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Coastal modelling for flood defence

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This paper reviews practices and trends in hydrodynamic and statistical analyses and modelling in the Netherlands with regard to the risk of coastal flooding. We restrict ourselves to the physical phenomena of tides, storm surges and wind waves. We first give a brief outline of established policy in the Netherlands regarding accepted levels of risk of flooding, and current changes therein. This is followed by a summary of a statistical reanalysis of historical storm-surge data combined with numerical hydrodynamic modelling, aimed at improved estimates of probabilities of occurrence of extreme water levels along the Dutch coast. Recent developments concerning the physical and numerical modelling of inundation of low-lying areas are presented. State-of-the-art modelling of wind waves in coastal areas is also reviewed. Research issues in the area of coastal modelling for flood defence are indicated.

Keywords: storm surges; estimation of extreme events; wind waves; coastal flooding

1. Introduction

The Netherlands has a long history of flooding, and many ways of coping with it have been developed. Until now these have always been focused on preventing flooding through technical means, which have varied from earth mounds and simple, small earth dams and dykes built in the early Middle Ages to contemporary advanced engineering works like movable storm-surge barriers. At present, there is a debate about the approach to be taken in the future, in view of the ongoing processes of land subsidence and sea-level rise, the latter possibly enhanced as a result of changes in the global climate. Against this background of defence against coastal flooding, we present a brief overview of the state-of-the-art and current trends in the modelling of the primary threatening physical processes, in particular, storm surges and wind waves, as well as in the modelling of hydrodynamic processes occurring once a flooding event develops.

The structure of this paper is as follows. Before our review of coastal modelling for flood defence, we give a brief outline of established policy in the Netherlands regarding accepted levels of risk of flooding, and current changes therein. We then give a summary of a recent statistical reanalysis of historical storm-surge data along the Dutch coast, covering a period of approximately 100 years, combined with numerical hydrodynamic modelling, aimed at improved estimates of probabilities of occurrence

One contribution of 18 to a Discussion Meeting 'Flood risk in a changing climate'.

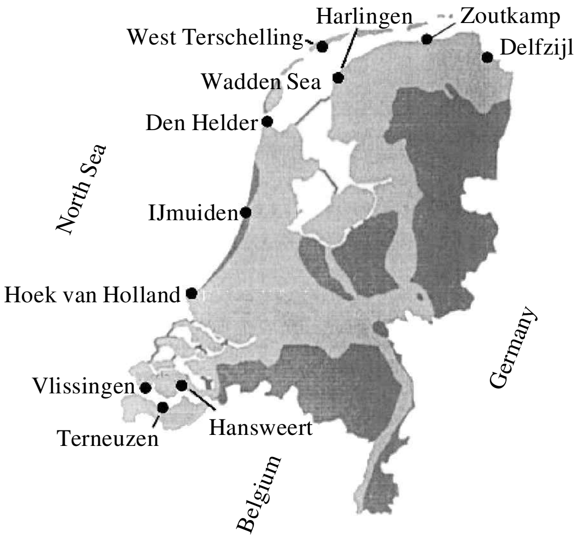

Figure 1. Locations of the principal tidal stations in the Netherlands; the lightly shaded area of the country is susceptible to flooding and needs artificial high-water defences.

of extreme water levels. Next, recent developments in the physical and numerical modelling of the breaching of dykes and of the inundation of low-lying areas are presented. In conjunction with the preceding two items, we describe the development of an operational, real-time cyclone flood warning system. This is followed by a review of the state-of-the-art modelling of wind waves in coastal areas. Finally, research issues in the area of coastal modelling for flood defence are indicated.

Since this is a review paper, rather than a research contribution, details are not presented; these are to be found in the references.

2. Design criteria for (coastal) defence heights in the Netherlands

(a) *Historic overview: the Middle Ages*

The geographical characteristics of the Netherlands are those of a low-lying area along the North Sea, forming the delta of the three rivers, the Rhine, the Meuse and the Scheldt. Without coastal defences, about half of the country would flood at mean spring tide (figure 1). Not surprisingly, the oldest type of formal local government is the water board, in which from the 13th century onwards people organized to share the burden of creating and maintaining dykes and dams to prevent flooding. By systematic damming and draining ‘inner-dyke’ marsh lands and ‘outer-dyke’ tidal flats, the sea could be kept out and pushed back, small islands could be united to form bigger ones, and the coastline could be shortened and made safer. Optimal use was made of the lay of the land and the ridges of higher sand dunes along the coast, while experience taught what dyke heights were needed. A complication in this was the subsiding soil, which largely consisted of peat.

