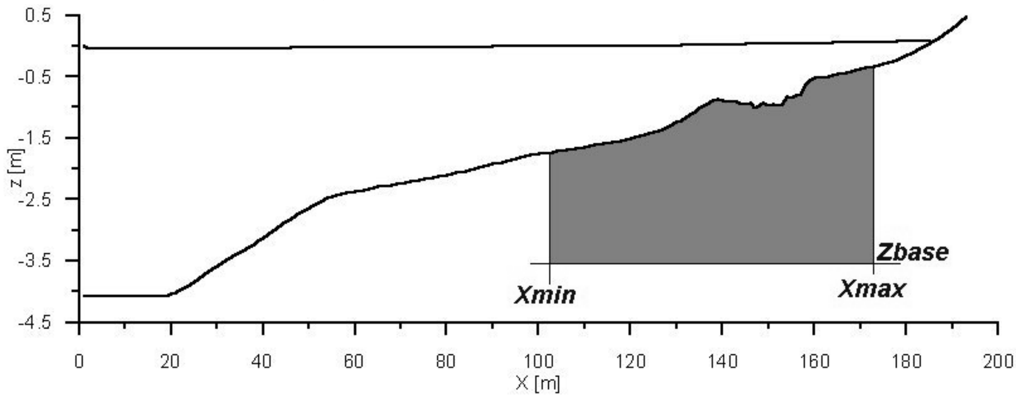


Figure 6.6 Definition of vertical cubing in Unibest-TC

Both cubing methods are standard output of Unibest-TC (in the *vol*-file). Typical output is a time series of volumes. An example is given in Figure 6.7, here a morphodynamic simulation is made over approximately 20 days for the Egmond case which is included in the database. In the top graph the initial and final bottom profile are shown, in the bottom graph the relative volume changes ($X_{base}=5000m$) for a number of vertical sections can be found (relative to initial volume). It can be seen that the shallow region is the most dynamic but seems to stabilise, the deeper regions do not show any significant changes on this relatively short time scale.

