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Probabilistic model Wadden Sea: Three alternatives for HBC 2011 and test results

WTI - HR Zout

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Title

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RWS - Waterdienst	1200103-057	1200103-057-HYE-0008	43

Keywords

Hydraulic Boundary Conditions, Wadden Sea, Hydra-K, HR 2011.

Summary

This report describes three different approaches for calculating the Hydraulic Boundary Conditions (HBC) in the Wadden Sea area. The three approaches that are described differ in terms of probabilistic method as well as driving mechanisms of the wave model and circulation model:

- The first approach is the most basic implementation, using the present setup of Hydra-K and input for SWAN calculations.
- A second approach uses the present Hydra-K set-up, but takes into account the influence of spatially varying water levels and currents in a pragmatic way. The hydrodynamics of model storms, calculated by WAQUA, are used as input for the SWAN simulations.
- Finally, a third method is described in which meteorological variables form the basis of the probabilistic model. Development of this method requires a redesign of Hydra-K.

The first and second method were implemented and tested in Hydra-K. Results indicate that both methods produce realistic HBC. It is proposed to use the second method for the HBC 2011, because it takes into account the effects of spatially varying water levels and currents. The calculated wave conditions are therefore considered more realistic.

It is expected that the third approach will produce even more accurate HBC. However, implementation and testing of this method requires considerable effort and cannot be done before 2011. It is recommended to start developing this method for HBC 2016.

References

WTI projectplan Deltares, Projectplan HR-Zout

Version	Remark	Date	Author	Initials	Review	Initials	Approval	Initials
1	Draft	9/06/2009	J. Beckers		H. Gerritsen			
2	Final Draft	22/06/2009	J. Beckers		H. Gerritsen		A. de Leeuw	
3	Final	8/07/2009	J. Beckers		H. Gerritsen		A. de Leeuw	
4	Update, draft	12/12/2009	J. Beckers		H. Gerritsen		M. van Gent	
5	Final	30/12/2009	J. Beckers		H. Gerritsen		M. van Gent	

State

final

Contents

Summary (in Dutch)

In het kader van het project WTI zullen de komende tijd Hydraulische Randvoorwaarden (HR) berekend gaan worden. Dit betreft o.a. de harde keringen rond de Waddenzee. Dit document beschrijft drie varianten van (een deel van) de rekenmethode en hun haalbaarheid voor HR 2011. De varianten verschillen qua probabilistische rekenmethode en qua aansturing van het golfmodel SWAN en hydrodynamisch model WAQUA.

- De eerste aanpak is de meest eenvoudige. Er wordt gebruik gemaakt van de meeste recente versies van de modellen Hydra-K en SWAN. De berekening is verder analoog aan die van de HR2006 voor de Hollandse kust. In de SWAN berekeningen wordt een vlakke waterspiegel aangenomen en stroming wordt verwaarloosd.

- De tweede methode gebruikt het huidige Hydra-K model, echter met een gewijzigde opzet van de SWAN berekeningen. De stroming en variabele waterstand voor elke SWAN berekening wordt gegenereerd door WAQUA simulaties van modelstormen. Hierdoor levert deze aanpak naar verwachting meer realistische randvoorwaarden. De testresultaten in dit rapport geven aan dat de hydraulische randvoorwaarden betrouwbaar zijn. Daarmee is de methode geschikt voor de productieberekeningen voor HR 2011.

- Een derde methode wordt beschreven, waarin het probabilistische model wordt aangepast. De statistiek van meteorologische omstandigheden bepaalt in dit model de statistiek van waterstanden en golfrandvoorwaarden. Deze aanpak zou de voorkeur hebben als voldoende tijd beschikbaar is voor onderzoek en ontwikkeling. Voor HR 2011 wordt een grondig herontwerp van Hydra-K echter niet haalbaar geacht.

Aanbevolen wordt om de tweede methode te gebruiken voor de productieberekeningen van HR 2011. De golfcondities berekend met deze methode zijn realistischer dan die van methode 1.

Verwacht wordt dat de derde methode nog nauwkeuriger HR zal opleveren. Gezien het lange ontwikkeltraject wordt aanbevolen om onderzoek naar deze aanpak in 2010 te beginnen en te richten op gebruik voor HR 2016.

1 Introduction

1.1 Background

In compliance with the Flood Defences Act ("Wet op de Waterkering, 1995"), the primary coastal structures must be monitored every five years (1996, 2001, 2006, 2011, etc.) for the required level of protection. This assessment is based on the Hydraulic Boundary Conditions (HBC) and the Safety Assessment Regulation (VTV: Voorschrift op Toetsen op Veiligheid). The HBC represent the hydraulic load (water level and wave conditions) that a flood defence must be able to withstand. The HBC are derived every five years and need to be approved by the Minister of Transport, Public Works and Water Management.

Following these regulations, the HBC for the closed coastline of Holland and the Zeeland Delta have been recalculated in 2006, using the probabilistic model Hydra-K and the wave transformation model SWAN (Booij et al., 1999, SWAN homepage http://www.fluidmechanics.tudelft.nl/swan). For the Wadden Sea, however, no recalculation was done, because of lack of data for calibration and validation of SWAN in the Wadden Sea. At the same time, the quality of the current HBC for the Wadden Sea has been questioned, because they were derived from construction studies (WL 2002).

Within the project "Strength and Load of Coastal Structures" (SBW: Sterkte en Belasting Waterkeringen) the models and methods that are used to derive the HBC have been investigated in great detail. The SWAN model was calibrated, validated and where possible improved to achieve maximum performance in the Dutch Wadden Sea, using all available data from that area and from elsewhere. Several hindcasts of storm events in the Wadden Sea have been performed (WL 2006; Haskoning 2006; Alkyon 2007a, b) as well as sensitivity and uncertainty analyses (WL 2007a, b; Haskoning 2008; Witteveen&Bos 2008; Alkyon, 2008b).

Now, for the fourth assessment round starting in 2011, the general opinion is that there is sufficient confidence in the SWAN results in the Wadden Sea. The aim is therefore to base the HBC 2011 on the SWAN and Hydra-K models. In order to do so, a probabilistic model within Hydra-K needs to be set-up for the Wadden Sea and connected to a database of SWAN results. This could easily be done by using a similar probabilistic model as is used for the North Sea coastline. However, there are indications that the wave conditions in the Wadden Sea are subject to influences from hydrodynamical effects (Alkyon, 2008a). If possible, these should be taken into account in the HBC calculation. In this study, methods will be investigated to take into account such effects for the HBC 2011.

The HBC calculation for the Wadden Sea is done within the framework of the project "Wettelijk Toetsinstrumentarium" (WTI). Given the time schedule for the HBC 2011, the probabilistic model for the Wadden Sea will have to be based on the present Hydra-K model. Minor modifications can be made as long as the model is ready and tested before the end of 2009. More extensive adjustments or even a reconsidering of the fundamentals of this probabilistic model will have to be done within the SBW project, which aims at improving the calculations for HBC 2016.

1.2 Aim

The aim of this document is to describe three different approaches and test results for calculating the Hydraulic Boundary Conditions in the Wadden Sea area. The approaches that are described differ in terms of probabilistic method as well as driving mechanism of the wave model and circulation model:

- The first method is the most basic implementation, using the present setup of Hydra-K and input for SWAN calculations.
- A second approach uses the present Hydra-K set-up, but the hydrodynamics of the SWAN input are generated with WAQUA simulations of model storms.
- Finally, a third method is described in which the meteorological variables form the basis of the probabilistic model. Development of this approach requires a redesign of Hydra-K.

A previous version of this report, which described the three different approaches, was submitted to Rijkswaterstaat Waterdienst in July 2009. After approval of that report, the activities for development and testing started in August of 2009. This report is an update of the previous version. It includes the test results and an advice which method to use for the HBC 2011 calculations.

1.3 Contents of the report

In Chapter 2 an overview of the present HBC calculation method is given. The three alternative probabilistic approaches to calculate the HBC are elaborated in Chapter 3. The test results of the first approach are given in Chapter 4. A further description and implementation of the second approach are outlined in Chapter 5. In Chapter 6 the test results are reported. Chapter 7 gives conclusions and recommendations.

2 HBC calculations using Hydra-K and SWAN

The HBC represent the hydraulic load (water level and wave conditions) that is expected to be exceeded once in a certain return period. For the coast of Holland this return period is 10,000 years. For the Frisian and Groningen Wadden Sea coast it is 4,000 years for the mainland and 2000 year for the islands. The HBC for locations along the Dutch coast are computed using Hydra-K. Hydra-K uses a probabilistic method, based on observed characteristics of storm events and empirical probability functions of wind speed, wind direction and water level. This method generates combinations of extreme water level and wind occurrences, using the assumption of asymptotic dependency, i.e. that the observed correlations between these variables also hold for extreme conditions (De Haan and Resnick, 1977). The probabilities of water level and wind speed are based on the Basispeilen and Rijkoort-Wieringa statistics respectively. Figure 1 shows an overview of the calculation.

An important part of the HBC calculation is the calculation of nearshore wave conditions for a given combination of wind and water level at the location of interest. For this purpose Hydra-K uses the wave simulation model SWAN, or more precisely, a database with pre-calculated SWAN results. For several combinations of local water level, wind speed and wind direction, the nearshore wave conditions are pre-calculated using SWAN. The SWAN results are stored in a database (indicated in bold in Figure 1), which forms a transformation matrix. This matrix is used as a lookup table for transformation of a water level at a location of interest, wind speed and wind direction to the nearshore wave conditions.

Hydra-K can compute the HBC for different failure mechanisms, such as wave run-up, or damage of the dike revetment. The HBC differ between the failure mechanisms, because these mechanisms are sensitive to different parameters.

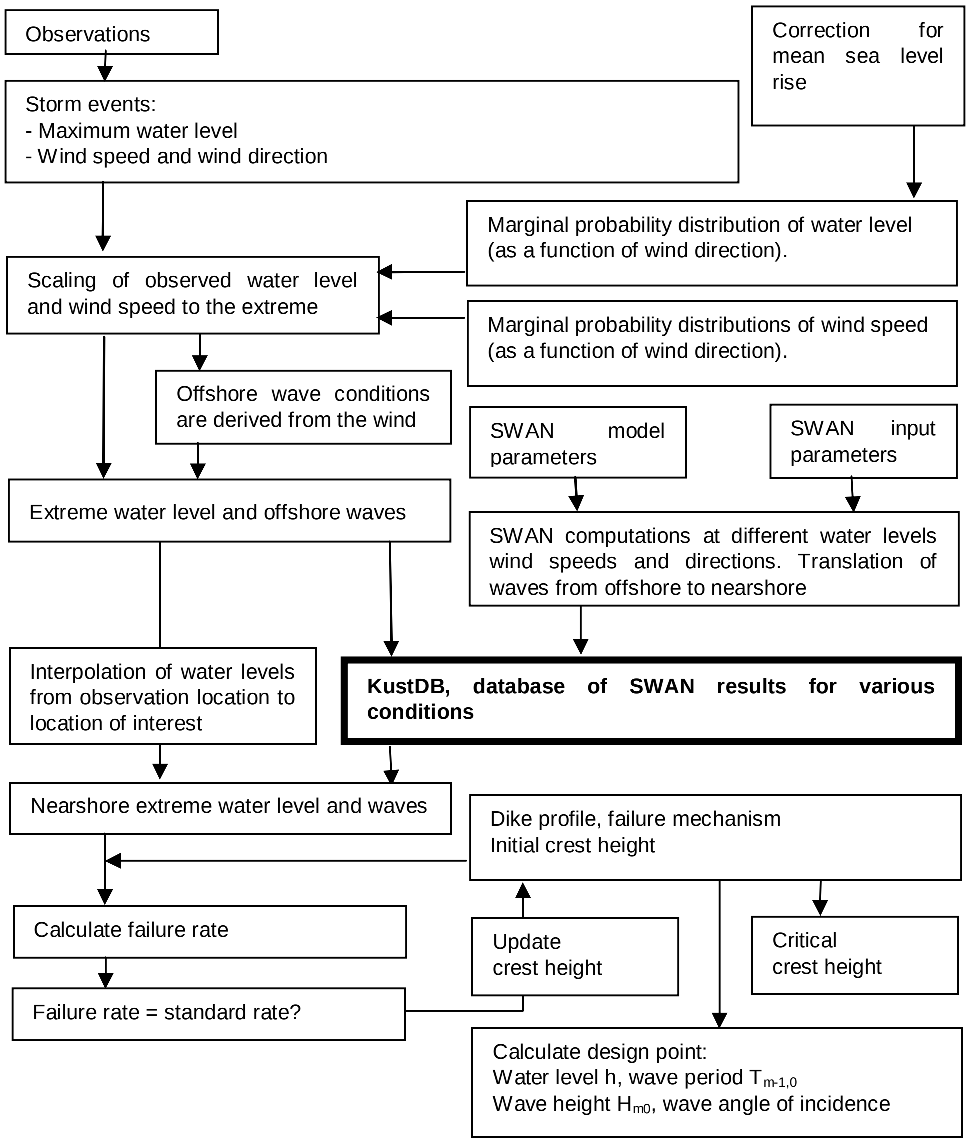


Figure 2.1 Schematic overview of the HBC calculation for the Wadden Sea.

3 Description of the three approaches

3.1 Method 1: Standard implementation

The most basic approach to calculate the HBC in the Wadden Sea is to connect the most recent SWAN model to the latest version of Hydra-K (version 3.5.2) without further improvements. This implies the following settings of the Hydra-K and SWAN models:

- Constant wind speed and wind direction in the SWAN calculations;
- Wind speed statistics based on Rijkoort-Wieringa (or, if established, a more recent update).
- Water level statistics based on the Basispeilen (RIKZ, 1995) at Den Helder, Harlingen and Delfzijl, using a linear interpolation to the HBC locations;
- Offshore wave conditions are deterministically derived from wind speed and direction.
- Most recent bathymetry
- Uniform water level field and neglect of currents

For the test calculations a large number of different wind speeds, directions and water levels were considered:

- 15 wind directions: between 30 and 360 (30˚ intervals) and 285, 315, 345˚.
- 6 absolute wind speeds: 15, 20, 25, 30, 35 and 40 m/s
- 7 constant water levels (in space and time): 1,2,3,4,5,6 and 7 m + NAP

The number of combinations could be reduced, saving computing time without loss of accuracy. This will not be elaborated further in this study.

Method 1 will serve as a reference for comparison to method 2 (see below) and as a fall-back option for HR 2011. The approach is similar to that for the coast of Holland. Effects of currents, water level variations or time dependency, such as changing wind direction are not included. These effects are considered to be important in the Wadden Sea. Method 1 should therefore be adopted as the official HR 2011 only if the development of the more advanced method 2 fails.

No problems were encountered in the development of method 1. HBC test calculations have been carried out using SWAN version 40.72AB i.e. including the bi-phase breaker model (Deltares, 2009a). Corrections for low frequency wave penetration, as proposed in Deltares (2009c), have not been considered. These corrections have no consequences for the coupling to Hydra-K.

3.2 Method 2: Including hydrodynamic effects

WL (2007c, 2007d, 2007e, 2008) recommended taking into account effects from currents and spatially varying water level in the wave propagation calculations for HBC. More recently, Alkyon (2008c) has shown that currents can have a significant influence on the nearshore wave conditions.

However, there is some reluctance to modifications of the Hydra-K program, because the basic modelling concept of Hydra-K is rather inflexible and the software is highly complex. Method 2 therefore aims to take into account the effects of currents and spatially varying water level in the SWAN calculations, without modifying the Hydra-K software.

Spatially varying current and water level fields are calculated by the hydrodynamic flow model WAQUA. Both WAQUA and SWAN are driven by time-varying wind fields. A steady-state wind forcing is not considered a realistic approximation, because dynamic effects play a significant role in the Wadden Sea (Alkyon 2008a, 2009). For example, the highest water levels at Delfzijl are typically observed if a western storm drives the water into the eastern Wadden Sea and then turns northwest to north, forcing the water into the Eems-Dollard estuary.

By allowing the wind and water level to vary in time, the number of possible combinations increases dramatically. The standard approach for this would be to define a set of determining stochastic variables that describe the temporal variation of wind and water level, determine their (correlated) probability distributions and make a probabilistic calculation. However, this approach would require a redesign of the probabilistic model Hydra-K, which is not considered feasible before 2011 (see discussion under method 3).

Therefore, an alternative method is proposed that is based on the current Hydra-K model. In the current setup of the SWAN calculations (and also in method 1), the water levels and wind velocity (speed and direction) are assumed constant in time. In method 2, a time evolution of a model storm is introduced. As a consequence, the WAQUA and SWAN simulations produce time series of currents, water levels and wave conditions. The water levels and wind are no longer constant in space and time. Whereas in method 1 they form a regular matrix, this is no longer the case for the simultaneous samples of wind and water level in method 2. However, a regular matrix can be constructed from the irregular samples by interpolation. Recall that the SWAN database is merely a lookup table of wave conditions for a given set of water level and wind. There is no reference to probability or statistics. The only requirements to the simulations are that they produce realistic conditions and that they span the range of water level, wind speed and wind direction that is required by Hydra-K.