The struggle against the sea was one of give and take. Major floods such as the St Elisabeth Flood (18–19 November 1421) and the All Saints Flood (31 October to 1 November 1570) took tens of thousands of lives, and in one night took back what land had been reclaimed from the sea over hundreds of years. During the period of the Dutch Republic of Seven Provinces (1578–1795), inspectors advised on draining activities and flood protection.

(*b*) *1800–1950: developing a scientific basis*

In 1798, very shortly after the unification into the Batavian Republic, Rijkswaterstaat was created as a single nationwide organization in charge of overall flood defence and larger water regulation issues, next to the local water and polder boards. While the basis of dyke design height was still taken as the largest recorded level plus three feet, a more scientific approach evolved. Around 1800, Inspector Jan Blanken experimented with groynes along the South Holland coast, to reduce dune erosion. Two years of tide measurements near Katwijk (1805–1807) provided the material for the design of sluice systems for the drainage of the low-lying central Holland and the estimates of local safety. The major damming of the Zuiderzee in 1932 by the Enclosure Dam, and the subsequent creation of large polders, was preceded by detailed analytical and quantified studies of the effects on the wave, tide and surge behaviour in the Wadden Sea, to establish the design height of the projected dam and estimate the (enhanced) surge effects for the existing dykes.

(*c*) *The 1953 disaster and the Delta Commission*

Internal government reports in the late 1930s and early 1940s already noted that the level of protection against sea floods in the Southwest part of the country was a serious concern. Due to World War II and its aftermath, it took several more years until in September 1952 plans for increasing dyke heights and studies on damming the major estuaries Eastern Scheldt, Haringvliet and Grevelingen were announced. The floods of 1 February 1953, in which 1835 lives were lost together with an enormous loss of livestock and infrastructural damage, made flood protection a national priority. The so-called Delta Commission (DC) was installed to analyse the flood and to advise on measures to ensure that such a disaster could not happen again. The outcome of this was that the safety standard for the design water level was fixed by law as the storm-surge water level with an expected frequency of exceedance of once in 10 000 years (a return period (RP) of 10^4 years) (DC 1960). This was seen as a reference value, to be applied to the vulnerable and heavily built-up and populated area of central Holland, with lower RP values for other areas. Based on instrumental data (available from about 1880), and statistical techniques of the mid 1950s, the DC developed a procedure to establish the design heights of dykes which by and large still forms the basis of the present approach in the Netherlands. Its central aim is, given a dataset of observed high water (HW) levels, to derive reliably the level corresponding to a given large RP of, for example, 10^4 years, which is well beyond the range directly available from observations. We return to this below, before presenting a more recent reanalysis, using different methods and covering about 50 more years of data.

(*d*) *2000: from frequencies of exceedance of HW levels to probability of flooding*

As indicated above, the DC focused on the occurrence of extremely high water levels as the cause of possible floods. Local design dyke crest heights were determined taking wave run-up (later also overtopping) into account as an additional effect. This has been standard practice in the Netherlands ever since. However, in recent years we have seen the advent of probabilistic design methods in various areas of engineering, involving detailed fault trees with associated probabilities of subevents. In the same vein, it has been increasingly realized in the present context that flooding can be caused by factors other than overflow of dykes, e.g. by dyke-failure mechanisms, such as sliding and piping, or failure of non-functional elements embedded in the body of a dyke, such as old houses that have become part of a dyke as these were widened and heightened in the course of time. Then there are functional, engineering structures, such as navigation locks, evacuation sluices and storm-surge barriers, each of which can cause flooding because of technical failure or malfunctioning (e.g. a gate that is not lowered when needed, due to, for example, power loss or human error). This then has led to the idea that an extreme water level along a stretch of an exposed dyke in itself is not a design criterion, but that attention should be focused on the possibility of flooding of a specific area, enclosed by a closed water-retaining contour that physically may consist of different elements (dunes, dykes, structures, etc.), each of which can 'fail', which in the present context means a lack of protection against high water (TAW 2000).

In the approach sketched above, the probability of flooding in a specific enclosed area—in the Netherlands typically a polder—is estimated from the combined probabilities of subprocesses that contribute to the final event of flooding, taking all relevant elements and associated failure mechanisms and their probabilities into account. This differs from the existing method of using storm-surge water levels only in three distinct aspects:

(i) it considers a closed protective 'dyke' ring instead of isolated stretches;

(ii) it takes account of all conceivable failure mechanisms that can cause flooding;

(iii) it systematically quantifies uncertainties in the estimated probability of flooding, instead of dealing with these post hoc through obscure safety factors.

