ELSEVIER

Coastal Engineering 25 (1995) 1–41

COASTAL ENGINEERING

Evaluation of 10 cross-shore sediment transport/ morphological models

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Received 17 August 1993; accepted 17 November 1994

Abstract

Cross-shore sediment transport models are used to model beach profile changes in order to determine, for example, coastal set-back lines, behaviour of beach fill and beach profile variations adjacent to coastal structures. A study was undertaken to evaluate ten of the most well-known mathematical cross-shore transport models with regard to different model requirements. The characteristics of these time-dependent models were investigated and the pros and cons of each are listed. The ranges of the data used to verify and calibrate these models are noted. It is concluded that the models can be classified generally into three groups with regard to their theoretical basis (re. mainly sediment transport) and the extent to which they were verified (re. mainly morphodynamics). These groups are termed the "best", "acceptable" and "less suitable" groups. However, it is very important to consider the specific purpose of a model application. In some instances one model may perform better while for a different purpose another model may be better. Data are generally lacking for accretionary events and for erosion cases where the significant wave heights exceed 2.5 m. Aspects presently usually not included in these models are also listed. Without direct comparative prototype tests the final conclusion as to which are the better models in practice cannot be given. Furthermore models may be best applicable under different specific conditions. Models are also constantly being improved and thus a comparative evaluation of the models can only be completely accurate for a relatively short time.

1. Introduction

1.1. General

The study of beach profile change in the broad sense covers nearshore processes that shape the beach on all spatial and temporal scales. Beach profile change is a phenomenon of fundamental interest and, as such, has been studied by geologists, oceanographers and

SSDI 0378-3839(94)00040-9

coastal engineers. In coastal engineering, quantitative understanding of beach profile change is pursued mainly to allow prediction of beach evolution in the vicinity of planned or existing coastal developments.

The types of coastal engineering problems for which predictive tools are needed, are beach and dune erosion that occurs under storm waves and high water levels, prediction of set-back lines, adjustment of beach-fill to long-term wave action and the prediction of sediment build-up or beach profile envelopes for the planning/construction of pipelines and structures such as tidal pools.

In the past, the prediction of beach evolution was mainly conducted by relying on experience and on the results of hydraulic model tests. In more recent years, however, numerical models have gradually been developed and are increasingly applied for this purpose.

Empirical methods involve forecasting based on observed trends of evolution of the relevant beach or estimating it by comparison with the evolution of other beaches under similar conditions. This method has the merits of simplicity and, to a certain degree, reliability in the sense that it is based on actual data. However, it is problematic to make a quantitative prediction of beach evolution solely by empirical methods.

In hydraulic model tests, beach evolution can be studied under controlled conditions using scale models of the beach (and structures) according to the construction plan. However, model tests of beach evolution involve significant scaling and calibration problems. In addition, hydraulic model tests usually require expensive facilities and a great deal of labour and time.

The limitations of the empirical methods and hydraulic model tests have been recognized and, as computers have become more powerful, numerical models have gradually replaced these conventional methods for predicting beach evolution. The development of numerical models has been accelerated by the requirement for higher accuracy in the prediction of beach evolution due to increasing human activities on the coast, and by progress in the understanding of the beach processes — which is one of the most difficult subjects in coastal engineering.

The erosion and accretion of sand beaches, caused by transport normal to the shoreline, has been widely investigated. A number of theories have been proposed to explain the observed behaviour. Some of these theories are based on analytical grounds and others on empirical results derived largely from model experiments. The models developed from these theories vary widely in predictive potential. They range from qualitative cause-and-effect statements through predictors only of the direction of net transport (that is, erosion or accretion), to models which provide quantitative estimates of local transport rates and time-dependent beach profiles. Comprehensive field investigations that allow concurrent evaluation of the various models are scarce. Models should normally be calibrated to local conditions.

Some previous evaluations of cross-shore sediment transport models have been reported. Seymour and King (1982) evaluated eight models for predicting cross-shore transport using beach profile data from the Torrey Pines experiment of the Nearshore Sediment Transport Study (NSTS). None of the models showed useful skill in predicting the direction of net transport. Seymour and Castel (1988) used more data sets to re-evaluate four of the models tested by Seymour and King as well as another six not previously tested. However, only two models that could predict profile changes were evaluated. These were the models by Quick and Har (1985) and Swart (1976). They found that the Quick and Har model did not give good results but that the Swart model gave good results on an unbarred beach.

Horikawa (1988) gives a review of some of the sediment transport models available at that time. Kraus et al. (1991) evaluated a number of beach erosion and accretion predictors. Kriebel et al. (1991) reviewed engineering methods for predicting beach profile response based on equilibrium profile concepts. The Vellinga (1986) and Bailard (1981) models have been reviewed by Birkemeier et al. (1987). Nairn and Southgate (1993) includes a brief review of the energetics approach (Bagnold, 1963, Bailard, 1981).