There is, however, a subtle difference between the Hydra-K definition of the water level and the water level from the WAQUA and SWAN simulations. Hydra-K uses observations and statistics of maximum water levels, whereas the water levels from the WAQUA and SWAN simulations are also taken at other time instants during a storm. This discrepancy introduces an error if the wave conditions at the time of the water level peak differ systematically from the wave conditions at other time instants. If this error exists, it is thought to be small for three reasons:

- Firstly, the wave conditions in the SWAN database are taken from time instants up to two hours from the peak of the storm (wind maximum). In many cases this will correspond to the conditions at the water level peak.
- Secondly, if the wave conditions at the water level peak differ from those at other time instants, then the latter would be more severe in most cases. At the peak, the water levels of the surrounding area are typically lower. The waves arriving at the location of interest will be lower because of depth induced breaking in the surrounding waters.
- Thirdly, the SWAN database is also used for failure mechanisms other than wave overtopping, for which the peak water level may not be the moment of highest load. For example, failure of dike revetments depends more on wave impact than on water

level. For such failure mechanisms the wave conditions at time instants other than the water level peak are very relevant.

The details of method 2 are discussed in Chapter 5.

3.3 Method 3: Redesign of the probabilistic model

The most advanced approach to treat the time- and space varying wind field is to define additional stochastic variables, which describe the course of the storm, and define a new probabilistic model. The tidal dynamics can also be included by adding the tidal phase and height as stochastic variables. One can think of a set of four stochastic driving forces:

- Wind direction (non-changing wind direction, slowly changing or fast changing);
- Wind speed (slowly or fast increasing and decreasing wind speed);
- Tidal phase (time shift between the peak of the storm and the tide);
- Tidal amplitude.

In this setup the water level is no longer an independent stochastic variable. The local water level is determined by hydrodynamic simulation. The probability of exceedance of a local water level follows from the direction dependent wind speed statistics and the tidal motion. The result can be compared to the statistics from the Basispeilen study.

The multivariate probability distribution must be defined for the set of stochastic variables, taking into account correlations. For example, the changing wind direction and changing wind speed are most likely coupled. This should be reflected in a joint probability distribution.

For each combination of random variables a hydrodynamic simulation can be performed, and the resulting water level and current fields used as input to SWAN. The SWAN simulation also takes the time-dependent and spatially varying wind field as input. The highest hydraulic load at each nearshore location should then be stored in the database, along with the settings of the stochastic variables.

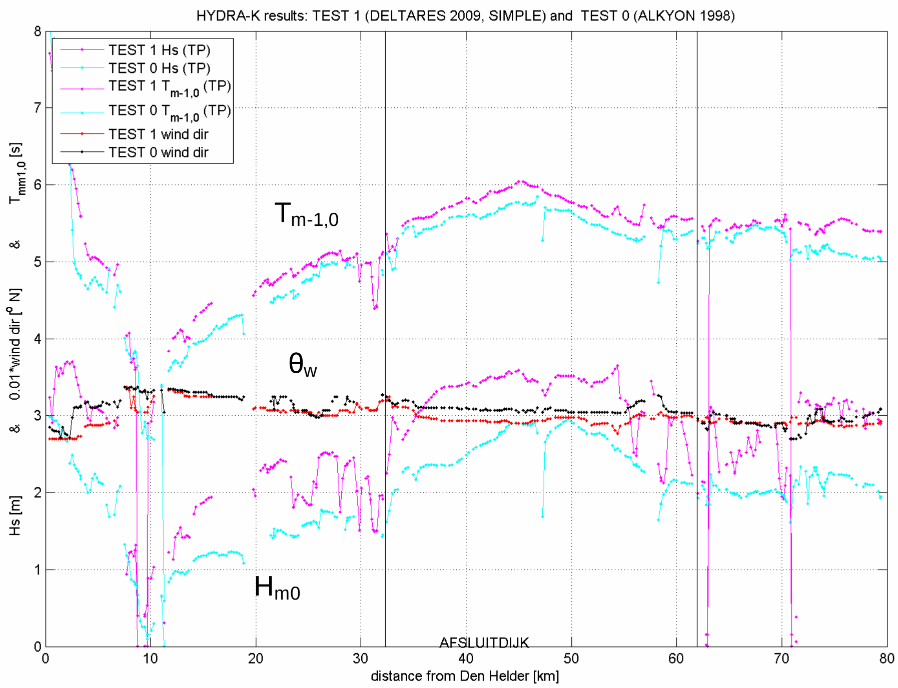
The development of method 3 is too ambitious for HBC 2011. Additional stochastic variables would require significant modification of Hydra-K, which is laborious. Moreover, for each additional variable, the probability distribution and correlations to other stochastic variables must be defined. To model wind fields, and obtaining corresponding statistics, requires many years of research, in a joint effort with experts from KNMI. This cannot be done within the time remaining before the final model must be delivered.

4 Test results of method 1

Method 1 was implemented and used for HBC test calculations. The results were compared to a reference calculation by HKV (2009), called 'test 0'. This HKV study aimed at testing the Hydra-K version 3.5.4, in preparation for HBC 2011. The SWAN data were taken from a database RAND2001, which contains results of SWAN simulations in the Wadden Sea done by (Alkyon, 1999). These calculations were done using SWAN version 30.62. The spectral wave periods ($T_{m-1,0}$) from these calculations were increased by 10% after hindcast studies in the Westerschelde showed that SWAN version 30.62 underestimated the wave period (see e.g. Haskoning, 2003). HKV put these SWAN results in the correct format for usage in Hydra-K and calculated HBC for 1340 locations in the Wadden Sea area.

The test calculations for method 1 (called 'test 1'), uses stationary SWAN calculations, SWAN version 40.72AB and a coarse-grained, curvilinear grid, based on the Kuststrook model. The water level and wind fields were assumed to be uniform in space. Currents were neglected in the SWAN simulations. Uniform off-shore boundary conditions were applied on the northern grid boundary, using a parameterized JONSWAP spectrum, based on wave measurements at ELD, taken from HKV (2005). The HBC locations are identical to the locations in the study by HKV mentioned before. A total of 630 different combinations of wind speeds (6) wind directions (15) and water levels (7) were simulated.

The aim of the 'test 1' exercise was to run the entire model chain, which was successful. The coarse grid that was used in these test runs produces inaccurate model results near the sea defenses. Therefore, a quantitative comparison to the RAND2001-based results is not considered useful. However, a qualitative comparison can still be made. Figure 4.1 shows the differences between the water level and wave height from the HKV study (test 0) and method 1 for all locations along the southern shore of the Wadden Sea. Clearly, method 1 produces higher wave heights than the RAND2001 results, but in general the same relative differences are found between the different locations.



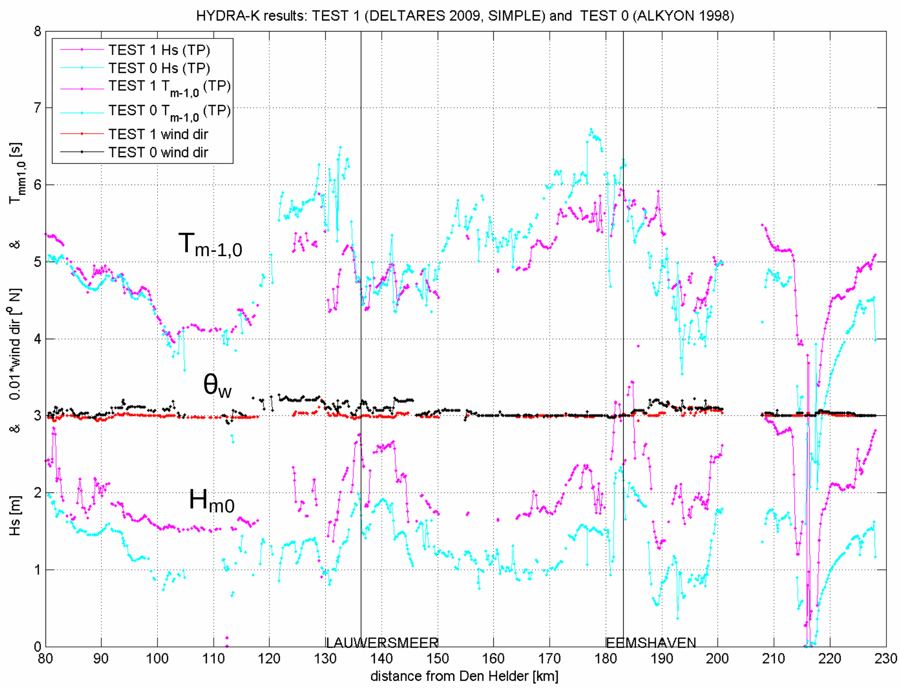


Figure 4.1 *Comparison between: Test 0: Hydra-K results from RAND2001 and Test 1: method 1, using a Kuststrook-grid and a recent SWAN version. The upper Figure shows results for the Western Wadden Sea. The lower Figure is for the Eastern Wadden Sea.*

5 Development of method 2

5.1 Outline

The steps that were taken in the development of method 2 are:

- Develop a standard storm profile that describes the rate of change of wind speed and direction.
- Run the WAQUA and SWAN simulations for different combinations of offshore surge, tidal phase, peak wind direction and peak wind speed.
- Interpolate the SWAN results at various water levels, wind directions and wind speeds to the regular grid required for Hydra-K.
- Perform test calculations. Compare the results to those from method 1 (fixed water level and no currents).

These steps are discussed in detail below.

5.2 Standard storm profile

The rate of changing wind speed and wind direction was taken from an observed set of real storms (Deltares, 2009b). Storms were selected using a threshold of 20 m/s (western storms) or 15 m/s (eastern wind). All selected storms were re-scaled by dividing the wind speeds by the storm peak value. A symmetric trapezium with a top plateau of 1 hour was used to describe to temporal evolution of the wind speed (see Figure 5.1).

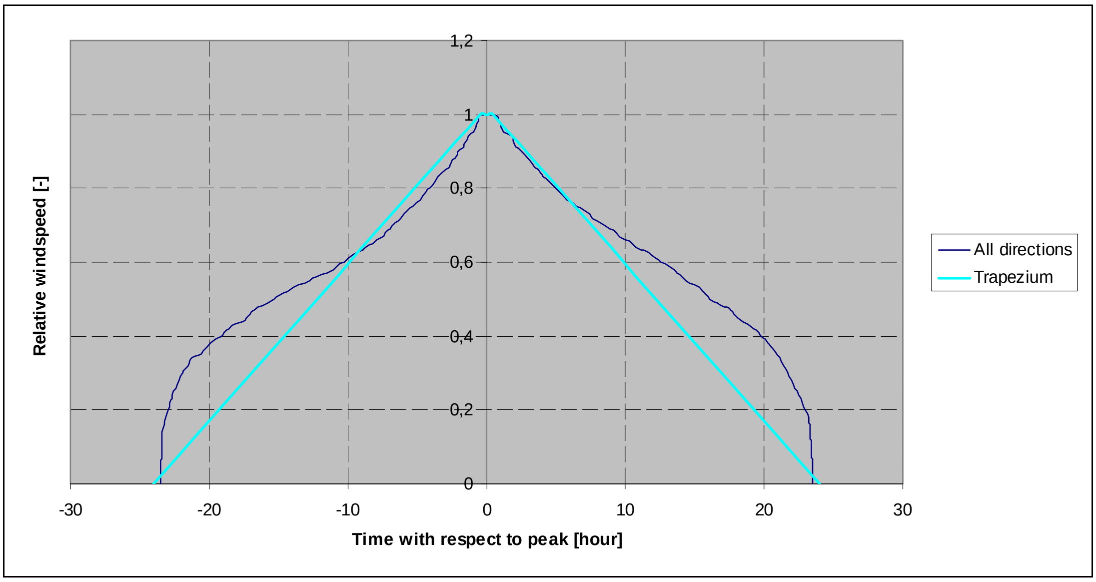


Figure 5.1 Standard temporal evolution of the wind speed, schematized by a trapezium.

The temporal evolution of the wind direction was derived using the change in direction relative to the direction of the peak wind speed. This change in wind direction was averaged over all western storms[1] and an error function was fitted to the result (see Figure 5.2). The schematized temporal evolutions of wind speed and direction were used as input to the WAQUA simulations (see next section). For eastern storms no typical pattern in wind direction could be derived from the observed storms, so a constant wind direction was assumed.

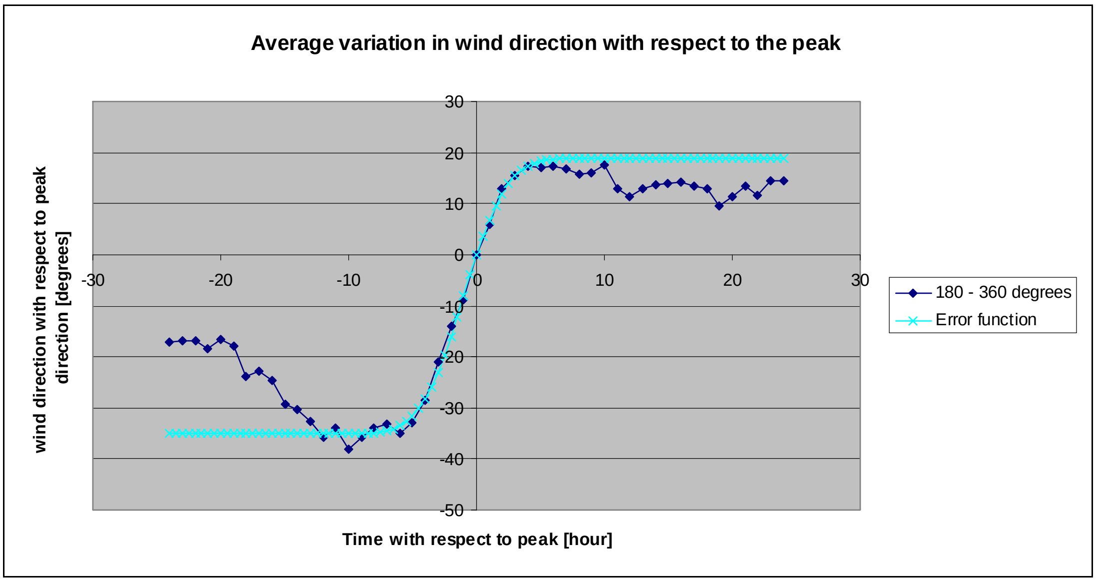


Figure 5.2: *Standard temporal evolution of the wind direction, relative to the direction of the peak wind speed, schematized by an error function.*

5.3 WAQUA and SWAN simulations

For several basic combinations of wind speed and wind direction temporarily varying wind fields are defined with the standard storm profile, see previous section. The WAQUA and SWAN simulations were set up such that they efficiently sample from a wide range of water levels under different conditions. This was done by applying a fixed surge of 2 or 4 m at the offshore boundaries, an average tide and three different tide phase shifts of 0, 4 and 8 hours. These phase shifts will produce different results at nearshore locations, because tide interacts with the changing wind. Especially for the less extreme wind speeds the relative importance of the tide becomes larger.

The following values for the different variables were used:

8 peak wind directions:	0, 90, 180, 210, 240, 270, 300 and 330˚N [2]
3 peak wind speeds:	20, 30 and 40 m/s
3 tidal phases:	0, 4, 8 hours between peak wind and max. water level
2 offshore surge levels:	2, 4 m + NAP

1. *A recent update of Deltares (2009b) advises different wind profiles for three western sectors (180-240, 240-300, 300-360°N). This will be used for future testing.*
2. *Note that eastern wind directions are sparse, while western wind directions are sampled more intensely, for obvious reasons.*

Thus, for 144 combinations of wind speed, wind direction, tidal phases and offshore surge level the WAQUA model provides time series of water level and current during the schematized storm.

Five time instants were selected from each WAQUA time series to provide input for a SWAN simulation. Each time instant represents a possible realization of a water level, wind and wave condition. The five time instants are chosen at 1 hour intervals (in order to reduce the number of data points) starting 2 hours before the peak of the storm up to 2 hours after the peak. The SWAN simulations were executed as 144 semi-stationary SWAN simulations covering 5 hours each, producing output at the five selected time instants.

The current and water level fields from WAQUA were used as input to each stationary SWAN simulation. The wind fields in the SWAN simulations were taken spatially uniform, as in the WAQUA simulations. The WAQUA and SWAN simulations thus produce samples of wind and local water levels with corresponding nearshore wave conditions at each location. These are stored in a temporary database: a list of samples of nearshore wave conditions for various values of local water level, wind speed and wind direction.