Recently, this approach has been applied on a trial basis to four separate areas in the Netherlands (TAW 2000). A lot was learned. The results were surprising in the sense that in several cases the engineering structures proved to be the weakest link in a 'dyke' ring. This offers the possibility of prioritizing maintenance and upgrading, as well as more balanced designs in the future.

The method quantifies uncertainties in the final estimates, not only those due to inherent variability in the natural processes and sampling errors, but modelling errors as well. The latter allows the principal gaps in present knowledge to be identified, thus guiding future research efforts.

The trial applications have shown the feasibility and usefulness of the method. It is presently being applied to all remaining low-lying areas of the Netherlands.

(*e*) *2000+: from probability of flooding to risk assessment*

Meanwhile, attention is also being given to the actual risks involved. After all, flooding in itself need not be a hazard; it becomes one only if it threatens vulnerable entities, in particular life, infrastructure, cultural assets, environmental values, etc. Therefore, with contributions from a variety of disciplines (economics, regional planning, agriculture, cultural history, psychology, environmental sciences, etc.), assessments will be made of damages, in the wide sense (including trauma, loss of life), to be expected in case of flooding. This should provide the basis for a more rational cost–benefit approach in the determination of safety levels. It also should help to allow regional prioritization in the investments for safety against flooding.

3. Analyses of historical storm-surge water-level data along the Dutch coast

Whereas the techniques referred to above are being developed and implemented on a trial basis, the legal basis for the determination of design criteria for HW defences, namely, those recommended by the DC, using storm-surge levels with assigned return periods, is as yet (2001) unaltered. In any case, the estimation of probabilities of occurrence of extremely high water levels will remain a key issue, even if it is no longer the only one. In this section we summarize the approach used by the DC (1960), after which we present the key points and results of a later reanalysis.

(*a*) *The DC's approach*

The DC made a thorough analysis of the depression tracks of major storms, followed by a statistical analysis of the higher high waters at Hoek van Holland from the period 1900–1950. Considering only the period November–January, and limiting the data to the highest observation per storm, a set of high waters was obtained that can be considered as independently sampled from a homogeneous population, allowing statistical analysis and manipulation. After various sensitivity analyses, an exponential distribution for extremes was fitted, leading to a level of 514 cm above NAP (the Dutch Ordnance Datum) for an RP of 10^4 years. Based on some further considerations, the DC finally and formally fixed the (exponential) exceedance frequency curve for Hoek van Holland through the level NAP + 500 cm at an RP of 10^4 years.

The exceedance curves for other principal tidal stations considered at the time (Vlissingen, IJmuiden, Zoutkamp, Delfzijl; see figure 1) were determined from the local observations up to an RP of 50 years. For longer RP values, the behaviour was assumed to be about the same as that established for Hoek van Holland, while the transition was estimated from the correlation. Two stations, Den Helder and Harlingen, were affected by the closure of the Zuiderzee in 1932, so that for these only about 25 years of relevant data were available. Here, estimates of extreme levels were obtained by interpolation between neighbouring stations. The results were thought to be conservative. The DC recommended recalculating the design heights for this region at a later stage, when more data would be available. The final DC results for the 10^{-4} HW quantiles at the principal stations, in cm relative to NAP, are presented in table 1, column 2.

(*b*) *Design level reanalysis 1985–1993*

Thirty years later, with more data and more advanced statistical theory and tools, and the Delta project of closing estuarine branches in the southwest of the country completed, Rijkswaterstaat started a major study to recalculate the HW-exceedance curves (De Ronde *et al.* 1995). It consisted of a statistical analysis (Dillingh *et al.* 1993) and a physical analysis (Philippart *et al.* 1993), in which the physical behaviour of storm surges along the coast was analysed using numerical models. Below we summarize the approach and the results.