Recently Brøker Hedegaard et al. (1992) made an inter-comparison of six different models for short-term coastal profile modelling. The models were tested against measured profile evolutions from a large wave flume (under dune erosion conditions). The six models were: LITCROSS (Danish Hydraulics Institute), UNIBEST (Delft Hydraulics), NPM (Hydraulic Research), WATAN 3 (University of Liverpool), SEDITEL (Laboratoire National d'Hydraulique), REPLA (SOGREAH). Brøker Hedegaard et al. conclude in general that:

- The models underestimate the offshore transport on relatively steep profiles.
- The swash zone processes and dune erosion are not described in the models.
- The velocity field in the area just before and after the break point is still understood rather poorly.
- Finally, they conclude that the understanding of cross-shore processes has now reached a stage where it is relevant to extend the models into 3D to find the "weakest point".

Models which only make qualitative cause-and-effect statements or predict only the direction of net transport or only predict (beach) slopes are beyond the scope of this study. Solutions for the maximum potential erosion as predicted by some of these methods tend to over-predict the actual erosion response. These solution procedures assume that the erosive conditions are maintained for a relatively long time such that the equilibrium profile response can be fully achieved. In reality, however, water-level and wave conditions are never constant, and a realistic solution must account for the relatively slow morphologic response of the profile in comparison to the faster variation in the applied hydrodynamic forcing conditions (Kriebel et al., 1991).

The purpose of this paper is to assess the theoretical merit of a number of mathematical cross-shore sediment transport models, to summarise the characteristics of these and to investigate the ranges of the data used to validate these models, mainly in terms of morphodynamics. Only time-dependent models were reviewed as they are considered to theoretically better represent actual coastal processes. This included most of the true two-dimensional models (or part of 3D models) of beach evolution. The models were chosen mainly on whether they are well-known internationally, whether they have already been widely applied with success and on the authors' initial perceptions of the models' merit.

The ten models discussed in this text are (in chronological order; detailed references are given later):

(1) the Swart model
(2) the Dally and Dean model
(3) the Bailard model (almost identical to Bowen, 1980; see Bailard, 1991).

(4) the Kriebel and Dean model
(5) the Shibayama model
(6) the Watanabe model
(7) the Nishimura and Sunamura model
(8) the Steetzel model
(9) the Larson and Kraus model
(10) the Danish Hydraulic Institute model

The models were classified according to the cross-shore sediment transport approach. For this reason, the models by Stive (1986) and Nairn (1990) have been reviewed as part of the Bailard model. No distinction was made between various types of transport (sheet flow vs. rippled bed or equilibrium load vs. actual load) as it is not possible at this stage to indicate generally what type of transport is relevant for a particular practical problem. Methods such as those by Bruun (1962), Edelman (1968), Quick (1983), Vellinga (1986), Nielsen et al. (1978), Hughes (1983) and Briand and Kamphuis (1990) were not considered.

It may be said that morphological models should not be judged only by their sediment transport module as there are many other aspects to such models which may even be more critical than the transport module. We agree with this and believe that a complete model is only as good as its worst part. However, in this paper we have taken the view that the transport module is the central part of the complete model to which the other modules (e.g. wave module) are linked. Our view is that, for example, many different wave modules could be linked to the same transport module. Thus our evaluation of the theoretical basis of the models is mainly concerned with the sediment transport module. On the other hand our evaluation of the verification data and validation of the models is mainly in morphological terms. It is well-known that representative prototype or field data on sediment transport is very hard to obtain. We have concentrated more on the "final results" as predicted by the models and on the usually more readily available morphological data. This approach also very much agrees with the view of the practitioner whereby the finally predicted beach profile or the envelope of profiles is by far the most important result of a model. Important practical decisions are usually based on such results.

1.2. Model requirements

Before an assessment is made, it is useful to state what is required of a beach evolution model and what aspects should be considered in order to evaluate different models. Many different requirements have been noted by various authors. The requirements as listed by Larson et al. (1990a) are repeated here.

Larson et al. state that for practical use, five model capabilities were considered to be essential:

- accurate and reliable beach change simulation compatible with input data routinely available at engineering projects;
- representation of sand transport and beach change on temporal and spatial scales of engineering interest;
- representation of general boundary conditions and coastal structure configurations;

- calculation robustness, meaning that uncertainties typically present at projects do not produce aberrant model predictions; and
- economical execution time.

The second capability implies that both short-term processes (e.g., storm-induced beach erosion/recovery, and cyclical daily and seasonal change in the beach profile shape and position) and medium-term processes (e.g., accretion and erosion at shore-normal structures) are simulated, including approach to an equilibrium bottom configuration under constant forcing and boundary conditions.

For a more extensive list of criteria to which an ideal profile change model should adhere, see Birkemeier et al. (1987).