5.4 Interpolation to a regular grid

The water levels and wind speeds in the temporary database differ from the regularly spaced values that Hydra-K needs as input. In method 1, the required water levels for Hydra-K are 1, 2, 3, 4, 5, 6, 7 m + NAP and the wind speeds are 15, 20, 25, 30, 35 and 40 m/s. Method 2 produces a temporary database of SWAN results for deviant values of water level and wind (e.g. 1.5 or 1.6 m + NAP and 16 or 18 m/s). The nearshore wave conditions at the regular values required by Hydra-K are determined by interpolation between the values in the temporary database. From several available techniques, the best results have been obtained by fitting a higher order polynomial to the data in the temporary database and calculating the values at the regular grid points for the Hydra-K database (the Matlab script can be found in Appendix B). This is done for each location separately. The procedure is visualized for 1D in Figure 5.3.

Using the polynomials, we can also recalculate the H_{m0} and $T_{m-1,0}$ at the original values of the water level, wind speed and wind direction in the temporary database. The difference between the reconstructed values from polynomial and the original values in the temporary database gives an impression of the goodness of fit, or the scatter of the data around the 3D hypersurface. It was found that the RMSE of this difference is usually less than 10% for a particular location in the Wadden Sea and the largest deviation is less than 30%. Small deviations are found for shallow water locations, because depth-induced wave breaking dominates all other process. The largest deviations occur for locations where currents play an important role. Varying conditions (incoming or outgoing tide) can have a significant effect on the wave height and period. The polynomial fit can probably be improved by including knowledge of the theoretical relationships between wind and waves, but the current version is considered sufficiently accurate to serve for these test purposes. In most cases, a fourth order polynomial was found to give the best results. If wave heights were found that are more than 20% larger than the highest wave height in the temporary database, then the order of the polynomial was reduced to 3rd and if necessary to 2nd.

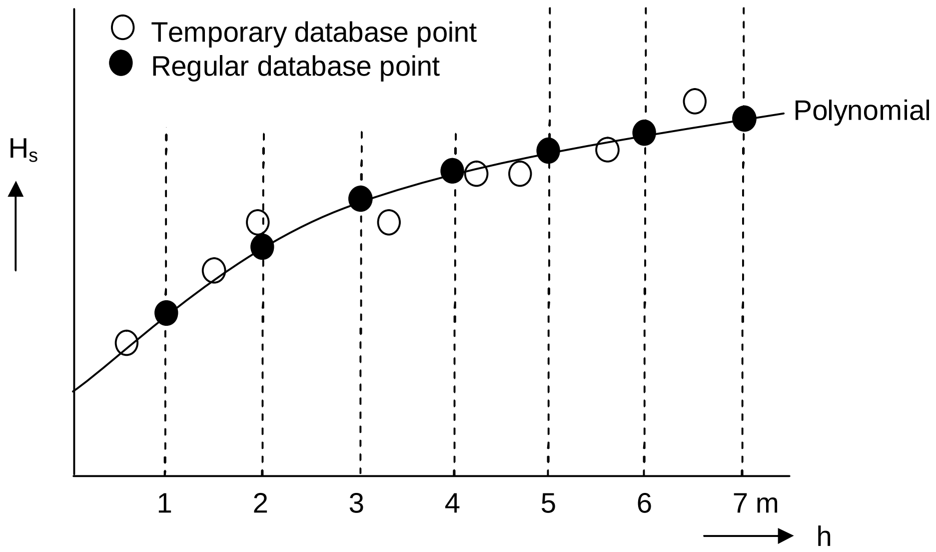


Figure 5.3 *Schematic 1D representation of the translation of the temporary database SWAN data to the regular Hydra-K database values. In reality the translation is 3D, with wind speed and wind direction being the additional dimensions.*

It is difficult to visualise the 3D polynomials, because of their high dimensionality. Figure 5.4 and Figure 5.5 give an impression of the spatial distribution of H_{m0} as a function of water level, wind direction and wind speed.

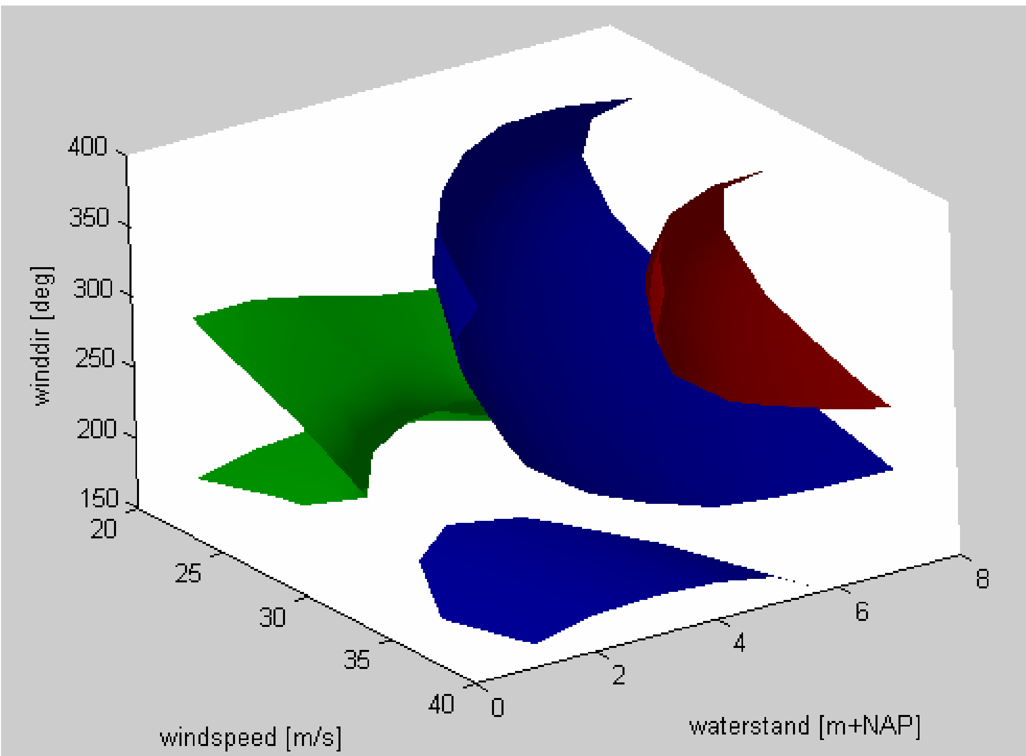


Figure 5.4 *Isosurfaces at 1 m (green), 2 m (blue) and 3 m (red) of the 3D polynomial of the significant wave height for a location in the Wadden Sea. The highest waves are found at the highest water level and wind speed and for wind directions around 350˚.*

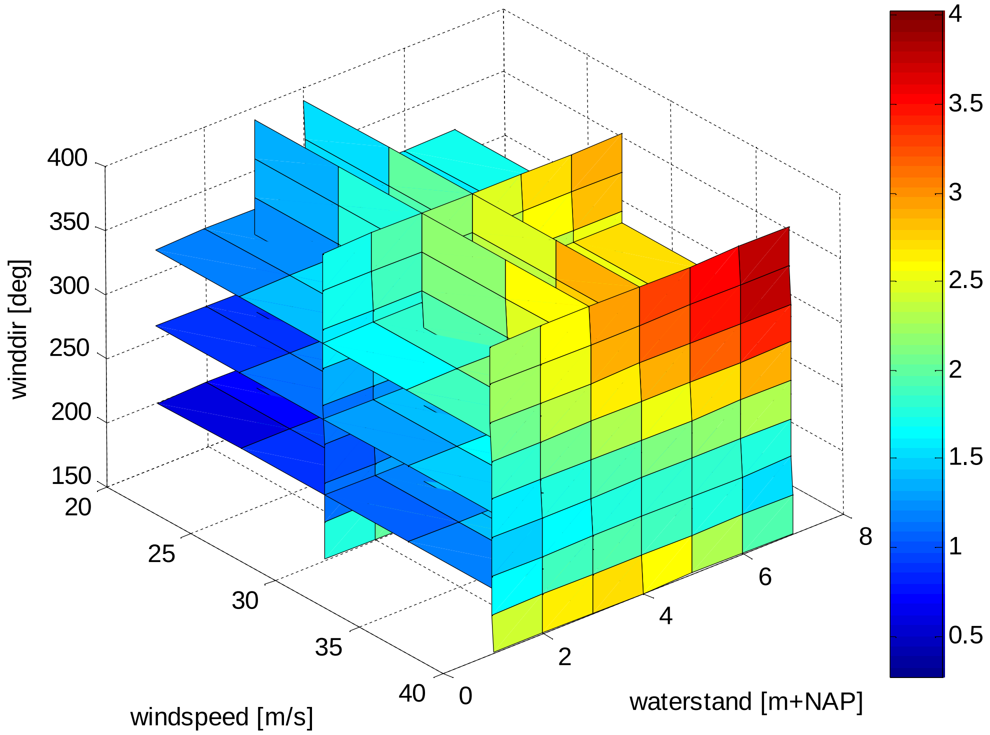


Figure 5.5 Slices through the 3D polynomial of the significant wave height.

6 Results of test calculations

6.1 Outline

In Sections 3, 4 and 5 the two calculation methods 1 and 2 have been described. In this section the resulting normative wave conditions are presented, which are obtained from Hydra-K for the two methods. Because the SWAN grid of the Wadden model contains in the order of 2.5 million active grid points, indicative computations are performed on the coarser Kuststrook-Fijn grid (see Figure 6.1). The grid resolution near the coast is relatively coarse: around 100 m in cross-shore direction. This will have an effect on the nearshore wave conditions. Consequently, the computational results should be used for qualitative comparison only.

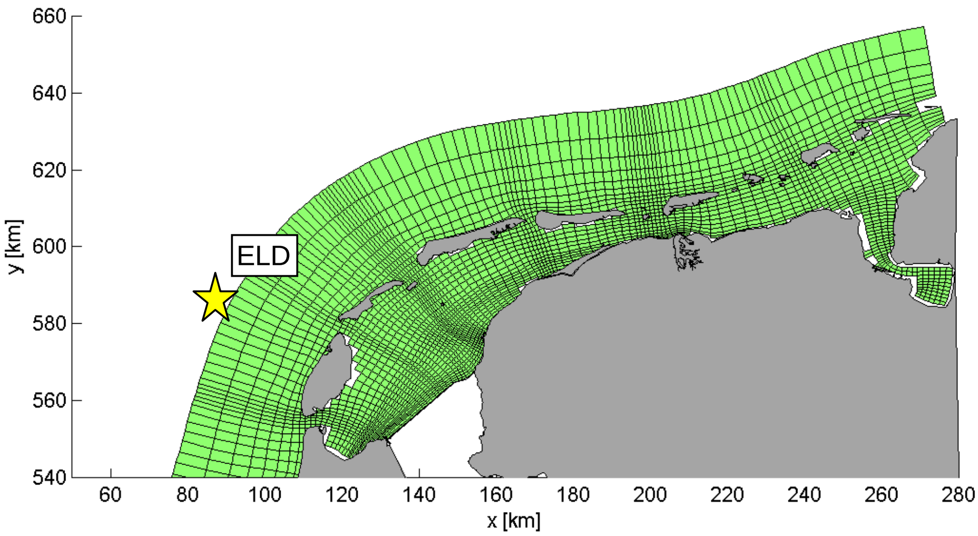


Figure 6.1 Kuststrook-Fijn grid (every 5th grid line is shown). The grid extends further south along the coastline, but this is of less interest to the current study.

6.2 SWAN computations for method 1 and 2

The SWAN computations have been carried out in semi-stationary mode with SWAN version 40.72AB. The settings are similar for both methods:

GEN3 WESTH
QUAD
TRIAD TRFAC=0.05 CUTFR=2.5
BREAKING BIPhase 0.90 2.5 -1.3963
FRICTION JONSWAP CFJON=0.067

Apart from the formulations suggested by Van der Westhuysen et al. (2007) for wind generation and whitecapping and the bi-phase model for depth-induced breaking, developed by Van der Westhuysen (2009), the default settings are applied.

The accuracy of the nearshore wave conditions is already limited by the coarseness of the grid. Therefore, in order to save computing time, the numerical convergence criterion is more relaxed than what is usually applied in hindcast studies, without further loss of accuracy. The criterion is set equal to the default setting:

NUM STOPC 0.00 0.02 0.005 98 STAT MXITST=50

The offshore boundary conditions are imposed as parameterized spectra. These spectra are assumed to have a Jonswap shape and are defined in terms of significant wave height and peak period. The values for these wave conditions are based on extreme value analyses and have been determined by HKV (2005) for offshore buoy locations along the Dutch coast. The statistics of the wave parameters at ELD (indicated in Figure 6.1) are used. The resulting Jonswap spectra are applied along the entire offshore boundary. The wave direction is set equal to the wind direction. The directional spreading is set to 25 degrees on either side of the wind direction. The eastern grid boundary in Germany and the southern boundary in Belgium have open boundary conditions.

For the testing of method 1 (see Chapter 4), a large number of different wind speeds, directions and water levels were considered:

- 15 wind directions: between 30 and 360˚ (30˚ intervals) and 285, 315, 345˚.
- 6 wind speeds: 15, 20, 25, 30, 35 and 40 m/s
- 7 water levels: 1, 2, 3, 4, 5, 6 and 7 m + NAP

leading to 630 combinations. The 90 wind fields and 7 water level fields are uniform in space and constant in time. Currents are not included.

For the testing of method 2, a total of 24 spatially uniform, but temporarily varying wind fields have been used. These 24 wind fields encompass all combinations of the following wind directions and wind speeds:

- 8 peak wind directions: 0, 90, 180, 210, 240, 270, 300 and 330˚
- 3 peak wind speeds: 20, 30 and 40 m/s

These 24 wind fields were combined with three tidal phases (0, 4, 8 hours) and two offshore surge levels (2 and 4 m + NAP) to provide a total of 144 different forcings to the WAQUA model. From these WAQUA simulations, a snapshot of the water level and current field is taken at five time instants around the peak of the storm. At these five instants a stationary SWAN computation is performed.

In each of the 720 SWAN computations a uniform wind field, corresponding to the wind at the time instant during the WAQUA simulations is imposed. The non-uniform water level and current fields from WAQUA are also used as input in the SWAN calculation. The offshore boundary conditions are based on a Jonswap spectrum, similar to the tests for method 1.

6.3 Hydra-K results

The results of both test series (method 1 and 2) were stored in two databases, which were used as input for Hydra-K to determine the normative wave conditions along the sea defences in the Wadden Sea. For both methods, the normative significant wave height H_{m0}, mean wave period $T_{m-1,0}$ at the assessment water level ('toetspeil') and wind direction θ_w are shown in Figure 6.2 as a function of the distance from Den Helder along the coast line. For some locations, Hydra-K does not generate output, because these locations lie outside the coarse SWAN grid and there is no SWAN output. These locations form 'gaps' in the graphs of Figure 6.2.