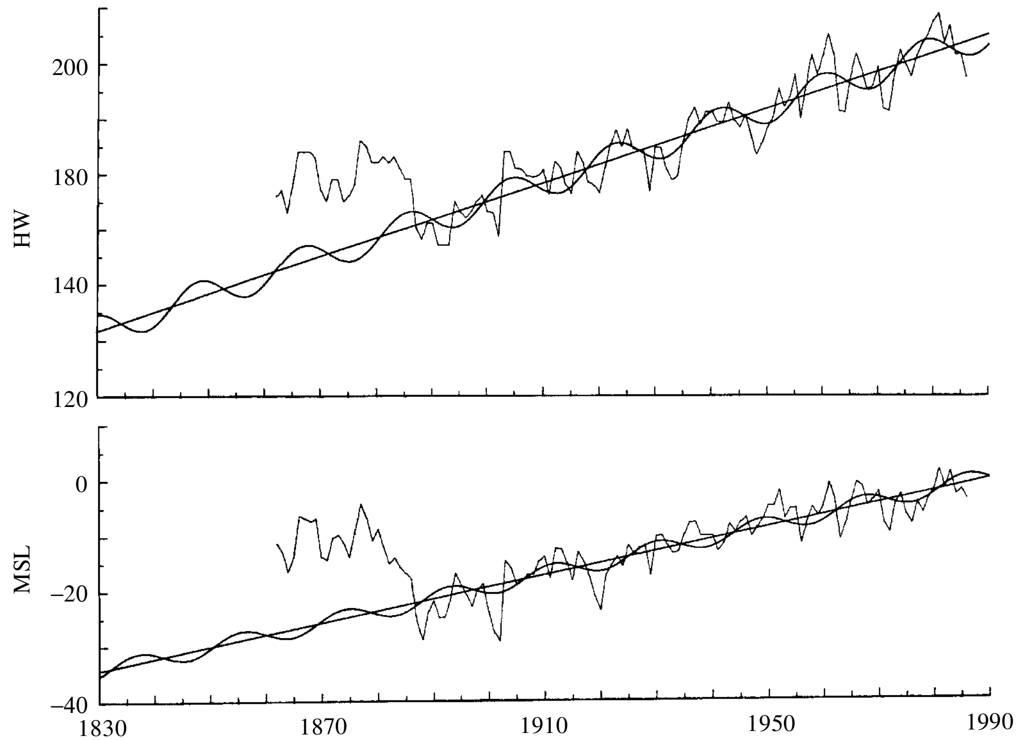

Figure 2. Variation of annual mean HW level and mean sea level (MSL) at Vlissingen, with linear trend and 18.6-year (nodal) cycle fitted to the data after the jump in 1887 (Dillingh *et al.* 1993).

(*c*) *The statistical analysis*

Data at five principal tidal stations were used, covering about 100 or about 50 years (the latter only for the stations affected by the Zuiderzee enclosure in 1932). The observed time-series of HW were homogenized to correspond to the year 1985 by removing existing datum jumps, nodal oscillations with period 18.6 years and a linear trend due to, for example, sea-level rise (figure 2).

The datasets were restricted to events with wind set-up in excess of 30 cm, in the period from 1 October to 15 March. To prevent sampling the same storm event more than once, only the HW levels that are higher than the four preceding and four following ones were retained. The resulting series of HW set-up *and* HW can be considered as independent stochastic samples. (This also holds for HW, notwithstanding the deterministic nature of astronomical tides!) Plotting the annual number of HW set-ups exceeding a given threshold, say u, for $u = 30(20)130$ cm, showed that the (homogenized) time-series is stationary, as required (figure 3).

Various peaks-over-threshold distributions were considered to establish the exceedance curves of HW values for the range exceeding the observational range: generalized Pareto distribution, generalized extreme value distribution or GEV (on storm season maxima), convolution method. The results differed essentially depending on the assumptions and the station considered. Finally, a distribution free method (DFM) was developed and used to estimate the 10^{-4} (and other) HW quantiles (De Haan

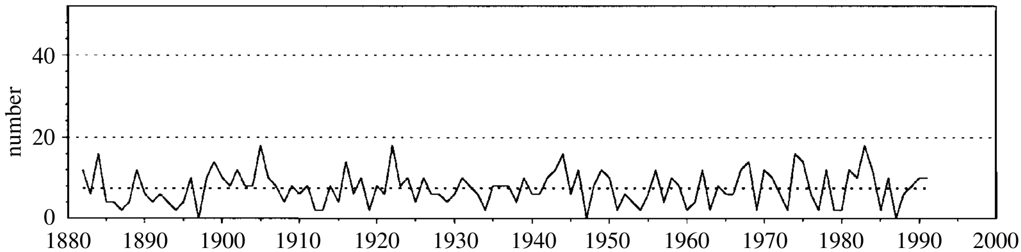

Figure 3. Annual number of HW wind set-ups at station Delfzijl exceeding a value $u = 110$ cm (Dillingh *et al.* 1993).

1990). Its application was thought to be the most appropriate for the problem at hand. Based on this, GEV-type exceedance frequency curves were determined for all stations, together with the 95% confidence bands.

The DFM-estimated 10^{-4} HW quantiles, in cm relative to NAP, are presented in table 1, column 3. The present statistical reanalysis clearly confirms the conservative bias of the design standard for the Wadden Sea area (Den Helder, West Terschelling, Harlingen) as applied by the DC.