For a model to be at least theoretically acceptable and practical to use, in the authors' opinion, it must comply with the following minimum basic requirements:

- The model must have a sound theoretical basis without inappropriate assumptions.
- The model must be calibrated and verified against a wide range of data/conditions.
- The model must be able to simulate both erosion and accretion events and thus provide an envelope of beach profile response.
- The dynamics of macro-scale profile change, such as the growth, movement and decay of both bars and troughs, must be simulated. The importance of this was emphasised by Birkemeier (1991).

These requirements will be examined for every model so that each can be categorised in either of three groups, namely, "best", "acceptable" and "less suitable" groups. The class of problems to which the model should be applicable for this evaluation can be specified as mainly short and medium-term problems.

The simulation of cohesive sediment transport and gravel/shingle transport were considered to be outside the scope of the present evaluation.

In the next section the theoretical basis and verification data on which each model is based, are discussed. Thereafter the characteristics of the models with regard to these aspects are compiled and discussed, followed by conclusions and recommendations.

2. Theoretical basis of transport theory and morphological verification data

2.1. Method

The theoretical basis of each model (re. mainly sediment transport) is briefly discussed as well as the assumptions that were made. Then the data ranges, testing, calibration and verification of the models (re. mainly morphodynamics) are summarized.

2.2. The Swart model (Swart, 1974a, b, 1976, 1986)

Theory

In deriving a 2-line beach evolution model for a coast with a groyne system, Bakker (1968) postulated that cross-shore sediment transport can be approximated by two characteristic horizontal distances. Swart (1974a, b) schematised the beach profile into 3 zones

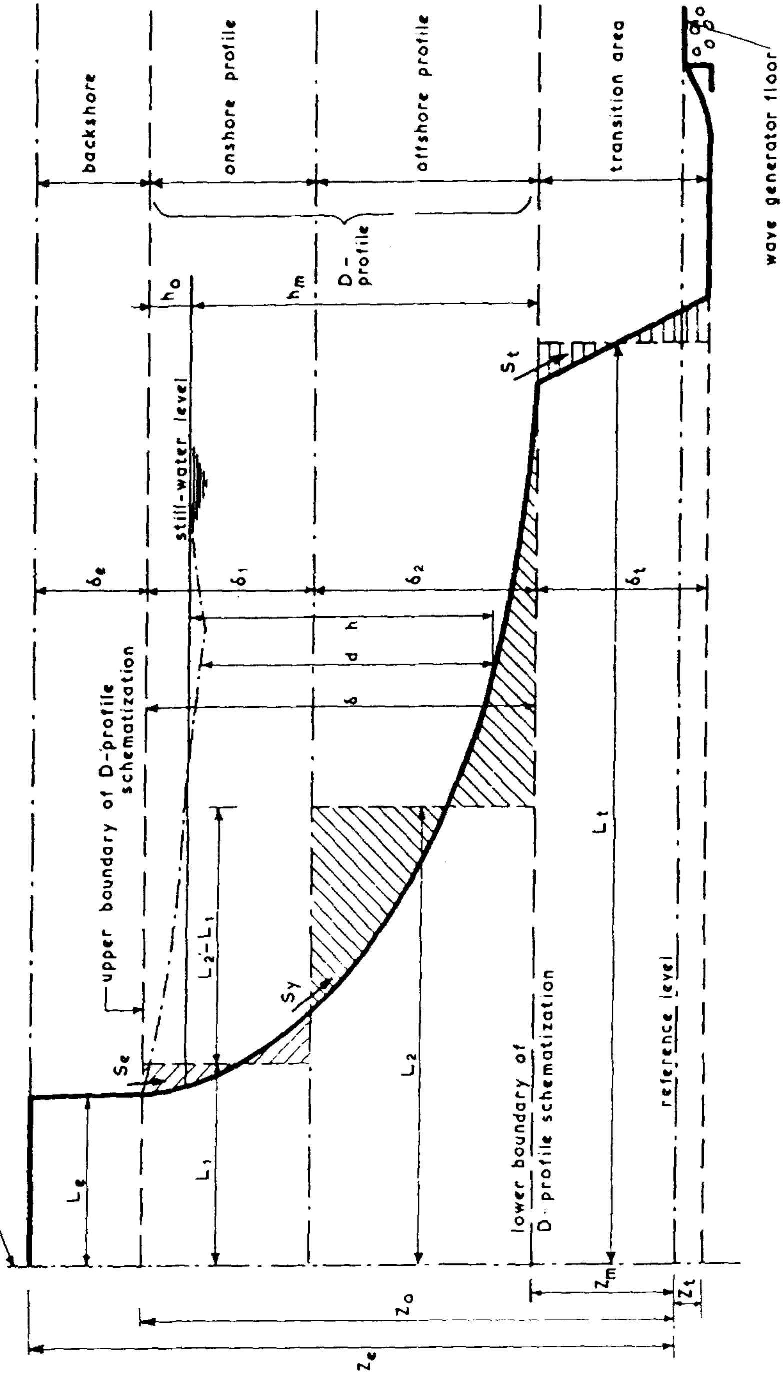

Fig. 1. Schematization of the beach profile (Swart, 1974a).