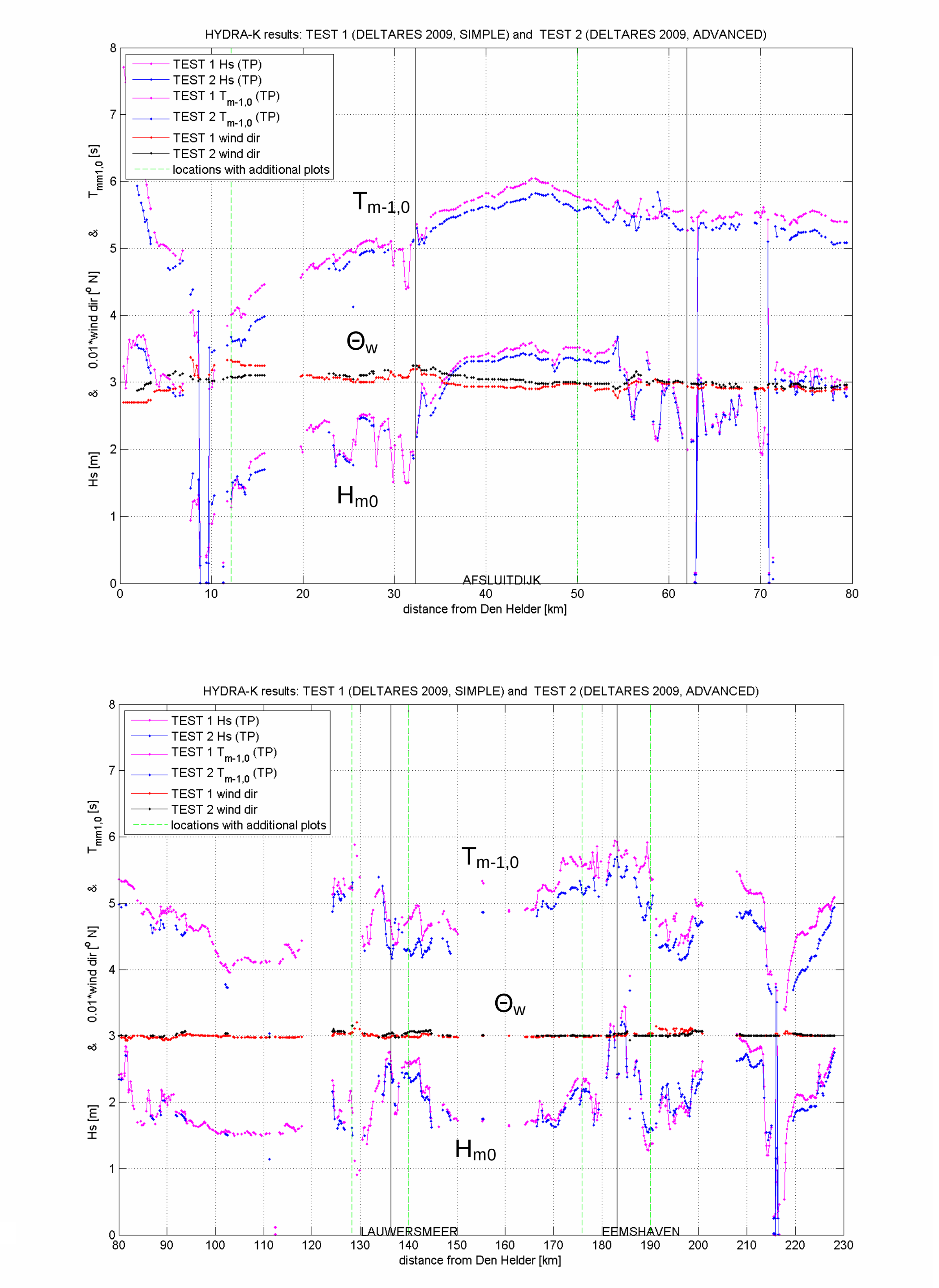


Figure 6.2 Hydra-K results for H_{m0}, $T_{m-1,0}$ and wind direction for test 1 and 2 (resp. method 1 and 2) in Western Wadden Sea (upper plot) and Eastern Wadden Sea (lower plot).

The normative wind direction varies slightly around 300 °N, due to the fact that the probability of occurrence of wind directions in the western sector is high compared to the other sectors. Den Helder is an exception to this, with a difference of up to 30° between the two tests.

In general, the wave periods form method 1 are 5-10% longer than those obtained from the more advanced method 2. The difference for the significant wave height is smaller. In shallow areas the differences are negligible. At deeper water locations, method 1 produces 5% higher values than method 2. For some exceptional locations method 2 produces higher wave heights, for example at ~9 km from Den Helder.

In Section 6.3, the results of the SWAN computations are further investigated to explain why method 2 results in smaller wave parameters. The focus is on 6 locations, which are indicated in Figure 6.2 by vertical green, dashed lines.

6.4 SWAN computations for method 1 and 2

At six of the more than 1300 output locations the SWAN results are considered in more detail. These locations are indicated in Figure 6.3:

- Loc. 58 (km 12): Balgzand, very shallow area;
- Loc. 371 (km 50): Afsluitdijk, shallow foreshore, 2 km from channel;
- Loc. 910 (km 128): Wierumerwad, with shallow foreshore;
- Loc. 990 (km 140): In Zoutkamperlaag channel;
- Loc. 1153 (km 176): Uithuizerwad, at shallow foreshore;
- Loc. 1211 (km 190): Bocht van Watum, close to channel

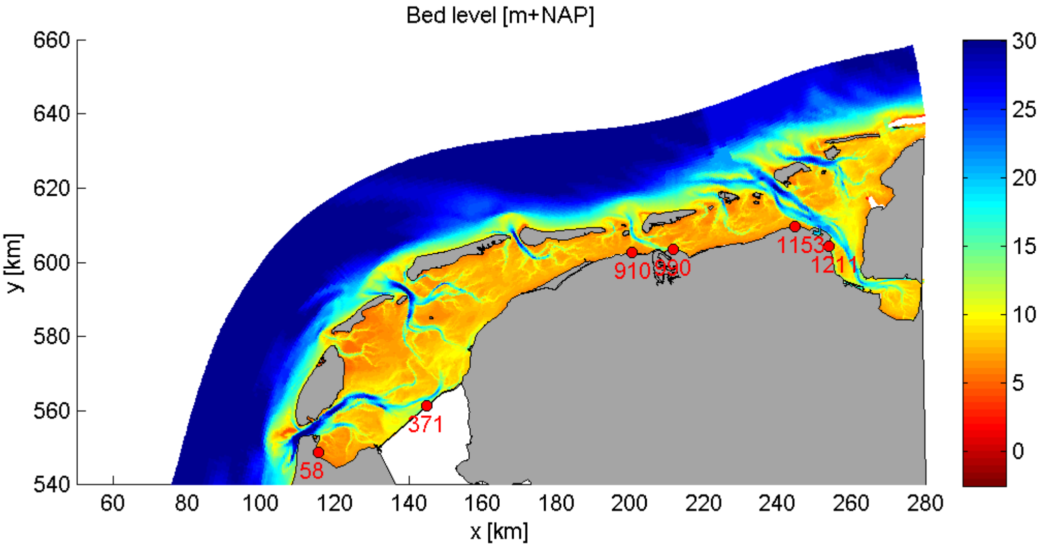


Figure 6.3 Bathymetry of Wadden Sea with the selected output locations

At these six locations, the significant wave heights and mean wave periods from SWAN computations using method 2 are plotted as a function of the local water level (see Figures

C.1 – C.6). For peak wind direction 300°N and peak wind speed 40 m/s, the results of method 2 are highlighted and can be compared to the results for method 1. The fitted polynomial for this peak wind speed and wind direction has also been plotted.

Figures D.1 – D.6 display the water levels and current fields for the peak wind speed from 300°N and 40 m/s at three time instants during the storm. The water is pushed into the Wadden Sea through the western inlets and at some time instants later the water is pushed out of the Wadden Sea through the Amelander Zeegat. The current velocities are largest at the shallow tidal flats. Because the currents in the Wadden Sea are wind-dominated, the currents are mainly in the same direction as the wave direction. The WAQUA model produces unrealistically high current velocities in the eastern part of the computational domain, offshore of the Wadden Sea. This is due to an improper water level boundary condition at the eastern boundary. Since the extreme values do not affect the wave conditions in the domain of interest, we have not adapted the WAQUA model set up.

The locations 910 and 1153 are on the shallow flats of the Wierumerwad and Uithuizerwad respectively. The small amount of scatter in H_{m0} and $T_{m-1,0}$ for the west to north wind sector indicates a strong depth limitation of the wave conditions (see Figures C.3 and C.5). The waves are at maximum height for this depth and the effect of turning winds and currents on the significant wave height is negligible. In contrast, the current does affect the wave period. Since the currents are mainly following the waves, the relative forcing decreases and the wave periods decrease, when the currents are included in method 2. At location 910 the decrease is 15%, at location 1153 10%. The interpolated surface projections are good fits through the results.

The total water depth at location 371 in front of the Afsluitdijk is larger than at the two former locations. Although there is still a clear depth-dependency on the significant wave height (see Figures C.2), the depth limitation is less than at the former locations. The waves are not fully saturated and effect of currents on wave height and of water level gradient on both parameters is not negligible. This causes some scatter in the results. For the same water level at the location near the dike in method 1 and 2, the water level gradient in method 2 leads to water levels upwind that are smaller than the nearshore water level. Consequently, the significant wave height at upwind locations will be smaller, which affects the wave height at the dike location. The difference between waves heights from method 1 and method 2 varies between 0 and 10%. The wave period obtained from method 2 is generally 10% smaller, due to a combination of the water level gradient and a following current. The same reasoning holds for location 990, at the end of a tidal channel. The wave height and wave period obtained from method 2 are approximately 10-15% smaller than those obtained from method 1 (see Figure C.4). For both locations the fits through the SWAN results are good.

The locations 58 and 1211 are both sheltered from western winds. Although 300°N appears to be the normative wind direction (see Figure 6.2), the most severe wave conditions are not obtained for this direction. Higher values for wave height and wave period are obtained from northern winds. At location 58 the wave conditions obtained from method 2 are scattered around those from method 1. Note that the current velocity at Balgzand is negligible. At location 1211 in the Bocht van Watum the presence of the channel leads to lower wave heights and wave periods. Also, the strong water level gradient causes the wave height to be smaller for method 2 than for method 1 (see Figure C.6).

The projection of the interpolation surface is a poor fit through the SWAN results, especially for the significant wave height. In Figures C.7 – C.10, only the results of the computations at location 1211 with peak directions 270, 300, 330 and 360 °N have been considered. The variation from one direction to the next is significant, due to the fact that for westerly directions location 1211 is sheltered by the mainland and the highest wave heights are obtained for northerly directions. The polynomial fit cannot follow this strong variation. Consequently, the low wave heights for 270 °N are overestimated by 100% by the fit and those for 360 °N are underestimated by 30%. For short-fetch (270 °N) and slanting-fetch situations, the coarse grid strongly influences the results. For finer grids, the transition of wave conditions from short to slanting fetch may be less abrupt. Therefore, the poor fit is accepted for the moment.

The number of combinations of variables wind speed, wind direction and surge is considered adequate. All relevant water levels occur in the simulations. Some combinations of water level and wind do not occur because they are unrealistic (very high water level and strong eastern wind). These will be given a value by the polynomial fit function. The fitted values will be inaccurate, because they are outside the sampled domain. However, since the combinations are unrealistic, they will not be used by Hydra-K.

6.5 Conclusion

Method 2 provides realistic estimates of wave height and wave period at most of the output locations. At depth-limited locations, the significant wave heights obtained from method 1 and 2 are very similar. Method 2 produces roughly 10% smaller wave heights at locations where wave conditions are not fully saturated. At some exceptional locations method 2 provides higher wave heights than method 1. These cases are rare and are probably due to a poor polynomial fit through the data entries. The polynomial fitting procedure should therefore be improved.

The SWAN wave periods from method 2 are 10% smaller compared to method 1. This is due to the fact that Hydra-K spreads the results from various wind directions.

7 Conclusions and recommendations

This report has discussed a number of different alternatives for calculating Hydraulic Boundary Conditions in the Wadden Sea:

- Method 1 uses the same setup as for earlier HBC calculations, using Hydra-K and a database of SWAN results. The SWAN calculations are performed assuming a uniform water level and neglecting the effect of current on waves . This most basic approach was developed and tested. Although no quantitative conclusions can be drawn due to the simple setup, the test results show that the method is useable and that higher wave heights are found compared to earlier studies, based on older SWAN versions.
- Method 2 uses the same setup of Hydra-K and SWAN, but takes into account the effects of current and spatially varying water levels on waves in a pragmatic way, that does not require modifications to the Hydra-K software. Method 2 was shown to produce realistic HBC.
- Development of method 3 is regarded as a further improvement to method 2. However, development of this method will require considerable effort and therefore cannot be used for HBC 2011 calculations.

In general method 2 leads to wave heights that are 0-10% lower and wave periods that are 5-10% smaller than those obtained from method 1. At depth-limited nearshore locations, the difference in wave heights from method 1 and 2 is negligible. At other locations, method 1 produces wave heights that are 5% higher than obtained with method 2. For some exceptional locations method 2 produces higher wave heights. These cases are rare.

It is recommended to use method 2 for the production of HBC 2011 for the Wadden Sea. The differences between method 1 and 2 can be explained from general knowledge of current-wave interactions. Because method 2 takes into account the effects of current and spatially varying water levels, it is believed to produce more realistic HBC. Method 2 can also be applied to the Western Scheldt area, which is similar to the Wadden Sea. Analysis of historical storms at station Vlissingen showed that the same typical storm pattern can be used for this area (see Appendix A).

At some locations, the polynomial fit function that is used in method 2 performs less than optimal, which can lead to errors. Although these cases are rare, it is recommended to investigate how the fit function can be improved.

Finally, it is recommended to start research activities for the development of method 3 for future application (after HBC 2011). In order to be ready by 2016, this research should be started as soon as possible.

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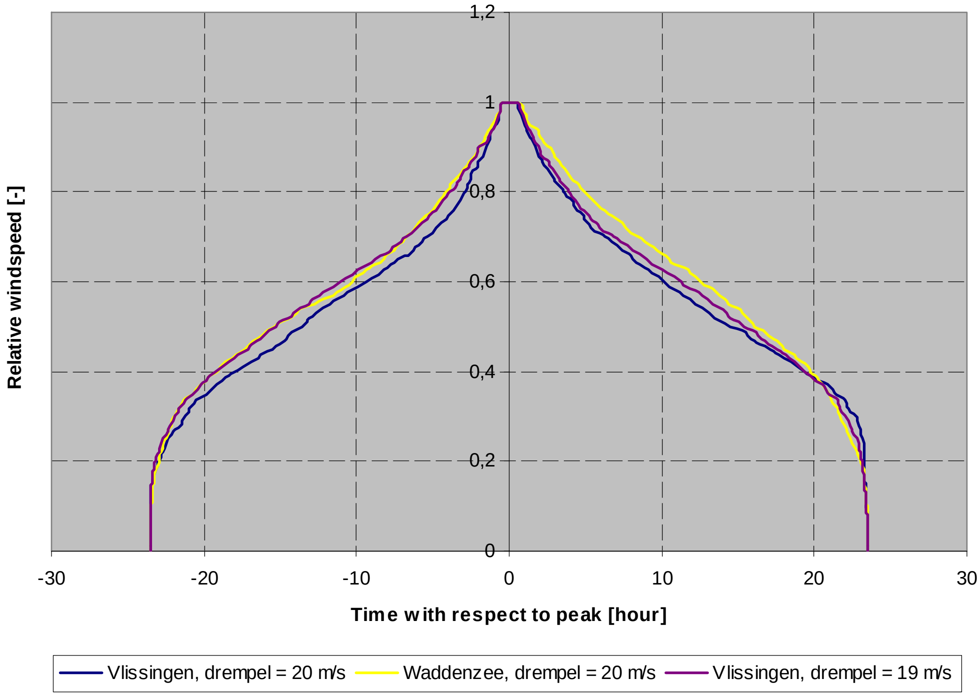
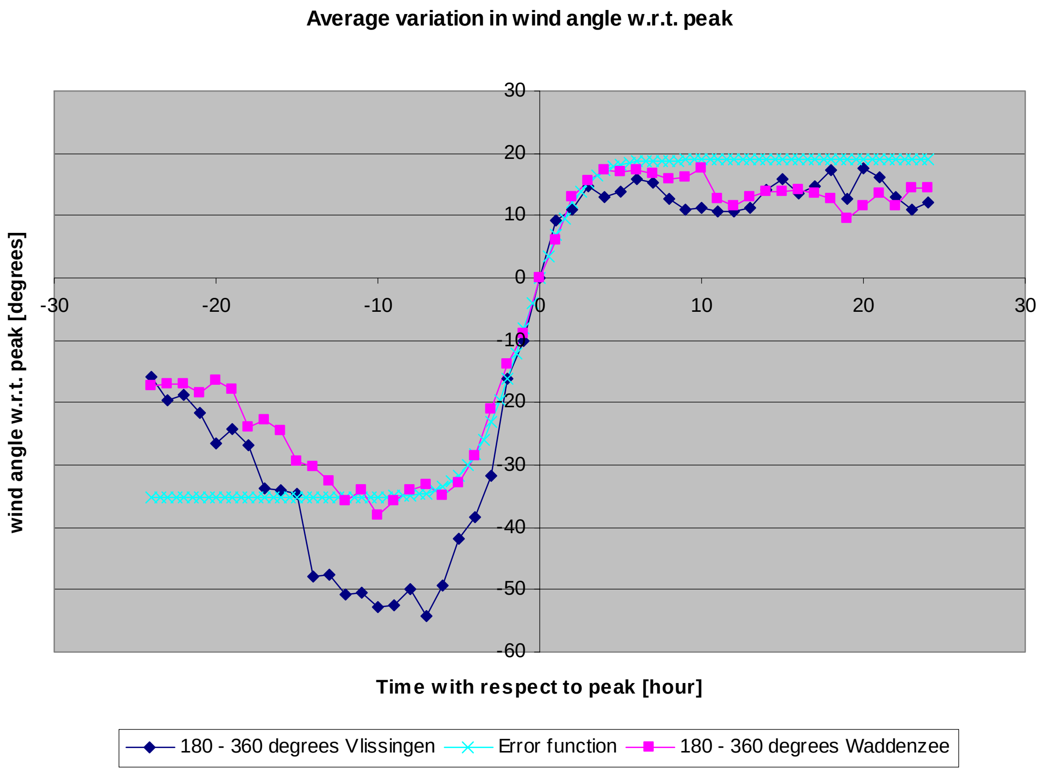
A Storm patterns for Vlissingen