(*d*) *Physical analysis: variation of intensity, duration, phase and track of extreme storms*

Using numerical hydrodynamic models, tides and storm surges in the North Sea were simulated to supplement the results from the purely statistical analysis, with two main aims: (1) to investigate the sensitivity of storm-surge level to variations in the wind fields, and (2) to investigate correlations between storm-surge levels at different stations and to obtain data for interpolation between principal tidal stations (enhanced spatial resolution).

In these simulations, historical extreme storms were modified within reasonable bounds, as seen from a meteorological viewpoint, but no *a priori* probabilities were assigned to the resulting combinations. The results are therefore not predictive on their own; they were tied in to the results from the statistical analysis of the HW levels at Hoek van Holland, which was used as a reference station because of its central location (also least affected by the major engineering works in the Zuiderzee and the southwest delta region), and to provide continuity with past approaches. The new results supplement the observational data but do not replace these. (However, if in the future reliable (regional) climate models were to be developed, as part of ongoing research on global climate change, it may become possible to assign probabilities to extreme windfield characteristics, in which case this methodology could be used to assess probabilities of extreme surge levels.)

(*e*) *Numerical models*

The approach was based on simulations of the behaviour of the surge in the North Sea and along the Dutch coast using the Dutch Continental Shelf Model, which is used operationally at KNMI four times a day to update the surge predictions for the coming 36 h (Verboom *et al.* 1992). Two more local models with higher grid resolution were nested in the Continental Shelf model: the southern North Sea model and the Wadden Sea model.

(*f*) *Variations of intensity, duration and phase*

Four historical storms were selected on the criteria that they resulted in potentially high levels along the whole coast and have the characteristics of being potentially extremely serious. For three of these storms, variations were considered in terms of intensity (pressure differences and wind speed multiplied by 1.00, 1.25, 1.50 and 1.75) and duration (factors 1.00, 1.25 and 1.50). The influence of the phase of the storm with respect to astronomical HW was varied as well, to determine the time shift leading to combinations of maximum levels for a given station when paired with Hoek van Holland. These 36 permutations were simulated and for each station the correlation of HW with that in Hoek van Holland was determined by fitting a regression line through the seven to nine most severe cases. At the quantile for an RP of 10 years these correlation lines were subsequently fitted to the local exceedance curve, based on observations (table 1, column 4).

(*g*) *Variations of storm track*

Lastly, effects of variation of storm track were investigated. From the point of view of the spatial scales of meteorological circulation patterns, a specific depression track and associated storm field over the North Sea could essentially have occurred with a horizontal shift of location of up to several hundreds of kilometres with the same probability. The 1953 storm situation, with wind speed enhanced by a factor of 1.25, was used to assess the influence of shifted storm fields on the HW set-up level, considering eastward shifts from −320 km to +480 km and northward shifts from −320 km to +400 km, both with 80 km intervals. The results obtained for shifts to sea of typical land wind ridges were discarded. The results show increased HW levels for the quite realistic shifts of 160–240 km to east–southeast. They were again adjusted to fit the reference 500 cm level for Hoek van Holland (table 1, column 5). Using weighted averaging, the results were combined with those from the variations in intensity and phase to obtain the final set of 10^{-4} HW quantiles from the physical investigation (table 1, column 6).

(*h*) *Results*

Lastly, weighted averages of the values from the purely statistical reanalysis and those from the physical analysis were determined, using the inverse of the estimated uncertainties in the results as weighting factors (Van Urk 1993). Table 1 (column 7) gives the results; the differences with the estimates given by the DC (1960) are given in the last column.

(*i*) *Adjustment of design heights for effects of climate change*

The approach for the determination of the design heights summarized above has taken into account the effects of sea-level rise. It has also considered the effects of shifts in the storm fields, based on the argument that at present climate conditions the probability of occurrence of a given storm field and the same one shifted up to several hundreds of kilometres is the same. When considering climate change, the above approach will remain basically the same. In case more information on the probability of occurrence of specific changes in storm tracks becomes available, this could be translated into a different weighting of intermediate and final results.