(Fig. 1) and developed this concept further into the first time-dependent cross-shore model and showed that

$$S_y = s_y(W-(L_2-L_1)_t) \tag{2.1}$$

where S_y = time-dependent cross-shore sediment transport rate, $(L_2-L_1)_t$ = value of (L_2-L_1) at time t (L_1 and L_2 are defined in Fig. 1), W = equilibrium value of (L_2-L_1), and s_y = a coastal constant for a specific set of boundary conditions.

This in effect means that the transport rate is related to how far the existing profile differs from the equilibrium profile. Swart proved with experiments that an equilibrium profile is attained. He derived analytical expressions for $(L_2-L_1)_t$ based on the conservation of mass.

The computational method basically consists of calculating the following:

- The limits of the three zones (backshore, developing profile and transition area).
- The schematised lengths L_e, (L_2-L_1) and L_t (Fig. 1) in an arbitrary beach profile (subscripts e and t denote "backshore" and "transition area" respectively).
- The geometry of the equilibrium profile W_r, that is W at the still-water level, the ratio of W at all the other levels in the developing profile to W_r, W_e and W_t, the equilibrium lengths for the backshore and the transition area respectively.
- The coastal constants s_y, s_e and s_t for perpendicular and oblique wave attack. This computation also allows for an increase in bed shear stress due to combined wave and current action by using the Bijker (1967) approach, thus accounting in a sense for 3-dimensional effects.
- Cross-shore sediment transport rate at time t.
- Total sediment transport rate up to time t.
- Time-dependent profiles; smoothing of the acquired profile is carried out.

Strictly speaking, the equation yielding the lower limit of the developing profile, is only applicable for median grain sizes less than 0.5 mm.

By assuming that the same fraction of the total transport at each location on the profile would have occurred at that location at any given time, Swart (1976, 1986) simplified his procedure. Further adjustments include the use of the root-mean-square wave height for prototype applications and a reduction in the bed slope of the transition area which is based on the method by Eagleson et al. (1963). This reduction was found to be necessary based on comparisons between computed slopes and the ones measured at Oranjemund, Namibia (Swart, 1986).

Calibration is normally achieved by a scaling factor by which the equilibrium distance W is multiplied. Typical values of this factor lie between 0.8 and 1.2.

Data

The original derivation is based on model tests of beach erosion (no accretion cases) caused by regular waves including small-scale (Swart, 1974a) and full-scale (Saville, 1957) simulations.

The ranges of these data are:

0.07 m < deep-water wave height (H_o) < 1.71 m

1.04 s < wave period (T) < 11.3 s

0.10 mm < median grain size (D_{50}) < 0.23 mm

Prototype applications were done on the North-Holland coast (storm of February 1953 with $H_o = 4.5$ m and $T = 10$ s; Swart, 1974a), at Scripps Beach, Santa Barbara and at Virginia Beach (Seymour and Castel, 1988; Swain and Houston, 1984) on the Gold Coast of Australia with $H < 5.6$ m and $T < 15$ s (Swart, 1986) and extensively around the South African and Namibian coasts (storms with $H_o \leq 6.0$ m and measured coastline retreat of up to 31 m; Swart, 1986). Usually good results were achieved except for the barred Virginia beach. Intensive calibration was, however, sometimes required.

2.3. *The Dally and Dean model (Dally, 1980; Dally and Dean, 1984, 1987)*

Theory

The model is based on net time-averaged flux of suspended sediment (S_{ys}) past a section in the nearshore zone given by:

$$S_{ys} = \int_{\text{bottom}}^{\text{water level}} u(z)C(z)\mathrm{d}z \tag{3.1}$$

where $u(z)$ = average horizontal velocity at level z, and $C(z)$ = suspended sediment concentration at level z.

The fluid flow regime is divided into an upper layer where only mean flow is considered and a lower layer where mean flow and orbital velocity are taken into account. The interface between the layers is determined by the distance that the assumed uniform sediment will fall in one wave period.

Linear wave theory is used to predict orbital velocities and stream function wave theory has been applied by Dally (1980) to obtain second-order mean flow velocities. The cross-shore wave heights are obtained from the breaker decay model of Dally (1980).

The concentration profile is exponential and based on the 1-dimensional solution of the diffusion equation for unidirectional flow. The shear velocity used in the solution is, however, assumed to be the sum of the shear velocities due to wave-induced bottom shear and breaking-induced turbulence.

The continuity equation is then solved in an explicit finite difference scheme. However, to prevent numerical instability just outside the surf zone, empirical transport spreading (smoothing) is applied.

Data

Limited runs were conducted to illustrate the effect of changes in wave and sediment characteristics and water levels on the nature of predicted beach profiles, including bars. A comparison, with one test in a large wave tank from Saville (1957), yielded a similar bar and trough pattern. Quantitatively, the beach profile shapes were, however, not very favourable.