B SWAN2GRID Matlab source

```
% Polyfitn path
addpath 'PolyfitnTools\PolyfitnTools';

% Read SWAN data
loc=500;

load res20
eval(['a=m' num2str(20) '(loc).out;']);

load res30
eval(['b=m' num2str(30) '(loc).out;']);

load res40
eval(['c=m' num2str(40) '(loc).out;']);

% Load values from res20, res30, res40
winddir=[a(:,10); b(:,10); c(:,10)];
windspeed=[a(:,9); b(:,9); c(:,9)];
wl=[a(:,8); b(:,8); c(:,8)];
hs=[a(:,11); b(:,11); c(:,11)];

% Shift zero to 100 deg
f=find(winddir<100); winddir(f)=winddir(f)+360;

% Remove SWAN missing data values hs = -9
f=find(hs<-8);
hs(f)=[];
winddir(f)=[];
windspeed(f)=[];
wl(f)=[];

% Define Hydra-K grid for wl, windspeed, winddir
classWL=[1:1:7];
classWindSpeed=[20:5:40];
classWindDir=[150:30:390];
[gWL,gWS,gWD]=ndgrid(classWL,classWindSpeed,classWindDir);

p = polyfitn([wl windspeed winddir], hs, 4);
hydraKHs = polyvaln(p,[reshape(gWL, prod(size(gWL)), 1), ...
  reshape(gWS, prod(size(gWS)), 1), reshape(gWD, prod(size(gWD)), 1)]);

% use lower order polynomials if extreme wave heigths occur.
if(max(hydraKHs)>1.2*max(hs))
  p = polyfitn([wl windspeed winddir], hs, 3);
  hydraKHs = polyvaln(p,[reshape(gWL, prod(size(gWL)), 1), ...
    reshape(gWS, prod(size(gWS)), 1), reshape(gWD, prod(size(gWD)), 1)]);
```

```
  if(max(hydraKHs)>1.2*max(hs))
    warning 'use 2nd order polynomial ';
    p = polyfitn([wl windspeed winddir], hs, 2);
    hydraKHs = polyvaln(p,[reshape(gWL, prod(size(gWL)), 1), ...
      reshape(gWS, prod(size(gWS)), 1), reshape(gWD, prod(size(gWD)), 1)]);
  else
    warning 'use 3rd order polynomial ';
  end
end

% replace zeros or negative values by minimal wave height 5 cm.
f=find(hydraKHs<0.05); hydraKHs(f)=0.05;

% reconstruct Hs at original points and calculate the difference
hsi = griddata3(gWS,gWL,gWD,hydraKHs,windspeed,wl,winddir);
rel_error = (hsi - hs)./hs;
f=find(isnan(rel_error)); rel_error(f)=[];
std(rel_error)
max(abs(rel_error))
```

C Comparison of Test 1 and Test 2 results

At six locations the results of method 2 are compared with those obtained with method 1. In Figures C.1 – C.6 the significant wave height and the mean wave period are shown as a function of the water level for all Test 2 computations. For the normative wind direction of 300 °N and wind speed of 40 m/s the interpolated result is compared to the result of method 1.

For location (1211), the wave computations with a peak direction in the north-west wind direction (270 - 360 °N) are plotted in Figures C.7 – C.10, respectively. For the four peak wind directions the interpolated results are compared to the result of method 1.

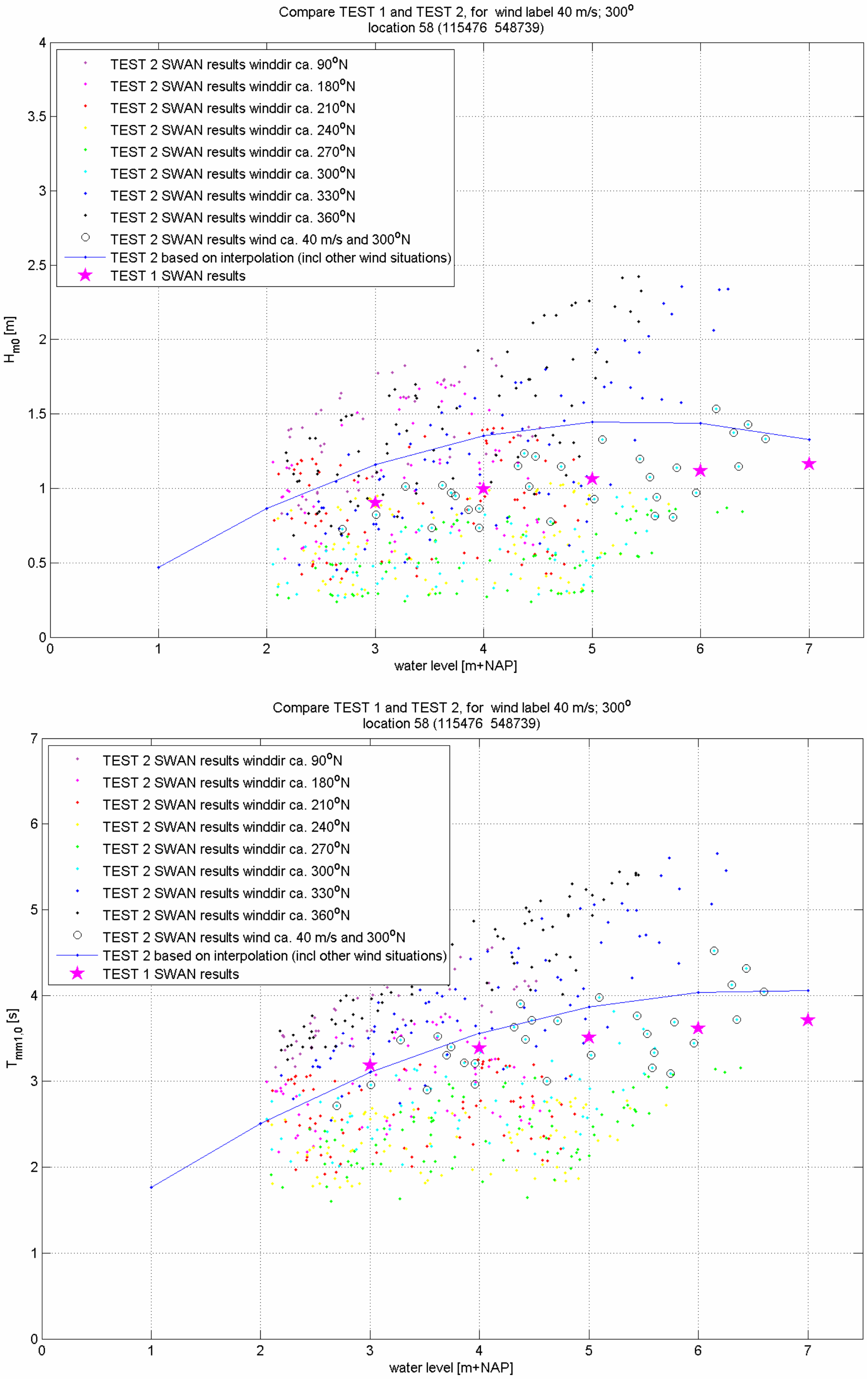
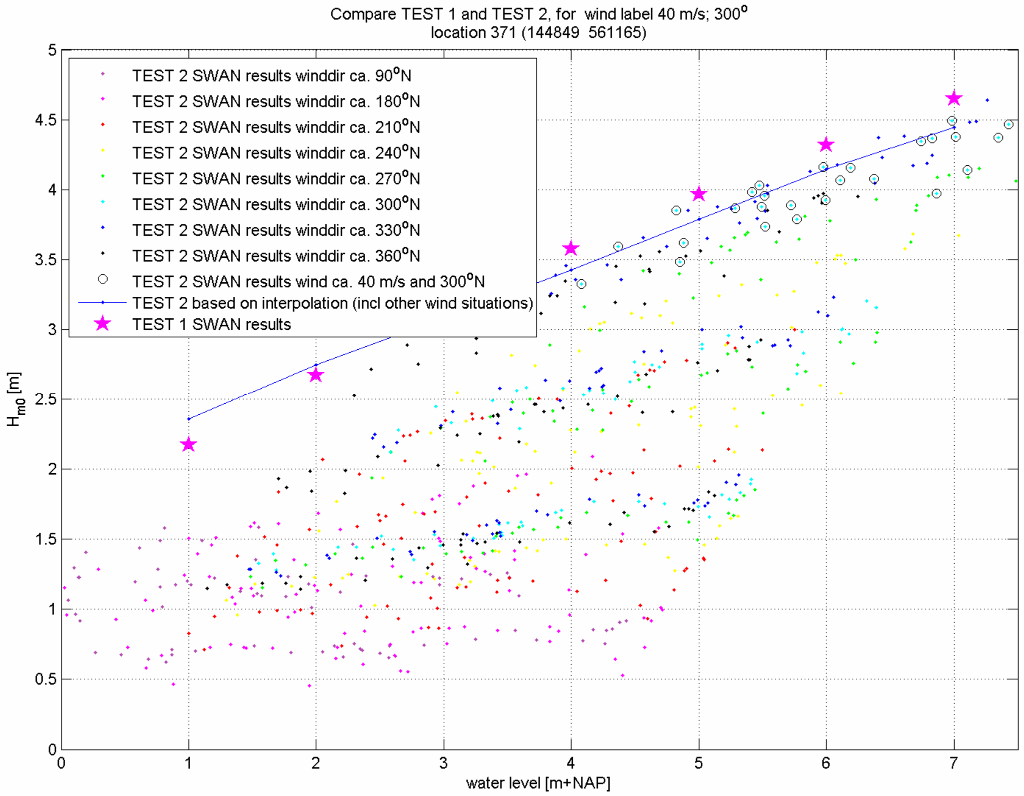


Figure C.1 Significant wave height (upper plot) and mean wave period (lower plot) for all Test 2 computations at location **58**, specified per wind direction. Comparison between Test 1 and Test 2 results for wind of 40 m/s, 300 °N.



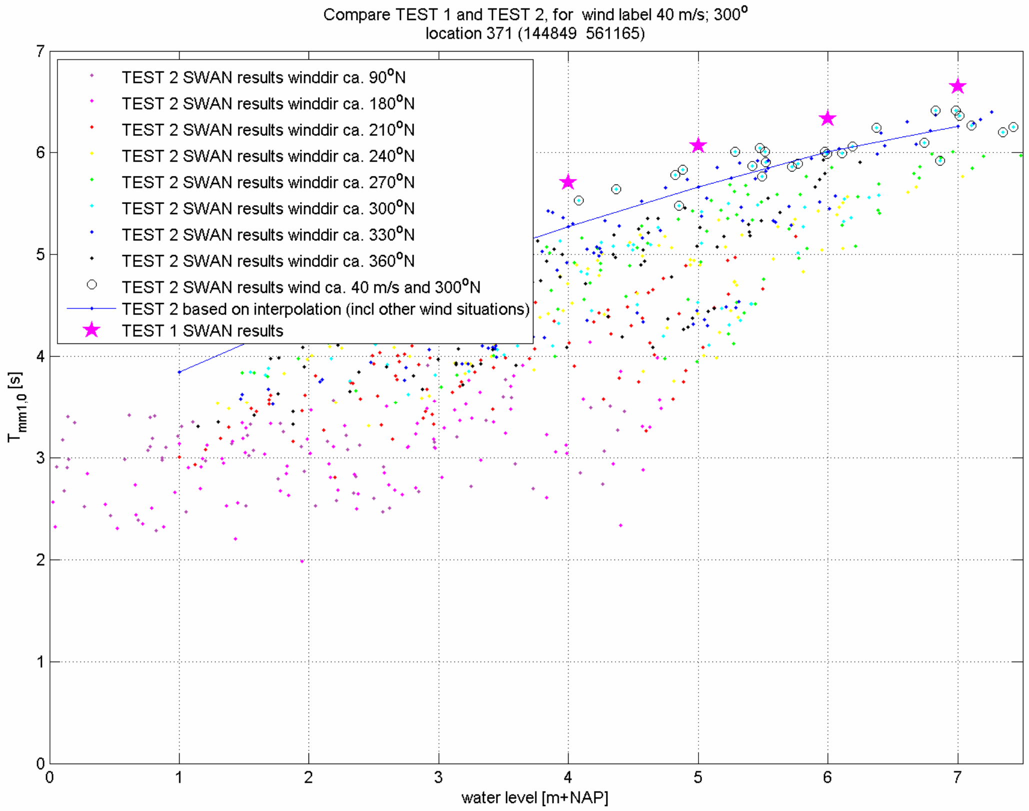


Figure C.2 Significant wave height (upper plot) and mean wave period (lower plot) for all Test 2 computations at location **371**, specified per wind direction. Comparison between Test 1 and Test 2 results for wind of 40 m/s, 300 °N.

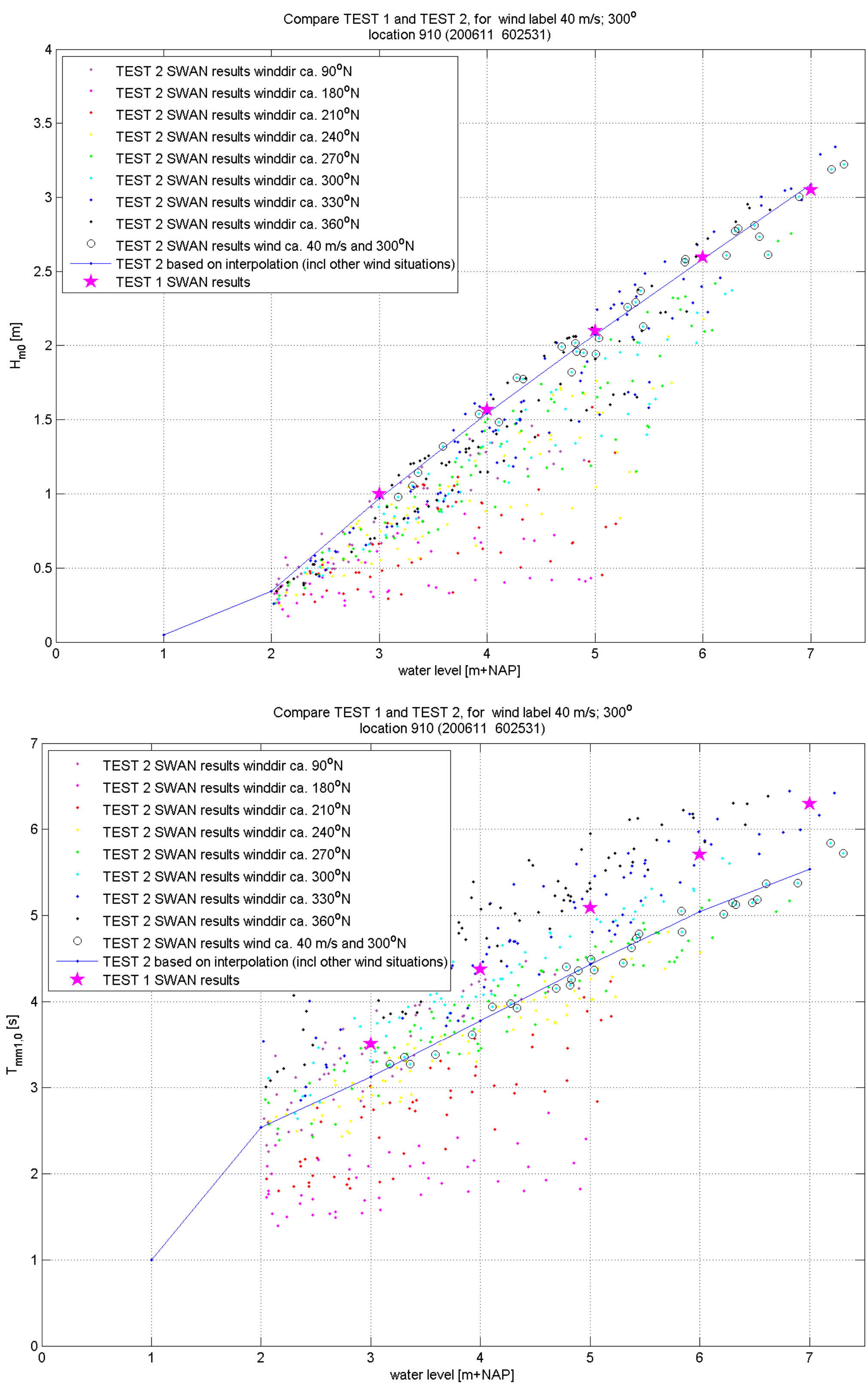


Figure C.3 Significant wave height (upper plot) and mean wave period (lower plot) for all Test 2 computations at location **910**, specified per wind direction. Comparison between Test 1 and Test 2 results for wind of 40 m/s, 300 °N.