Table 1. *Final* 10^{-4} *HW quantiles*

(HW quantiles (in cm above NAP) by weighted combination of statistical and physical results (column 7), compared with the 10^{-4} quantile results of the DC (column 2).)

			physical investigation				
station	DC (1960)	statistical reanalysis	scaling/ phase	track shift	weighted	combined weighted	corr. to DC (1960)
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(7)–(2)
Vlissingen	565	540	555	550	552	545	−20
Hoek van Holland	500	500	500 (ref.)	500 (ref.)	500 (ref.)	500	0
Den Helder	505	425	487	396	447	441	−64
West Terschelling	530	405	496	409	457	428	−102
Harlingen	580	460	569	517	548	501	−79
Delfzijl	640	600	665	409	634	613	−27

4. Modelling of (coastal) flooding

Modelling of surge levels for low-lying coastal areas that are not, or insufficiently, protected by natural dunes or man-made dykes will only give realistic results if overland flooding is included. Such flows involve three different types of transient transitions: those between flow and dry land, between essentially one-dimensional flow in conduits like ditches, canals, creeks and rivers and essentially two-dimensional overland flow, and between subcritical and supercritical flow. Recently, a robust and accurate numerical code has been developed for the representation of these phenomena (Stelling *et al*. 1998; Stelling 2000), implemented as the Delft–FLS package of Delft Hydraulics. It is based on the positivity of the water depth and the conservation of mass and momentum, implemented on a staggered grid. The following statements concerning this software package are taken almost literally from Stelling (2000).

The model is specially suited to simulating the dynamic behaviour of overland flow over initially dry land, as well as flooding and drying processes on every kind of geometry, including lowlands and mountain areas. It also gives accurate and stable results in computations of flow on very steep slopes, such as dyke walls, structures, etc.

The package is provided with an automatic time-step estimator which reduces or enlarges the computational time-step according to the flow characteristics at any moment of the simulation. The package requires good topographical data. It can be given as a GIS-layer (Geographical Information System) or manually on a suitable rectangular grid. The levels and positions of roads, levees and other infrastructure are also required. A land-use map (or a similar) is also necessary. The 'land use' must be converted to hydraulic roughness through standard experience-based ecotype-roughness relationships. Internal boundary conditions are included to simulate dam- or dyke-break events. The possible outputs are water levels, water depths, velocities (magnitude and velocity fields) and inundation depth class intervals. These results can be given in GIS format for presentation and further post-processing. Presentation of the results using other packages is also possible.

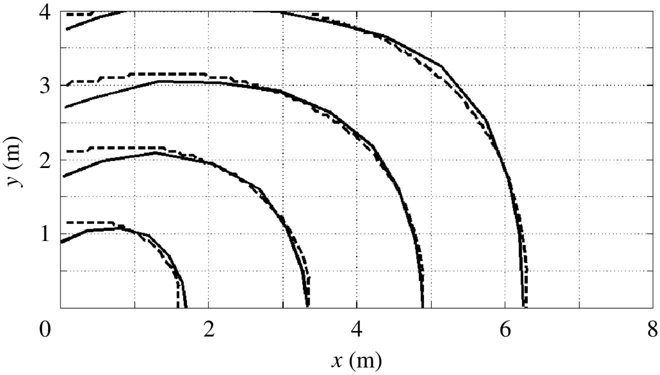

Figure 4. Plan view showing positions of front of inundation surge on a horizontal bottom at 1 s, 2 s, 3 s and 4 s after the sudden opening of a 40 cm wide gate in the centreline of the basin, allowing free flow out of a reservoir into the basin. The initial water depth is 5 cm and the initial head difference is 55 cm. Downstream (x) and lateral (y) coordinates are given, with the origin in the centre of the gate opening. Only one-half of the basin is shown because of symmetry with respect to the centreline ($y = 0$ m). Solid line, results from camera observations; dashed line, results from numerical simulation with Delft–FLS (Duinmeijer 2002).

The Delft–FLS package can be used for flood simulation, damage assessment and risk evaluation for coastal areas, river valleys, mountain areas and lowland areas. The flow module is used to determine the flooded area at GIS pixel level including the water depth and inundation period.

Damage assessment is possible provided sufficient data of land use and investments are available within a GIS system. Risk maps can be built based on the damage estimation and the combined probability of occurrence of high water and other disasters such as a dam or dyke break or any other hazard. It can be used during landscape- and urban-planning phases, showing the influence of the existing or future infrastructure on the flow pattern. It is extremely useful for the design of evacuation plans, since it shows the inundated areas and the available transport ways at any moment in time. The results can be used within a decision support system. (Here ends the almost literal quotation from Stelling (2000).)

The flow simulation package described by Stelling (2000) is for two-dimensional flow. It has recently been verified against observations of flow over a horizontal concrete bottom in a laboratory basin, following a simulated dyke breach (Duinmeijer 2002). The observed rate of progress of the advancing and laterally spreading front as well as the ensuing water-depth variation in time was very well reproduced by the model, both for flooding of an initially dry bed and in the case of an inflow into an initial shallow layer of stagnant water (figure 4).