2.4. The Bailard model (Bailard and Inman, 1981; Bailard, 1981, 1982a, 1982b, 1985, 1991; Stive, 1986; Nairn, 1990, 1991)

Theory

The Bailard model was developed on the basis of concepts derived by Bagnold (1963, 1966) for sediment transport in streams. Bagnold compared the stream to a machine having specific efficiencies in transporting bedload and suspended load. In essence, he assumed that the instantaneous sediment transport rate is directly proportional and reacts immediately to the instantaneous energy dissipation rate per unit bed area. Bailard and Inman (1979) and Bailard (1981, 1982a,b) derived general expressions for time-averaged longshore and cross-shore transport rates. In doing so, he assumed that the total velocity vector, u_t, is expressed in terms of the steady and time-varying vectors. This resulted in determining velocity moments by using a binomial expansion and truncating it where errors are less than 10%. Despite this, a rather complicated formula for the local cross-shore transport rate was derived. Bailard (1982a, 1985) analysed velocity measurements from the Nearshore Sediment Transport Study (NSTS) at Torrey Pines and Leadbetter Beaches (Gable, 1979, 1980) and averaged the velocity moments both temporally and spatially to obtain mean surf-zone values as a function of the significant wave height and beach slope. Bailard's (1985) formula for the time-averaged local (immersed weight) cross-shore transport (i_y) is:

$$i_y=\rho c_f u_m^3\frac{\epsilon_B}{\tan\phi}\left[\psi_1+1.5\delta_u-\frac{\tan\alpha}{\tan\phi}u_3^*\right]+\rho c_f u_m^4\frac{\epsilon_s}{w}\left[\psi_2+4u_3^*\delta_u-\epsilon_s\frac{u_m}{w}\tan\alpha u_5^*\right] \tag{4.1}$$

where ρ = density of sea water, c_f = drag coefficient of the bed, ϵ_B = bedload efficiency factor = 0.13, ϕ = internal angle of friction for the sediment, $\tan\alpha$ = bed slope, ϵ_s = suspended load efficiency factor = 0.032, w = fall velocity of the sediment, δ_u = steady onshore current velocity, u_m = orbital velocity magnitude, u_3^* and u_5^* are total velocity moments, and ψ_1 and ψ_2 are skewness parameters.

Bailard and Inman (1981) derived different expressions for the special cases of weak and strong longshore currents and for near-normal wave incident angles.

Other assumptions in the derivation include:

- Cross-shore transport is valid for a plane bed and sheet flow only. This is because phase-dependent sediment suspension due to vortex generation over ripples is not taken into account.
- A planar beach, that is, a beach having a constant beach slope.
- The drag coefficient is independent of u_t (u_t = total velocity vector) and constant across the surf zone.
- No incipient motion criterion.
- Ignore the effect of wave breaking turbulence.
- ϵ_s and ϵ_B are constants.
- $\tan\alpha/\tan\phi \ll 1$.
- The bottom shear stress varies according to $\rho c_f|u_t|u_t$ instantaneously with no phase difference.

Stive (1986) applied Bailard's concepts and determined the required velocity moments by applying the approaches by Stive and Wind (1986) and De Vriend and Stive (1987). He used the Battjes and Janssen (1978) method of wave height decay in the surf zone. In addition, Stive ignored cross-shore transport in the swash zone. Further work was done by Roelvink and Stive (1989) on a variety of cross-shore flows and their roles in generating bars on a beach. Roelvink (1991, 1993) developed an accurate time-dependent hydrodynamic model which describes the propagation and dissipation of wave groups over an arbitrary profile and the long waves generated by these wave groups. Although the transport formulation by Bailard is still used (and it is therefore classified as a Bailard-type model here) this model (called UNIBEST-TC) differs in many respects from the original Bailard model.

Nairn (1990) refined the Bailard model by assuming that the mean velocity applied to the bed load should be that determined for the boundary layer and similarly, a velocity at some height above the boundary layer should be used for the suspended load. In addition, more terms were retained in the velocity expansion used to determine the velocity moments and a new friction factor for sheet flow was introduced.

Data

Bailard (1985) stated that although his model remains unverified, it exhibits a qualitative behaviour which corresponds to field data of beach profiles. He used the NSTS data sets collected at Torrey Pines and Leadbetter beaches (Gable, 1979, 1980). The range of the data is as follows:

0.2 m < significant wave height (H_s) < 2.0 m

5 s < zero-crossing wave period (T_z) < 20 s

$0.02 < \tan\alpha < 0.038$

0.17 mm < D_{50} < 0.22 mm

Stive (1986) used three field and laboratory data sets to verify his version of the model. The field data collected at Voordelta, Netherlands, consisted of the migration of a delta. Both a plane bed and a bar were modelled in the laboratory. The range of his data is:

0.081 m < incident root-mean-square wave height (H_{rms}) < 1.50 m

2 s < peak energy wave period T_p < 5.9 s

0.090 mm < D_{50} < 0.225 mm

Nairn (1990, 1991) conducted extensive calibration tests, not only on the prediction of the beach profile but also on the wave transformation through the surf zone, the hydrodynamics (including mean wave-induced and time-varying flows) and the effect of long waves. The **erosion events** were from tests in the Grossen Wellenkanal (Big Wave Flume) (Dette and Uliczka, 1986, 1987), at Oranjemund in Namibia (Möller and Swart, 1988) and during hurricane Eloise on the Florida coast (Kriebel and Dean, 1984, 1985). The data ranges are:

$0.99\ \text{m} < H_{\text{rms}} < 2.6\ \text{m}$

$6\ \text{s} < T_{\text{p}} < 14\ \text{s}$

$21.6\ \text{m}^3/\text{m} <$ cross-shore transport rate above mean sea level during the storm

$< 123\ \text{m}^3/\text{m}$

$0\ \text{m} <$ storm surge $< 3.2\ \text{m}$

$4.2\ \text{h} <$ duration $< 23\ \text{h}$

$0.22\ \text{mm} < D_{50} < 0.51\ \text{mm}$

Accretion events (onshore transport) tested, consisted of both (1) small-scale laboratory results and (2) prototype-scale laboratory and field studies: Data from the Grossen Wellenkanal by Dette, Uliczka and Nairn (Nairn, 1990), from the Duck 82 field study (Jaffe et al., 1984) and from Naka Beach, Japan (Sunamura and Takeda, 1984). The range of the data for the above-mentioned category (2) is:

$0.25\ \text{m} < H_{\text{rms}} < 1.7\ \text{m}$

$3\ \text{s} < T_{\text{p}} < 14\ \text{s}$

$0.22\ \text{mm} < D_{50}$

$< 0.3\ \text{mm}$ (= schematised values; D_{50} varies between 0.18 mm and 2 mm)

$24\ \text{h} <$ duration $< 72\ \text{h}$

Nairn (1990) also investigated **bar maintenance and the role of infragravity waves**. The test cases were field experiments conducted at Wendake Beach, Lake Huron (Greenwood and Sherman, 1984), Duck 85 (e.g. Sallenger and Howd, 1989) and Duck 82 (e.g. Sallenger et al., 1985 and Sallenger and Howd, 1989). The ranges of the data are as follows:

$0.6\ \text{m} < H_{\text{rms}} < 1.5\ \text{m}$

$3\ \text{s} < T_{\text{p}} < 9\ \text{s}$

$14\ \text{h} <$ duration $< 60\ \text{h}$

$0.6\ \text{mm} < D_{50} < 0.3\ \text{mm}$

2.5. The Kriebel and Dean model (Kriebel and Dean, 1984, 1985; Kriebel, 1986, 1990, 1991)

Theory

Kriebel and Dean (1984, 1985) assumed that a beach profile will always move towards its equilibrium profile which, based on work by Bruun (1954) and Dean (1977), is given by

$$h = Ax^{0.67} \tag{5.1}$$

where h = depth at a distance x from the shoreline, and A = scale parameter dependent on the grain size.

The cross-shore sediment transport rate (S_y) is related to the difference between the actual and the equilibrium values of the energy dissipation per unit volume (D and D_* respectively):

$$S_y = K(D - D^*) \tag{5.2}$$

where K = transport rate parameter found during calibration tests to be 8.75×10^{-6} m^4/N (Kriebel, 1990, 1991).

Using shallow-water wave theory and assuming spilling breakers,

$$D = \frac{5}{16} \rho g^{15} \gamma^2 h^{0.5} \frac{\mathrm{d}h}{\mathrm{d}x} \tag{5.3}$$

with γ = breaker index = 0.78, ρ = density of sea water, and g = gravitational acceleration.

In order to obtain time-dependent profile evolution, the continuity of sand in the cross-shore direction is solved by means of an implicit finite difference method.

Data

Calibration and testing of the model were done by using data collected in large wave tanks (Saville, 1957; Vellinga, 1986; Dette and Uliczka, 1987) and prototype data collected after hurricane Eloise in Bay and Walton Counties, Florida, Point Pleasant, New Jersey, Gulf Shores, Alabama and El Segundo, California (Kriebel, 1986, 1991).

The ranges of the data are as follows:

1.06 m < maximum breaking H_{rms} (H_{brms}) < 7.14 m

5.4 s < T_p < 11.33 s

0 m < maximum water level < 3.2 m

0.22 mm < median grain size < 0.45 mm

0 m^3/m < cross-shore transport rate during the storm < 100 m^3/m

0 h < storm duration < 92 h

2.6. The Shibayama model (Shibayama, 1984; Shibayama and Horikawa, 1982, 1985; Shibayama et al., 1988)

Theory

Wave transformation and the velocity field are calculated from the incident wave conditions and bottom profile. Mean water-level change due to radiation stress (set-up and set-down) is also considered.