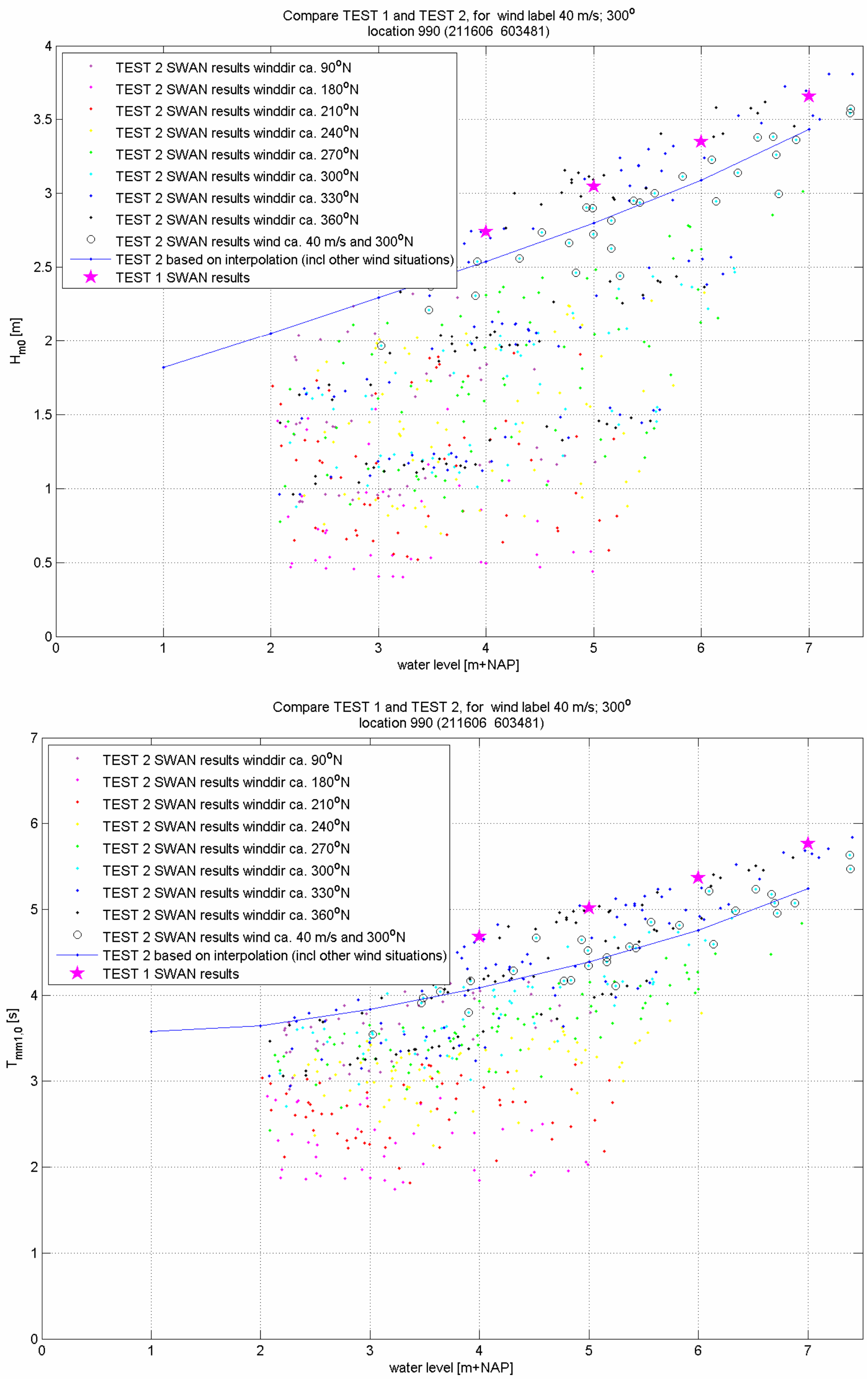
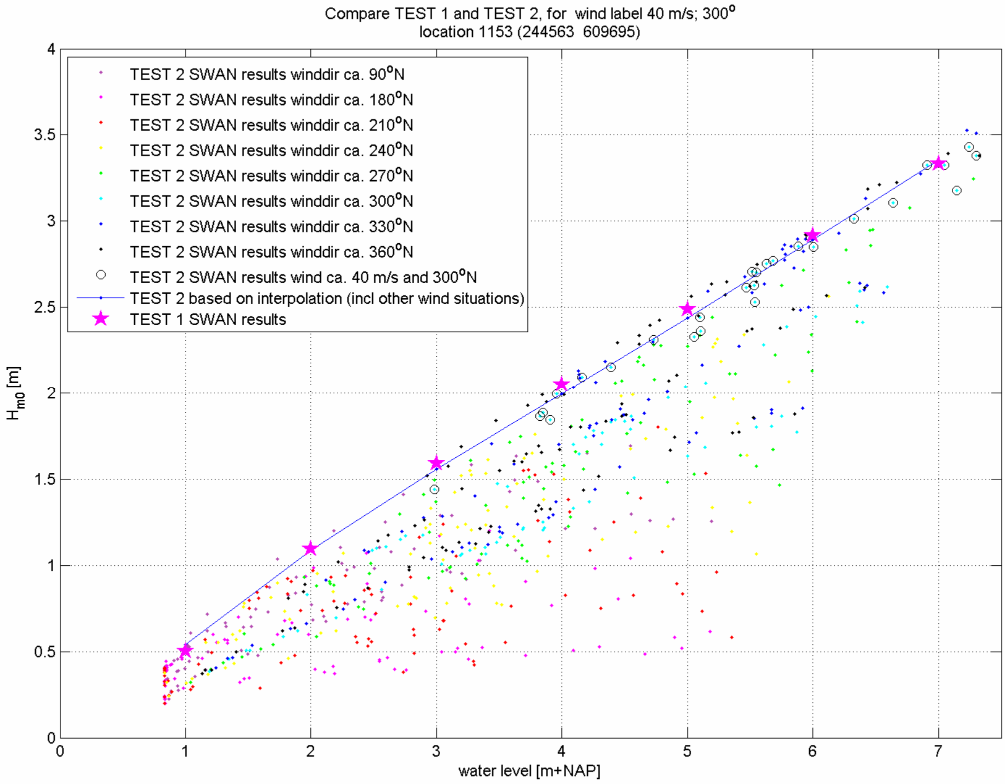


Figure C.4 Significant wave height (upper plot) and mean wave period (lower plot) for all Test 2 computations at location **990**, specified per wind direction. Comparison between Test 1 and Test 2 results for wind of 40 m/s, 300 °N.



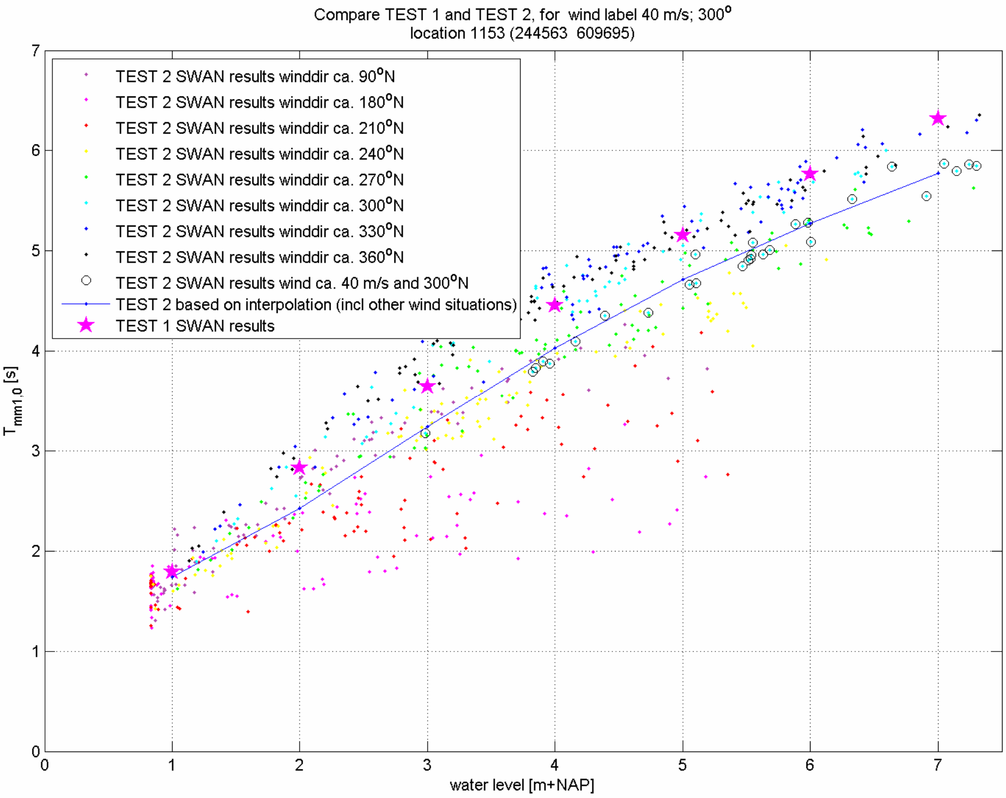
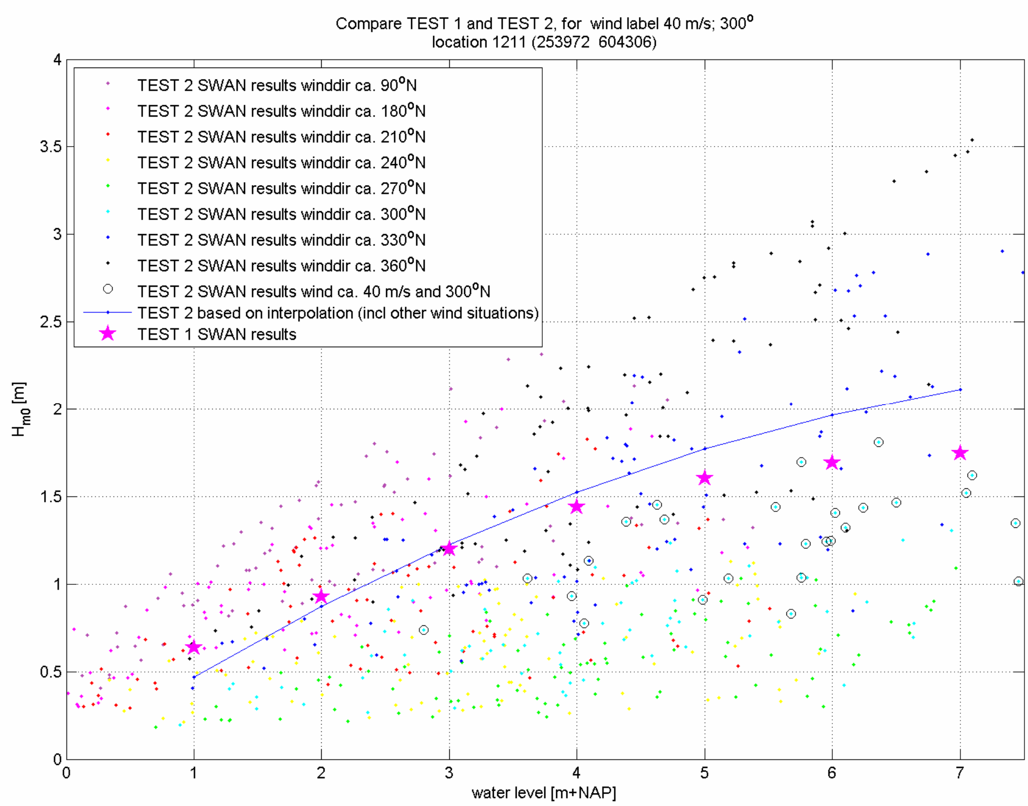


Figure C.5 Significant wave height (upper plot) and mean wave period (lower plot) for all Test 2 computations at location **1153**, specified per wind direction. Comparison between Test 1 and Test 2 results for wind of 40 m/s, 300 °N.



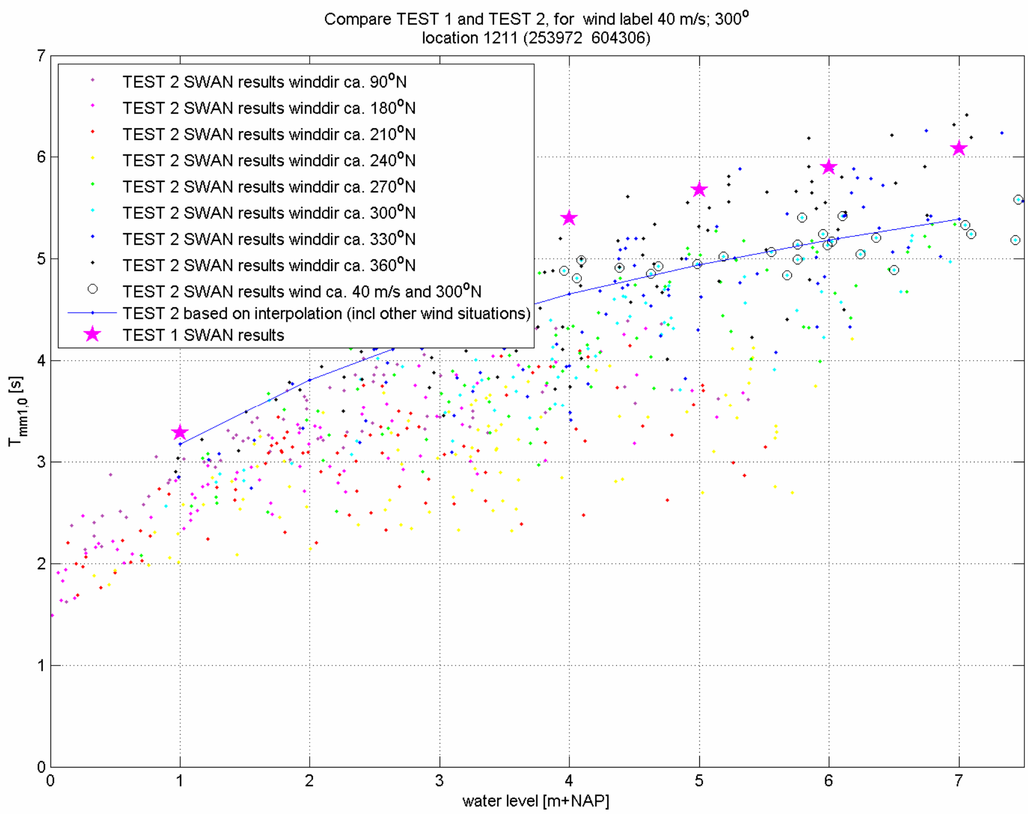


Figure C.6 Significant wave height (upper plot) and mean wave period (lower plot) for all Test 2 computations at location **1211**, specified per wind direction. Comparison between Test 1 and Test 2 results for wind of 40 m/s, 300 °N.

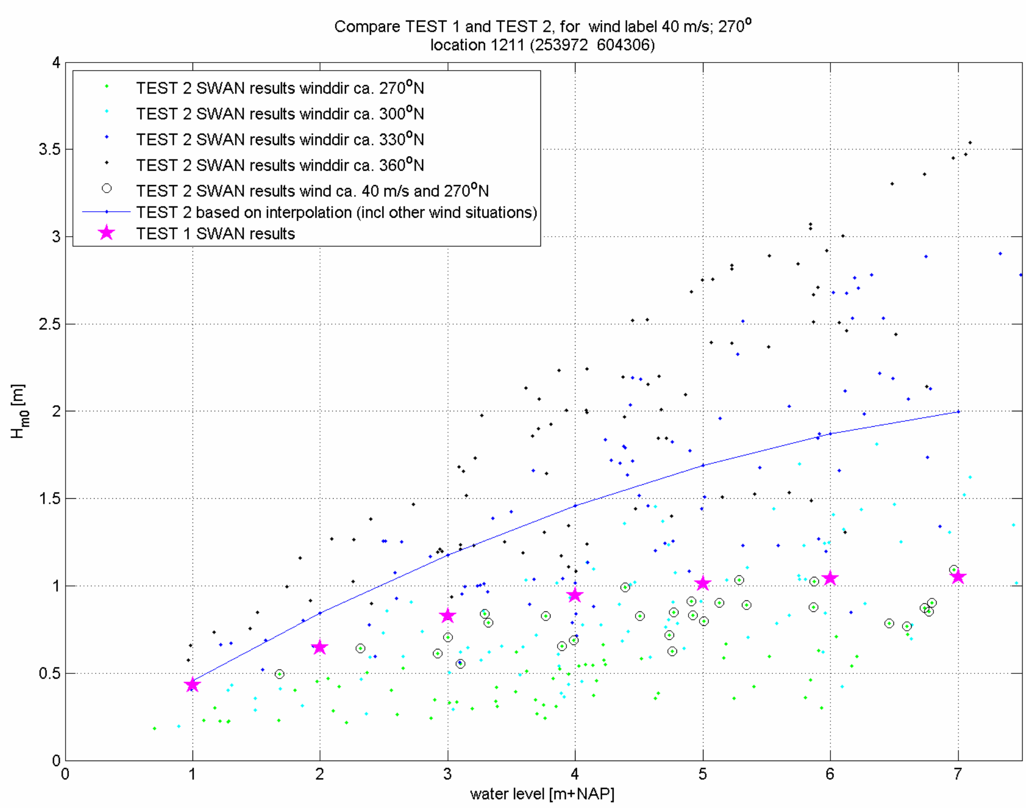


Figure C.7 Significant wave height for all Test 2 computations with peak wind direction in north-west wind sector at location **1211**, specified per wind direction. Comparison between Test 1 and Test 2 results for wind of 40 m/s, 270 °N.