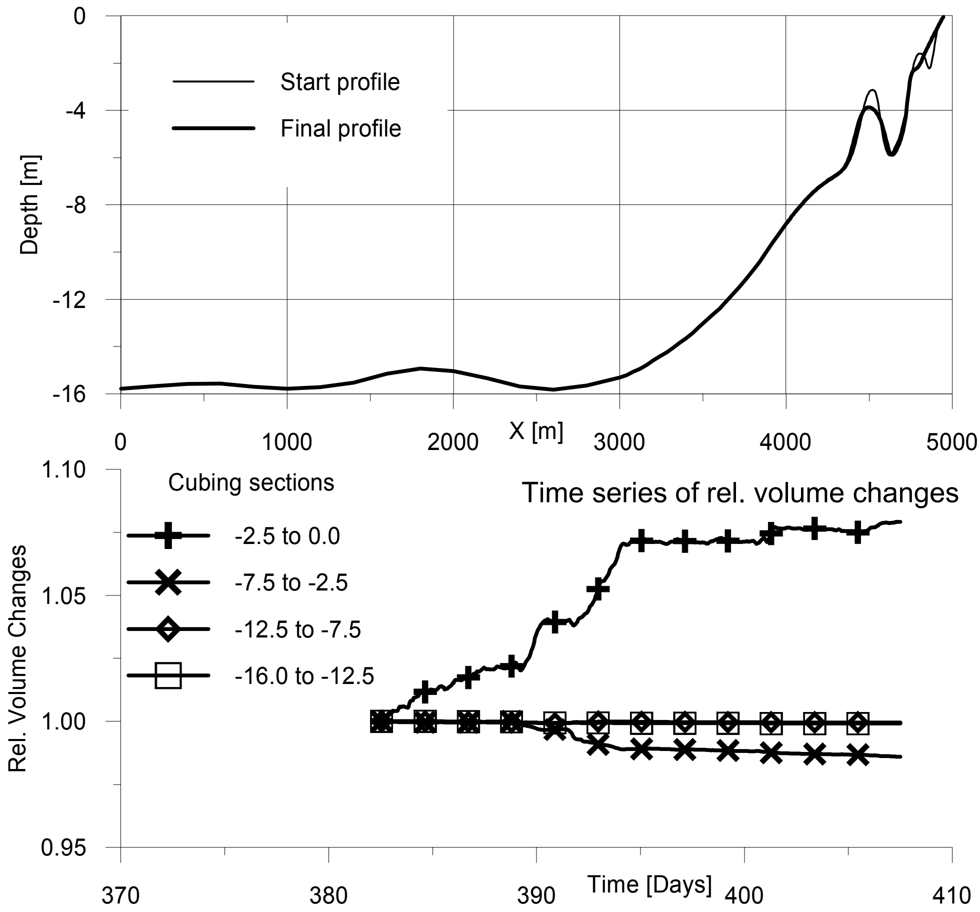


Figure 6.7 Example of horizontal cubing (Egmond case from Testbank).

Monitoring of coastline position

The position of the coastline (defined as *z = 0*) is also standard output of Unibest-TC can also be found as a time series in the *vol*-file. It has to be noted however that the coastline prediction in Unibest-TC should be used with caution as the process descriptions in very shallow water are not included. The model extrapolates the sediment transport in the last considered cell across the dry beach.

Migration and generation of breaker bars

The migration of the breaker bar is a good indication of the morphological variability of the investigated bottom profile. This is best done via a visual inspection of the profile development by using the Ani-TC utility. Generation of a time stack of the simulated bottom profiles is another visualisation option which gives a good impression of the bar migration and generation (e.g. see Figure 6.8, which is standard output of Viz-TC). Unibest-TC has a standard output of the first two encountered minimum and maximum values in the bottom profile (from sea to shore). This is a simplified method of determining the bar crests and bar troughs. The user has to be careful however to check if the maximum and minimum values found by the model are actual bar related features and not small scale ripples. This output can also be found as a time series on the *vol*-file.

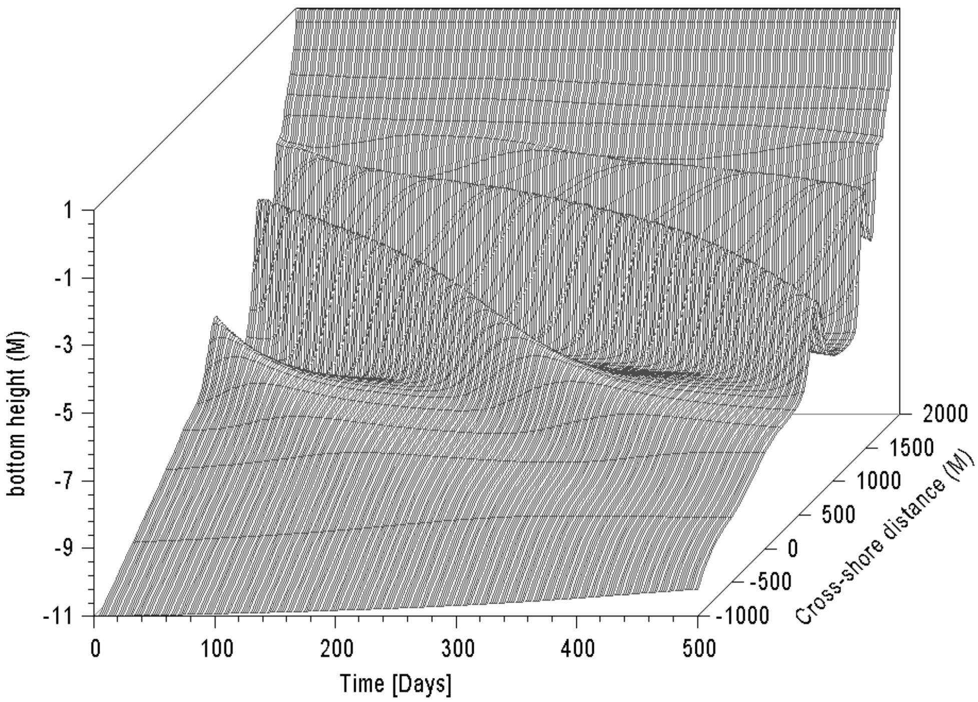


Figure 6.8 Time stack of profile development as simulated by Unibest-TC.