In order to calculate the sediment transport rate, the time history of the near bottom velocity is required. Therefore, a method to calculate the wave height and near bottom velocity from deep to shallow water was developed.

The wave transformation is based on the energy flux method (Isobe and Horikawa, 1981). The wave breaking condition employed, is that of Goda (1975). Wave deformation in the surf zone is based on the wave amplitude, the mean water surface elevation, the angular wave frequency and the eddy viscosity (as proposed by Mizuguchi, 1980). The near-bottom velocity is then determined by means of the approximate method of Koyama and Iwata (1983) using cnoidal wave theory. This model does not include return flow or wave reflection.

The sediment transport rate is calculated by using the formulas developed by Shibayama and Horikawa (1982). They classified the sediment transport into three major types and combinations of these types through consideration of dominance of either bed load or suspended load, and of the sediment transport direction. The types are classified by two parameters, namely, the Shields parameter (which is the non-dimensional bed shear stress) and the ratio of maximum fluid velocity to sediment particle fall velocity.

In order to divide the subtypes, the ratio of water particle orbital diameter to ripple wave length was used. Because of wave asymmetry due to shoaling over a sloping beach, the maximum water particle velocity in the onshore-direction is greater than that in the offshore-direction. Since the maximum water particle velocities in the onshore and offshore directions can be calculated individually, the onshore-directed sediment transport and the offshore-directed sediment transport can be calculated separately. The net sediment transport rate is obtained as the sum of the onshore-directed sediment transport rate and the offshore-directed sediment transport rate.

The repose angle of a sand bed under water, β was determined. If during the calculation the local bottom slope became greater than $\tan\beta$, the bottom surface was assumed to become unstable and adjust to $\tan\beta$.

In order to employ a finite difference scheme, the study area, from the maximum run-up point to the depth of initial movement, was divided into intervals. At each point, the wave height, near-bottom velocity and sediment transport rate were computed starting from the offshore input. The mean surface elevation was calculated by iteration with the initial value given by the backward Euler method. The bottom elevation and thus the beach profile changes were computed by solving the continuity equation for bed materials, in finite difference form, by using the forward Euler method. The time step Δt was selected to be the wave period because the transport formula is based on the time duration of a wave period. The grid interval Δx was selected to be of the order of the ripple wavelength because the transport formula describes sediment movement over one ripple.

The agreement with the laboratory results was reasonably good in estimating overall patterns. The reasons for the disagreement of small scale structures were also considered. According to Shibayama and Horikawa (1985) the following two effects were considered to be most important and should be included in future development of the model:

- the effect of the large scale vortex created by wave breaking, and
- the complex nature of the velocity field in the surf zone such as the return flow, wave reflection and small-scale vortex effects.

Shibayama et al. (1988) further investigated sediment transport due to breaking waves. In particular they focused on sand suspension due to large-scale vortices and the accompanying transport in the turbulent flow. Small-scale laboratory experiments were performed and simulation results were obtained. The authors themselves state that the results were not satisfactory, but judged them to be promising for further development. They also concluded that more precise modelling of the sand pick-up rate and the velocity field were required.

Data

Calibration and testing of the model were done by using data collected in a small wave flume.

The ranges of the data are as follows:

$1.5\ \mathrm{cm} < H_o < 20.0\ \mathrm{cm}$

$1.0 < T_p < 2.0\ \mathrm{s}$

$0.2\ \mathrm{mm} <$ median grain size $< 0.7\ \mathrm{mm}$

2.7. The Watanabe model (Watanabe, 1985, 1988; Watanabe and Dibajnia, 1988; Watanabe et al., 1991)

Theory

The total model consists of three submodels namely, for calculating (1) waves, (2) nearshore currents, and (3) sediment transport and beach change. As a first step, the initial beach topography and the geometry of the structures for the study area are given as input data, thereafter the nearshore wave field is computed for the prescribed incident wave conditions and tidal (water) level. The computed wave field is then used to estimate the spatial distributions of radiation stresses and near-bottom orbital velocities.

Computation of the nearshore currents follows that of the wave field. The nearshore current velocity and the mean water level are calculated by numerically integrating the mean momentum and continuity equations for the fluid motion.

The nearshore wave and current fields thus computed, are utilised in the submodel of sediment transport and beach change. The distributions of sediment transport rates produced by the waves and currents are determined and then the change of local bottom elevation is computed by solving the equation of sediment mass conservation. The sediment transport rate q at each local point is calculated from local wave–current conditions by the formulas below. The transport rate q is considered to be due to mean currents q_c and to waves q_w. These formulas are based on the wave power concept and assume that the sediments set in motion by the excess shear stress under combined wave–current action, are transported by both mean currents and wave motion into the respective directions. Thus the transport rate q_w is in the direction of wave propagation and q_c is in the direction of the mean current.

$$q_c = A_c(\tau - \tau_c)U/\rho g \tag{7.1}$$

$$q_w = A_w F_D(\tau - \tau_c)u_b/\rho g \tag{7.2}$$

where A_c and A_w are non-dimensional coefficients, U the current velocity vector, u_b the maximum near-bottom orbital velocity vector, F_D a direction function for wave-induced net transport, τ the maximum bottom shear stress in a wave–current coexistent system, τ_c the critical shear stress for the onset of the general movement, ρ the water density, and g the gravitational acceleration.