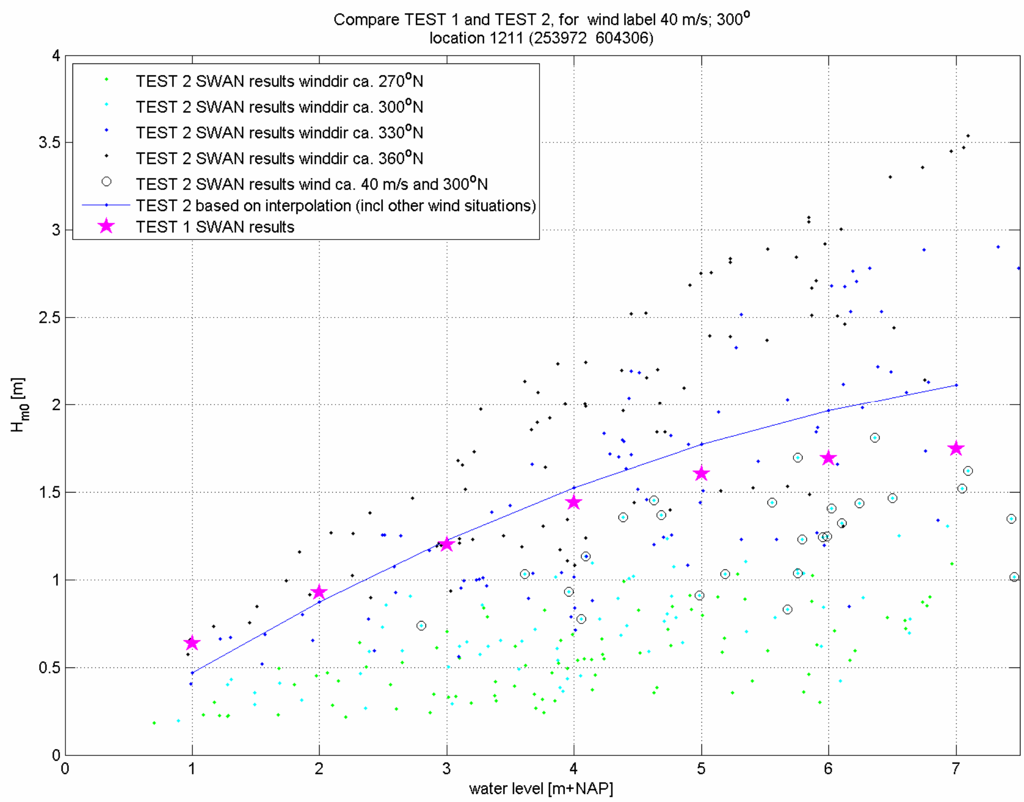


Figure C.8 Significant wave height for all Test 2 computations with peak wind direction in north-west wind sector at location **1211**, specified per wind direction. Comparison between Test 1 and Test 2 results for wind of 40 m/s, 300 °N.

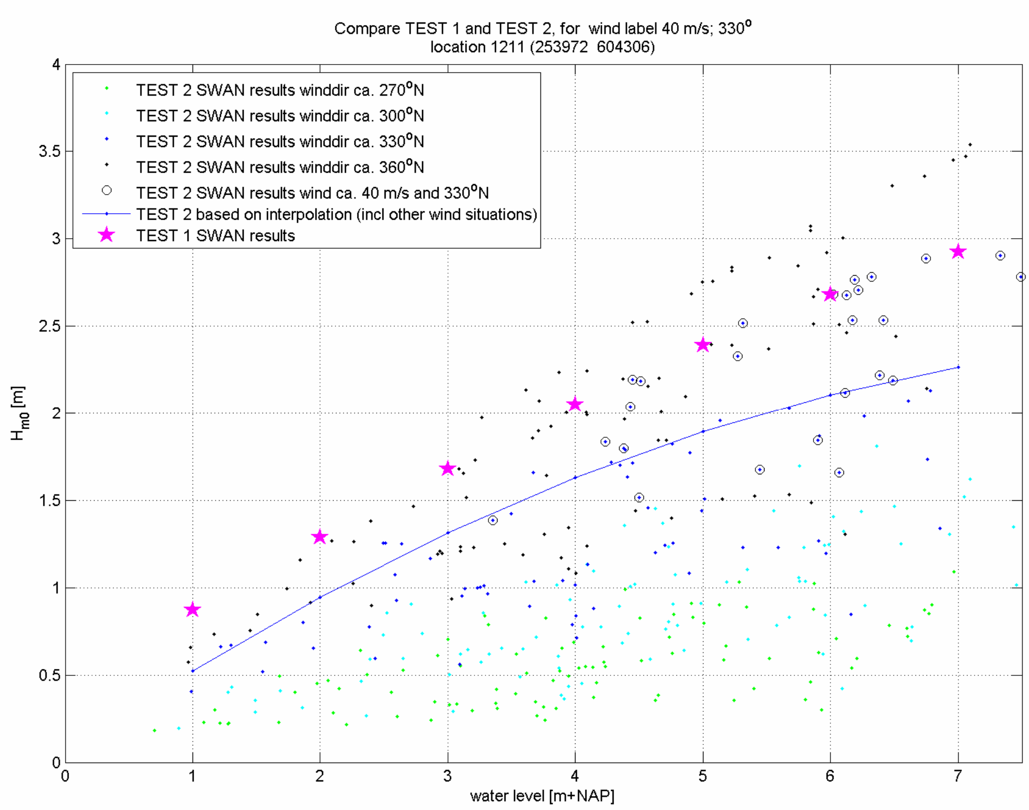


Figure C.9 Significant wave height for all Test 2 computations with peak wind direction in north-west wind sector at location **1211**, specified per wind direction. Comparison between Test 1 and Test 2 results for wind of 40 m/s, 330 °N.

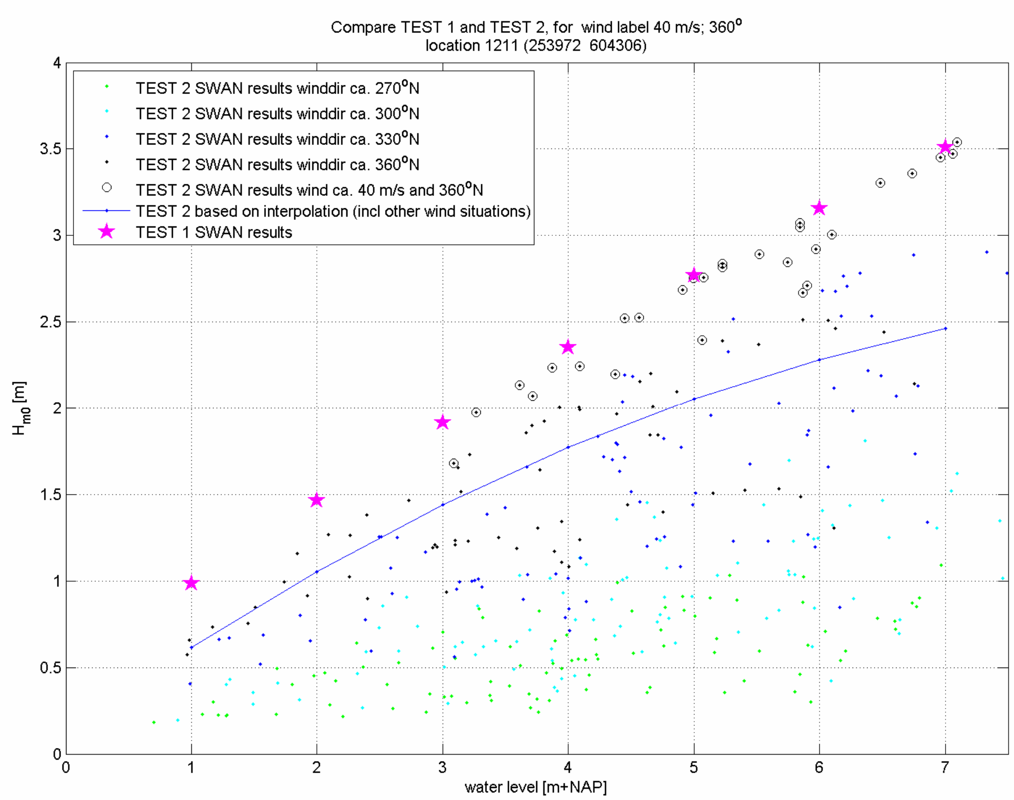


Figure C.10 Significant wave height for all Test 2 computations with peak wind direction in north-west wind sector at location **1211**, specified per wind direction. Comparison between Test 1 and Test 2 results for wind of 40 m/s, 360 °N.

D Water level and current fields for one test case

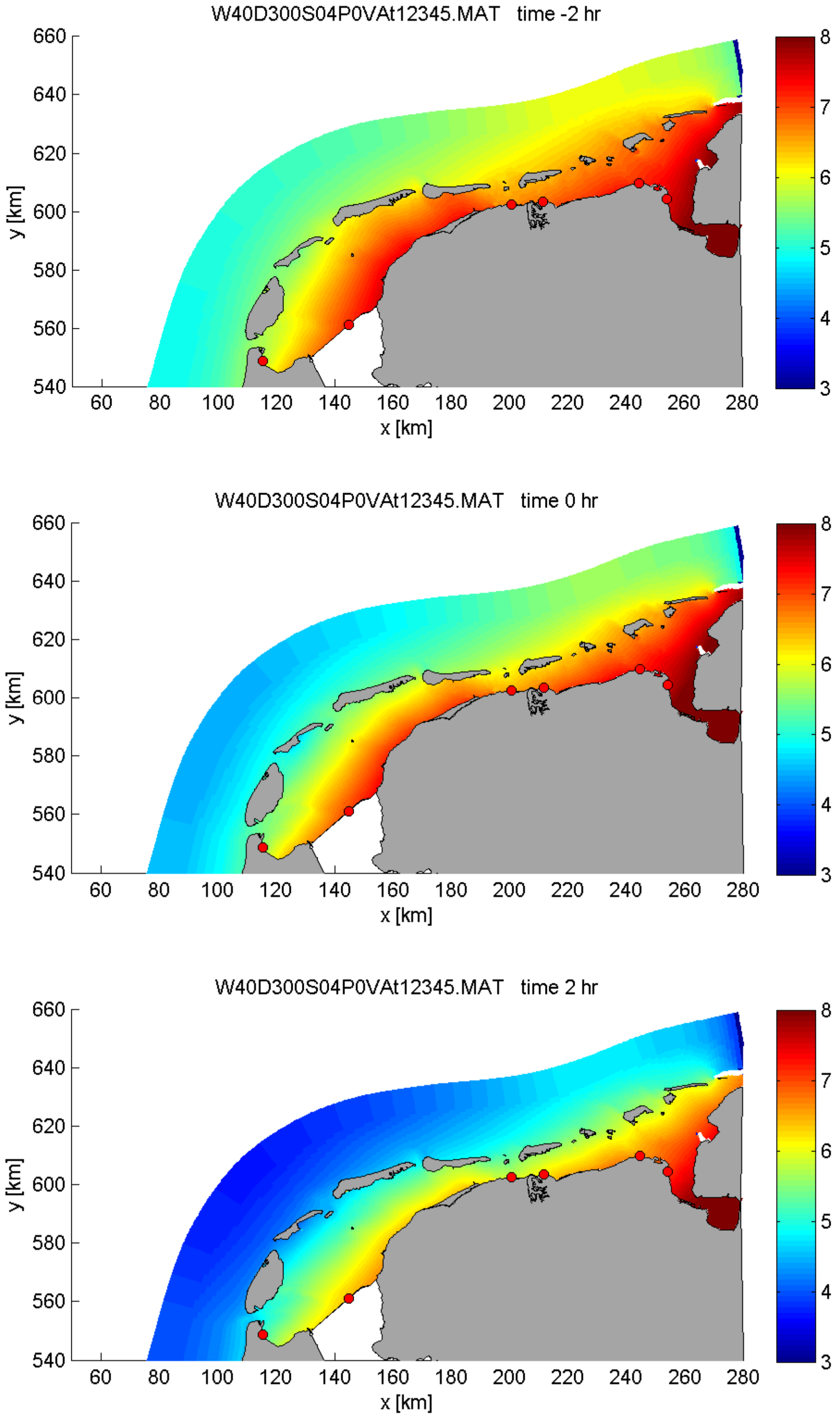


Figure D.1 Water level fields for case with peak wind field of 40 m/s, peak wind direction of 300 °N, offshore surge level of 4 m, and tidal phase difference of 0 hours at 2 hours before the peak, at the peak and 2 hours after the peak of the storm.

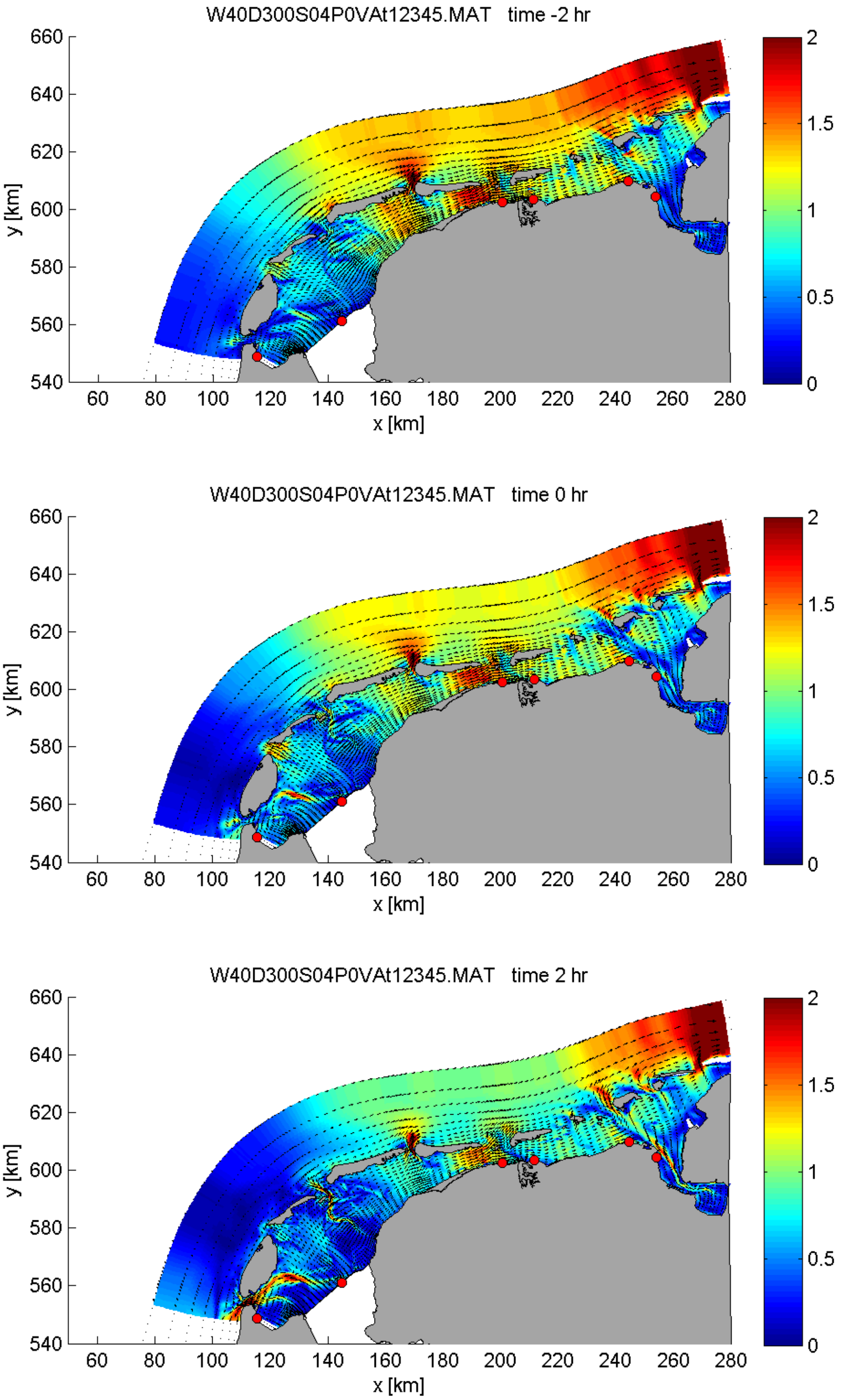


Figure D.2 Current fields for case with peak wind field of 40 m/s, peak wind direction of 300 °N, offshore surge level of 4 m, and tidal phase difference of 0 hours at 2 hours before the peak, at the peak and 2 hours after the peak of the storm.