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A Tidal schematisation

A.1 Van Rijn (1993)

Sand transport computations in tidal conditions usually requires representation of a neap-spring tidal cycle. This can be simply done by multiplying the velocities of the mean tidal cycle by a correction factor, ζ, to account for the higher velocities and hence higher transport rates (power relationship of transport and velocity) during spring tide conditions.

In Van Rijn (1993) this correction factor, ζ, is formally derived, assuming a power-law relationship, b, between the sediment transport, q_t, and the depth averaged velocity, $\overline{u}$: $q_t = a\overline{u}^b$.

In the figure below the resulting correction factors as a function of the tidal coefficient, A, (ratio between maximum tidal range and mean tidal range: $A = \frac{H_{\max}}{H_{mean}} - 1$) and the power relationship, b, between sediment transport and the depth averaged velocities multiplied with the power relationship, n, between the maximum depth averaged velocity and the tidal range, H: $\hat{\overline{u}} \approx H^n$ is shown. In coastal conditions (open sea) the n-coefficient maybe as small as n=0.5 but is usually close to n=1. Measurements should be analysed to derive the exact n-value for each specific location (see Van Rijn, 1993, for more details).

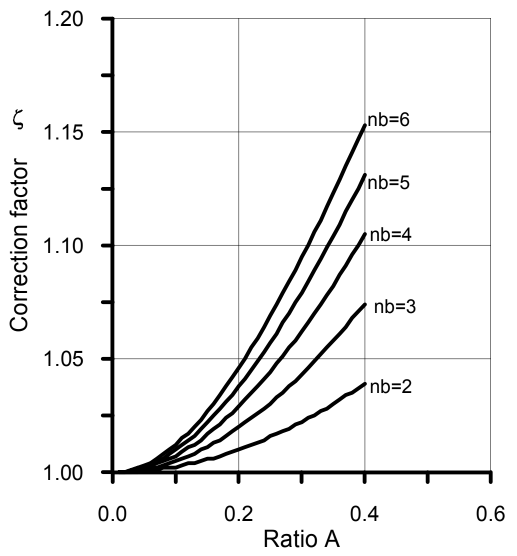

Figure A.1 Correction factor representing astronomical effects

Along the Dutch coast in general a correction factor of order ζ=1.1 is applied (nb=4). It is noted that in the derivation above climatologic effects are ignored, more information on this aspect can also be found in Van Rijn (1993).

A.2 Latteux (1995)

The method of Latteux is based on a statistical description of the performance of a tidal schematisation with respect to the optimal representation of the derived or measured residual (e.g. yearly averaged) sediment transports.

At a number of relevant locations, the residual transports are determined from preliminary simulations or derived from measurements as a reference. Next, one attempts to identify a (short) period in which the residual transports in all considered locations give an optimal representation of the reference transports. In this it is allowed that the residual transports are scaled with *one* factor (the same for all considered locations). The period which has the smallest deviation in residual transports is chosen as the morphological tide. This deviation, or relative error, is determined by averaging over all the considered locations. Here it assumed that each location has the same relevance. It is however also possible to assign a weight to each location to specify their relevance.

Below a pointwise description of the method is given:

1. Calculate (or derive from measurements) the residual (reference) transports, $\overline{S_i}$, in each location.
2. Calculate the running average over a period, τ, which represents one or two tides with a small time step (e.g. 5 minutes) of the residual transports in each location: $\overline{T}_i(t) = \dfrac{1}{\tau} \displaystyle\int_{t-0.5\tau}^{t+0.5\tau} \left(T_i(t)\right) dt$,
3. Determine the ratio, *W(t)*, (averaged over all the considered locations, *N*) between the residual transports, $\overline{T_i}(t)$, and the reference transports, $\overline{S_i}$: $W(t) = \dfrac{1}{N} \displaystyle\sum_{i=1}^{N} \dfrac{\overline{S_i}}{\overline{T_i}(t)}$,
4. Determine the difference between the scaled residual transports and the reference transports for each location: $\overline{\varepsilon_i}(t) = W(t)\overline{T_i}(t) - \overline{S_i}$,
5. Finally, determine the root mean square error (averaged over all locations) as: $$E_{rms}(t) = \sqrt{\frac{1}{N} \sum_{i=1}^{N} \left(\frac{\left|\overline{\varepsilon_i}(t)\right|}{\overline{S_i}} \right)^2} \ .$$

The ratio, *W(t)*, and the rms. error, $E_{rms}(t)$, can now be evaluated to determine the morphological tide. Ideally, one would like to select a tide of which the ratio, *W(t)*, is close to one and the rms. error, $E_{rms}(t)$, minimal. In the figure below a typical distribution of both parameters is shown. From this figure it is easy to select a tide which gives and optimal representation of the reference transports.

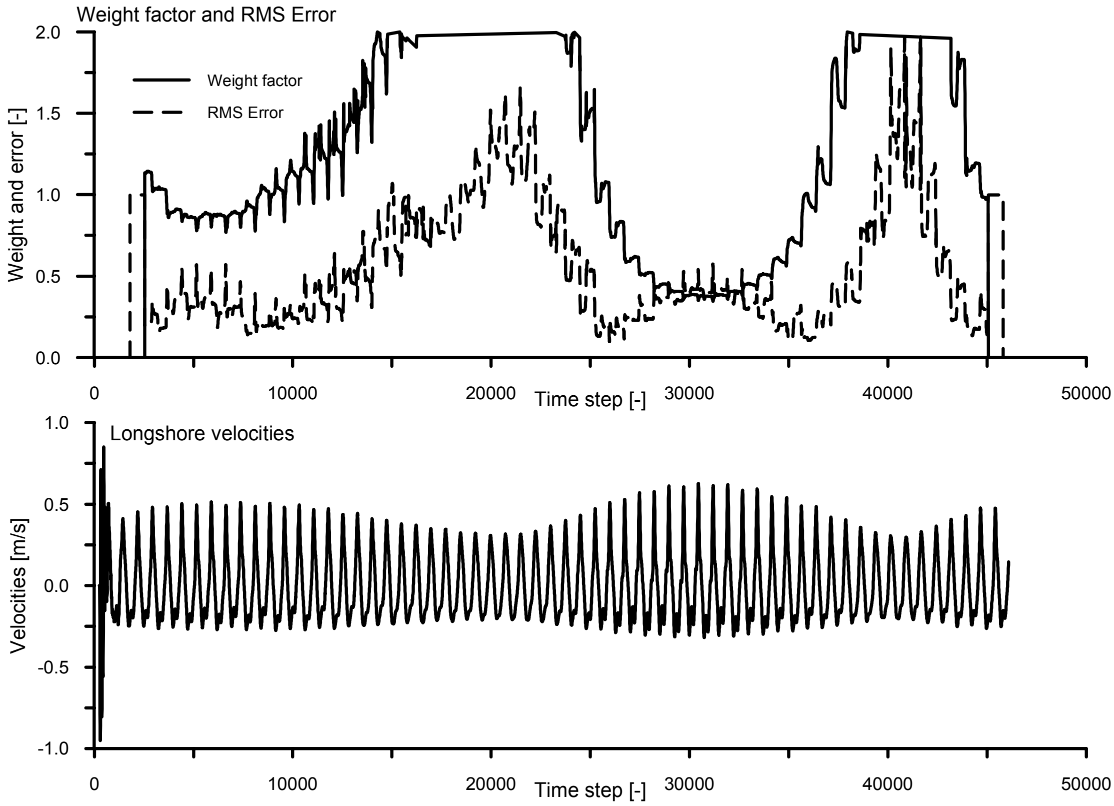

Figure A.2 Example of resulting time series of weight factor and RMS Error.