The two-dimensional model has recently been coupled to an existing one-dimensional model, allowing full exchange between the two flow types, such as occurs in flooding and drying of low rural areas dissected by ditches, canals, creeks, rivers, etc., but also of urban areas with storm-water sewage systems.

5. Operational forecasting of storm surge and flooding

Operational storm-surge forecasting has been used in different areas worldwide for many years. In the Netherlands, an automatic production line at the Royal Netherlands Meteorological Institute is used for that purpose. It contains a limited-area atmospheric model (called HIRLAM), the output of which drives a surge forecast model for which the above-mentioned Dutch Continental Shelf Model (DCSM) is used (Gerritsen *et al*. 1995). Also, wind-wave forecasts are made using NEDWAM, a regional, North Sea version of the more general WAM model (wave model; see Komen *et al*. (1994)).

The DCSM model covers the area from 48° to 62.33° N and from −12° to 13° E, with a resolution of $\frac{1}{12}^{\circ}$ in northern and $\frac{1}{8}^{\circ}$ in eastern directions. Given the progressive nature of most surges, which travel south along the UK East coast, the initial state is optimized very effectively by a Kalman filtering module that assimilates upstream water levels along the UK coast in combination with oil platform data. Model forecasts extend for 36 h and are refreshed every 6 h. The model is continually being improved by extensive evaluation of the results for each storm season. Presently, the assimilation of atmospheric pressure and wind speed data is being investigated, as errors in these forcing parameters appear to be the major sources of the remaining forecast inaccuracies.

Vatvani *et al*. (2002) describe a suite of coupled models as part of an operational Storm Surge and Flood Forecast System for the east coast of India that was recently developed and implemented. It contains models for the prediction of

(i) atmospheric pressure and wind speed and direction,

(ii) tides and storm surges,

(iii) flooding of low-lying coastal areas,

(iv) rainfall,

(v) wind damage,

embedded in a GIS-based decision support system.

A key feature is a scheme for the parametric representation of the cyclone wind and pressure fields that force the flow, with high spatial and temporal resolution. The typhoon/cyclone wind force is simulated using the analytical model of Holland (1980), with its model parameters based on the track advisories, the maximum expected wind speed, etc. For operational use, enhanced typhoon/cyclone wind forecasts combined with background wind from the India Meteorological Department are foreseen. However, the module allows for input data from arbitrary sources.

A second feature is the module to simulate the overland flooding of the coastal zone and river delta areas, using the Delft–FSL package. The effect of tide–storm-surge interaction is included. The concepts and the application have been validated using a number of actual cases including the supercyclone ORISSA. It is shown that the surge and flooding model is able to reproduce the inundation with a high degree of accuracy.

6. Modelling of wind-generated waves in coastal water

The review given above has so far been restricted mainly to modelling of (extreme) water levels as potential hazards for coastal flooding. In cases of low areas protected by dunes or dykes, as in the low countries surrounding the southern North Sea, wind waves occurring in storms may severely damage these high-water defences and cause their failure, thus posing another flooding hazard that has to be considered.

(*a*) *Numerical wave prediction models*

Present-day numerical wind-wave prediction models for oceans and shelf seas are based on the concept of the spectral energy balance equation, describing the space-time evolution of the two-dimensional spectrum with advection based on linear potential wave theory and source terms accounting for wind–wave interaction, nonlinear wave–wave interaction, wave–current interaction and dissipation. Reference is made to Komen *et al*. (1994) for a general review and a detailed description of the so-called WAM model, presently the most widely used model in this category.

Modelling of wind-wave propagation in coastal areas differs essentially from that in deeper waters because the small depths cause different processes to be dominant, in particular, triad interactions and depth-induced breaking, whereas depth variation on small spatial scales requires a numerical resolution for which conventional deep-water models are not suited because of the Courant stability condition. Thus, dedicated models have been developed specifically for shallow water and coastal areas, even extending into the surf zone. These can be distinguished in phase-resolving and phase-averaged models (see Battjes (1994) for a review of shallow-water wave modelling). The former are used to study details of wave transformation and wave–structure interaction, the latter for propagation and local generation in more extended areas for which the phase-resolving models are not feasible.

The present-day prototype for shallow-water phase-averaged random wave models is the so-called SWAN model (Booij *et al*. 1999; Ris *et al*. 1999) (available in the public domain; see http://swan.ct.tudelft.nl/). Its basic formulation is the same as that of WAM, but there are numerous modifications and additions (among others surf breaking and triad interactions) as well as a completely different, unconditionally stable numerical scheme. SWAN is used extensively to establish design wave conditions in coastal areas.