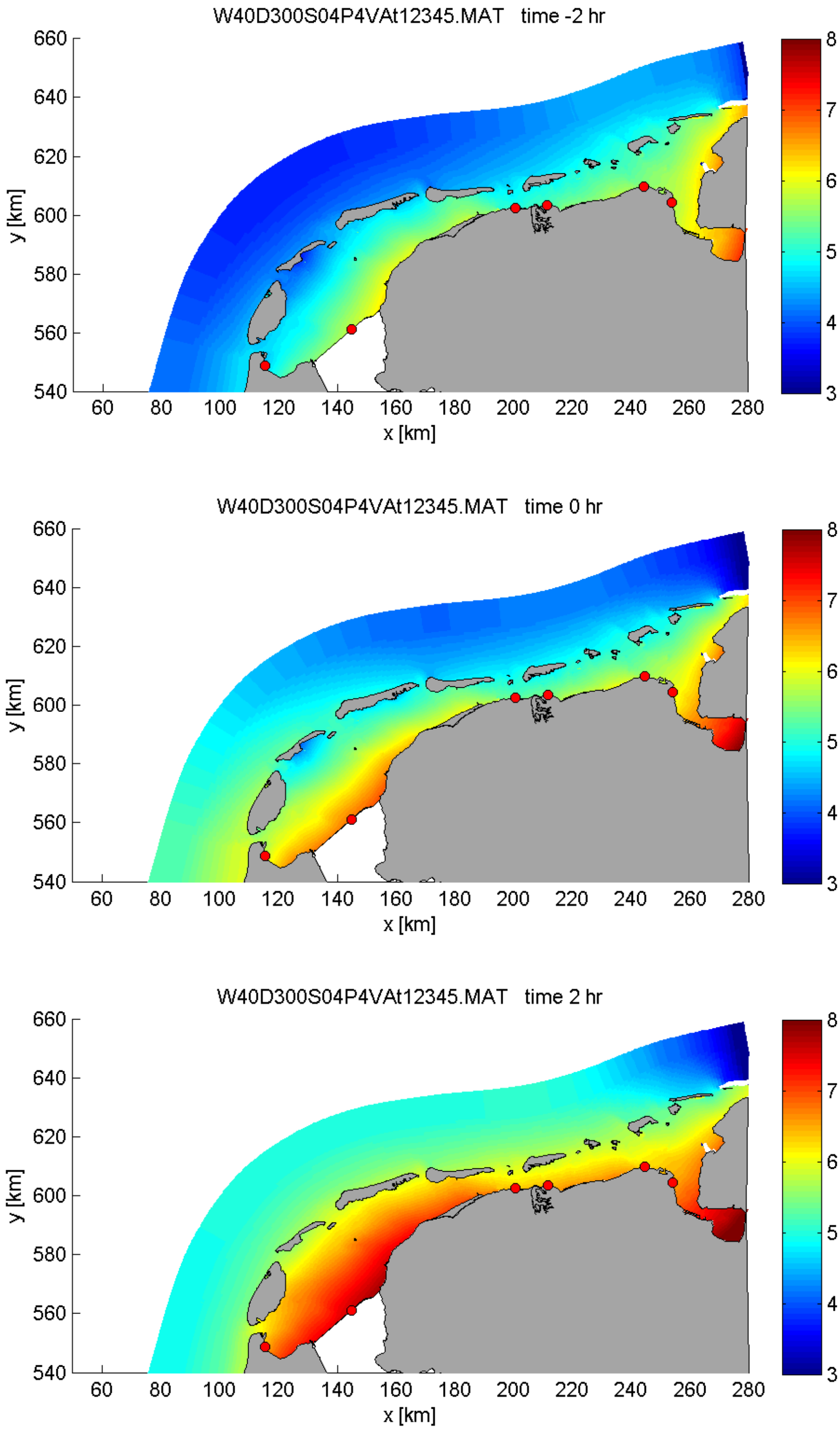


Figure D.3 Water level fields for case with peak wind field of 40 m/s, peak wind direction of 300 °N, offshore surge level of 4 m, and tidal phase difference of 4 hours at 2 hours before the peak, at the peak and 2 hours after the peak of the storm.

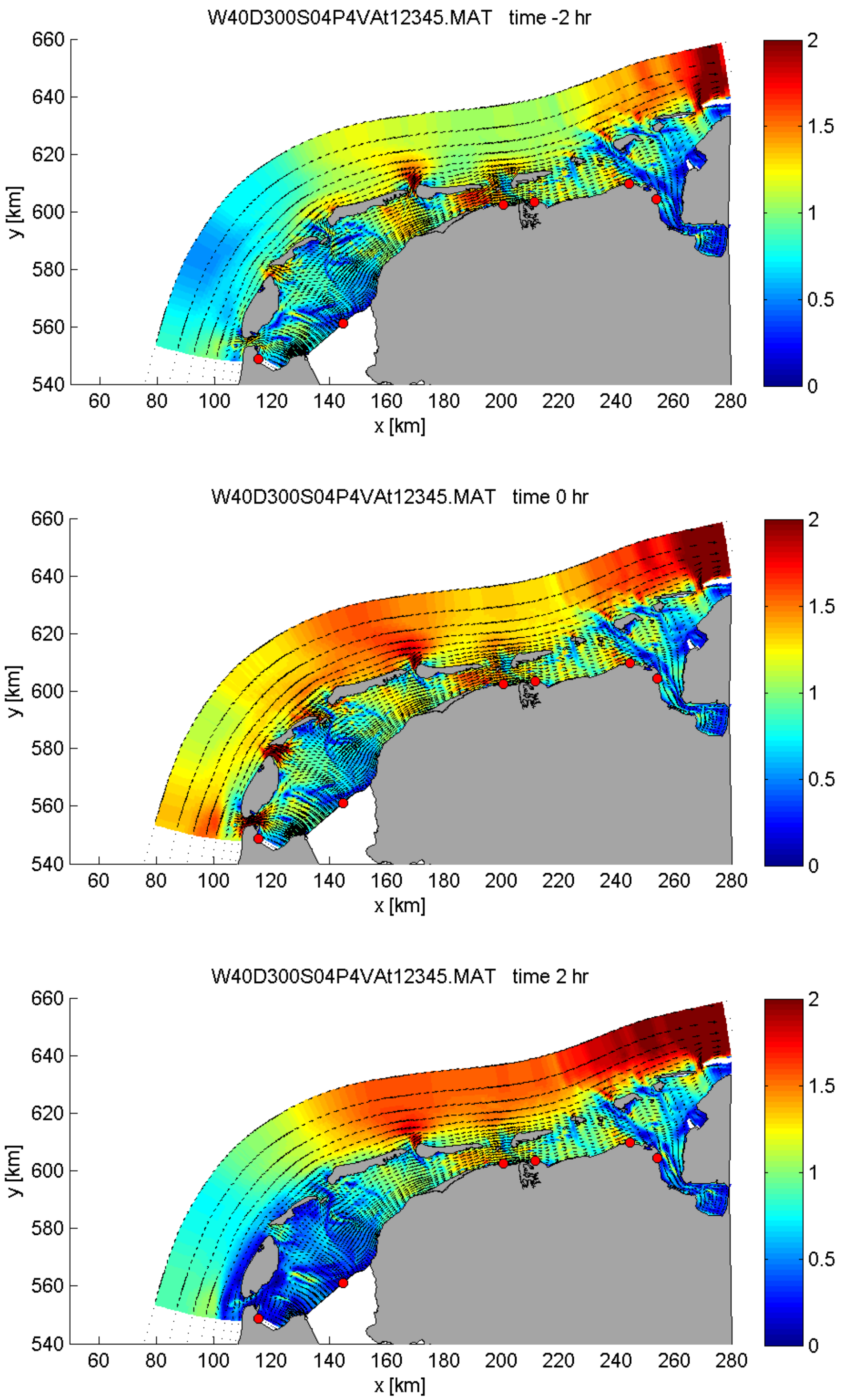


Figure D.4 Current fields for case with peak wind field of 40 m/s, peak wind direction of 300 °N, offshore surge level of 4 m, and tidal phase difference of 4 hours at 2 hours before the peak, at the peak and 2 hours after the peak of the storm.

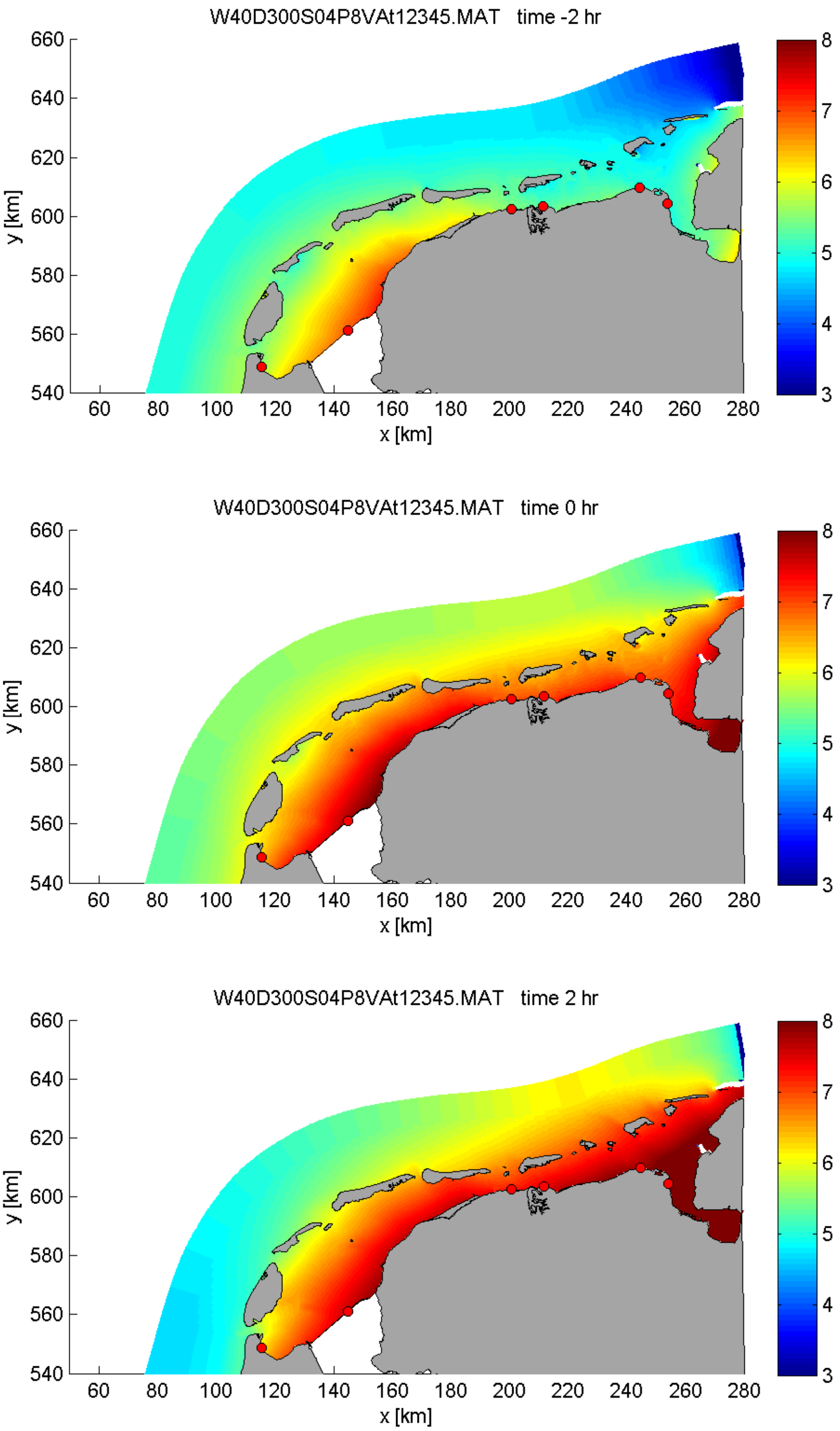


Figure D.5 Water level fields for case with peak wind field of 40 m/s, peak wind direction of 300 °N, offshore surge level of 4 m, and tidal phase difference of 8 hours at 2 hours before the peak, at the peak and 2 hours after the peak of the storm.

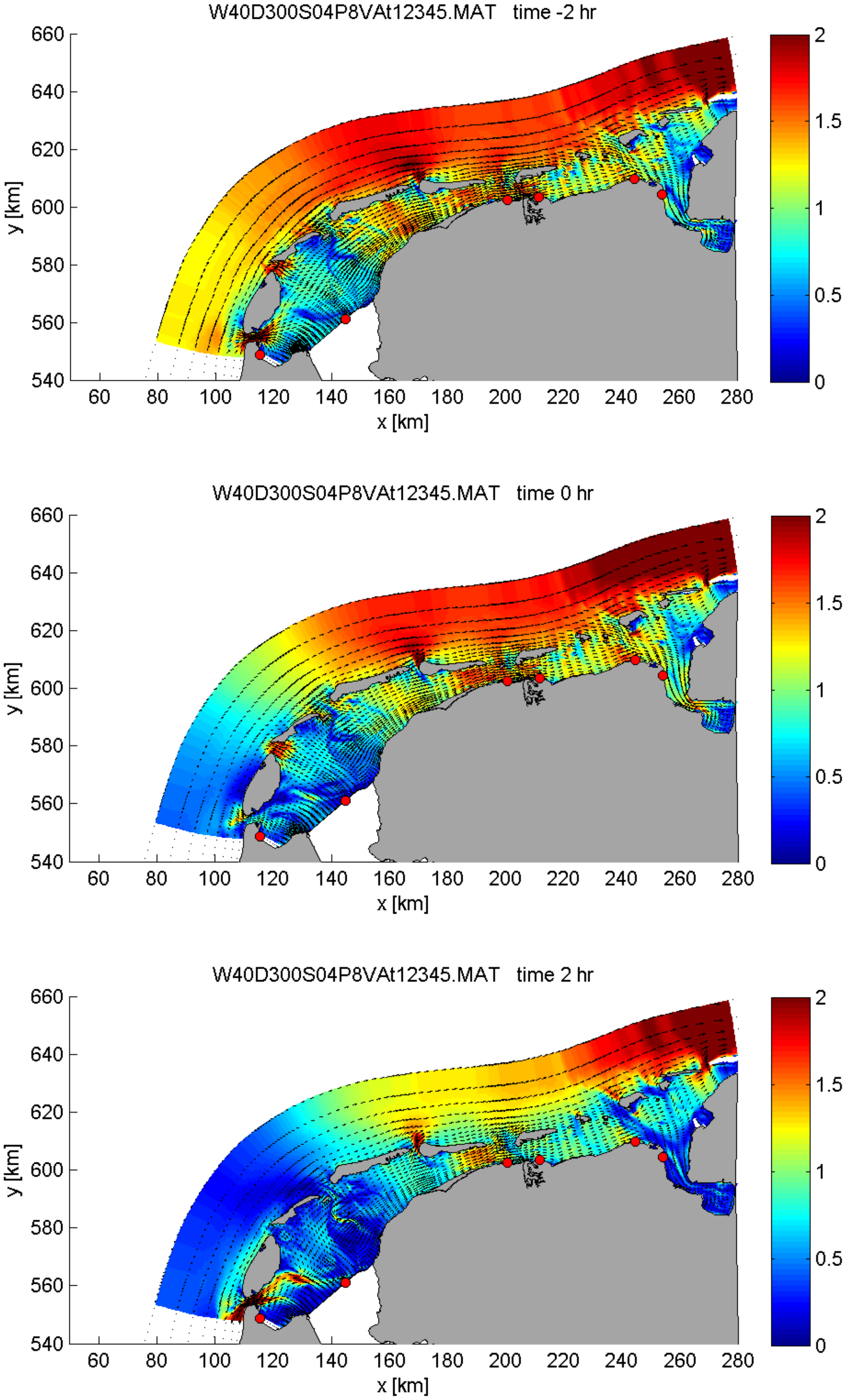


Figure D.6 Current fields for case with peak wind field of 40 m/s, peak wind direction of 300 °N, offshore surge level of 4 m, and tidal phase difference of 8 hours at 2 hours before the peak, at the peak and 2 hours after the peak of the storm.