Whereas all phase-averaged models are quite similar in essence, this is not the case for the phase-resolving models. The so-called Boussinesq models form an important subset (see, for example, Madsen & Schäffer 1998; Chen *et al*. 2000; Kennedy *et al*. 2000; Borsboom *et al*. 2001). These are depth integrated and therefore less demanding than the more powerful models that fully resolve the vertical, e.g. volume-of-fluid (VOF) models (see, for example, Lin & Liu 1998). A versatile VOF model developed specifically for wave–structure interaction including breaking, run-up, overtopping and pore flow is presented by Van Gent *et al*. (1994). A simpler, hydrostatic and depth-integrated wave–structure interaction model is given by Van Gent (1994).

(*b*) *Combined extreme-value statistics for water levels and waves*

Extreme wave conditions and extreme water levels are often treated as fully correlated. This is a simple but conservative approach. It is formally better, but more complicated, to take account of the joint probabilities of the relevant variables. In these cases, the scarcity of data is perhaps even more a limiting factor than it is in cases of a single random variable, and modelling as an additional tool becomes more important. An example of this can be found in the establishment of the design conditions for the movable storm-surge barrier in the Oosterschelde, an estuary in the southwest of the Netherlands (Vrijling & Bruinsma 1990). The approach was based on combinations of storms of random duration and intensity and in random phase with respect to HW, with incident waves from the North Sea limited by the shallows in the mouth of the estuary, and local wave generation.

Some preliminary analyses for a structure near the Dutch Wadden island of Texel concerned the joint extreme-value statistics of water levels and wave heights. Starting from the marginal statistics and the determination of the correlation structure, the joint extreme value distribution was calculated. The joint distribution of extreme levels and wave heights was then used to determine the failure area for a given exceedance frequency (De Valk 1994).

In estuaries and coastal regions near outflow of major rivers, combinations of storm-surge water levels and high river discharges need to be considered. In fact, these are less likely to be strongly correlated than surge water levels and waves, since those both result from the action of the wind. Thus, a simple approach of combining extremes of equal marginal exceedance probability is overly conservative, and a joint probability approach is called for.

7. Closing comments

We have given a sketch of current practices and trends in the Netherlands concerning coastal modelling for flood defence. In closing, we indicate areas where further research is needed.

(*a*) *Research issues in modelling of wind-induced flows*

To assess the flood risk in a changing climate by modelling, the transport (flow) characteristics of our models should represent actual transports, as well as levels. A well-known and still outstanding issue is the modelling of air–sea interaction processes for more general circumstances. At present, a simple quadratic friction law is generally used, with a so-called drag coefficient that is fitted on observations to represent the local basin characteristics. The coefficient is generally assumed to be uniform and a simple function of the wind speed, and effectively lumps the effects of basin depth, length-scales and wave fetch length.

The calibration of air–sea drag coefficients is usually based on the evaluation of water levels for persistent and extreme wind effects over relatively short periods. Since the associated short time-scale transports in those cases are also largely persistent in direction, it may be assumed that the associated transport is also reasonably well represented. However, using the same calibration, the time-integrated transport modelled for non-extreme, meteorological forcing, which fluctuates on the time-scale of hours, is not properly represented at all. This is borne out by the large variation of model estimates for the tidal and wind-induced net transport through the Straits of Dover as reported in literature and summarized in Gerritsen *et al.* (2000).

A fundamental investigation of the air–sea interaction, including the effects of wave growth and the associated change of sea-surface roughness and energy available for flow motion, plus the matching of turbulence quantities in the lower atmosphere with those in the upper part of the water phase, is warranted. Equally, improvement of the vertical penetration further into the water column, presently enhanced by assuming a decaying background vertical mixing coefficient, needs to be addressed in a more fundamental way.

(*b*) *Research issues in the modelling of inundation*

The advent of numerical models for inundation such as Delft–FLS is a significant advance. Much effort should be given to further calibration and validation, particularly with respect to the storage capacity and flow resistance due to various subgrid natural or man-made elements in the terrain, as in urban areas. A related topic needing further development is the process of breaching of dykes or dams. These are needed for realistic predictions of flooding of low areas, since a breach widens as the flow proceeds, thus enhancing the rate of influx. Visser (1998*a*, *b*) has presented a model for breach growth in sand dykes, validated against laboratory experiments and full-scale field tests. It needs to be extended to make it applicable to other materials of the dyke body (e.g. clay) and/or cover layers.

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