The bottom shear stress for a wave–current coexistent system is evaluated by the friction law proposed by Tanaka and Shuto (1981). The critical shear stress is calculated from the critical value of the Shields parameter, which is 0.11 for fine sands and 0.06 for coarse sands depending on the grain size and the thickness of the boundary layer (Watanabe et al., 1980).

The criterion for the local net transport direction has been derived by assuming a typical cross-shore distribution of the net sand transport rate on a transitional type beach between eroding and accreting type beaches; the direction is shoreward in the zone from the shoreline to the null-point (where the net rate of transport is zero) and the direction is seaward in the zone from the null-point to the location for the onset of the general movement. Watanabe et al. (1986) have introduced a parameter (Π), which expresses the effects of the relative flow intensity and the asymmetry of the orbital motion.

$$\Pi = \Psi^{1} \frac{h}{L_o}$$

The parameter Ψ^1 represents the relative intensity of wave-induced orbital motion. L_o is the deepwater wavelength and h is the water depth.

The transport direction function F_D is defined as

$$F_D = \tanh k_d \frac{\Pi_c - \Pi}{\Pi_c} \tag{7.3}$$

in which Π_c is a critical value of Π at the null-point and k_d is a coefficient which controls the degree of change in the cross-shore transport rate around the null-point. Positive values of F_D correspond to the net transport in the direction of wave propagation (onshore) and negative values to the opposite direction.

The assumed distribution of the net cross-shore transport rate was found to be applicable to beach profile changes in small-scale wave flume experiments. However, the distribution is not always consistent with the results of prototype-scale experiments.

The numerical model presented by Watanabe et al. (1986) was modified in order to improve its accuracy in predicting those phenomena under conditions of regular incident waves with no structures. The then newly derived sediment transport rate formula incorporated the effect of breaker-induced turbulence on the sediment transport in the surf zone (Watanabe and Dibajnia, 1988).

Watanabe and Dibajnia stated that the modified model needed essential improvement for the computation of sediment transport rates in the swash zone. The effect of undertow on the transport rates and general functional forms of the involved coefficients should also be studied. In the formula there are various empirical coefficients to be determined through comparison of computed beach changes with measurements at each site. In addition, the criterion for the direction of wave-induced net sediment transport was derived mainly from the results of laboratory experiments and its applicability to the field had not been thoroughly investigated.

Consequently, therefore, Watanabe et al. (1991) investigated the direction of net sediment transport due to waves in relation to sediment transport modes and they evaluated appropriate values of the coefficients in the sediment transport rate formula on the basis of field data.

They found that, because sheet flow movement can easily occur under field conditions, the critical conditions, for the onset of onshore movement under sheet flow condition were significant in judging the direction of the net rate as well as that of offshore movement due to formation of sand ripples. By setting the values of A_c and A_w to be approximately 2.0 and

0.2 respectively, they could estimate the local sediment transport rate in the field with the required accuracy for practical use, irrespective of the sediment transport modes. However, the transport due to undertow and the effect of turbulence due to wave breaking on the bottom shear stress were also found to be significant in the surf zone, and further investigation of these effects is needed in order to predict short-term beach profile changes on a natural beach.

Data

Calibration and testing of the model were done by using data collected in the laboratory and prototype data (Watanabe et al., 1991) (Table 1).

The ranges of the data are as follows:

$0.01\ \mathrm{m} < H_o$ or $H_s < 2.0\ \mathrm{m}$

$0.83\ \mathrm{s} < T_p < 10.0\ \mathrm{s}$

$0.2\ \mathrm{m} <$ maximum water depth $< 10.0\ \mathrm{m}$

$0.2\ \mathrm{mm} <$ median grain size $< 0.7\ \mathrm{mm}$

$0\ \mathrm{m^3/m}$ per year $<$ cross-shore transport rate $< 0.005\ \mathrm{m^3/m}$ per year

These ranges are not complete as all the data ranges could not be determined from the literature available to the authors.

2.8. *The Nishimura and Sunamura model (Nishimura and Sunamura, 1986)*

Theory

The local rate of net on/offshore sediment transport is empirically formulated as a function of the Ursell number and Hallermeier parameter (defined in Eq. 8.1). A sub-model of two-dimensional wave transformation includes the wave shoaling, breaking and damping

Table 1
Calibration and testing

Reference	Experimental conditions or verification data
Watanabe (1982)	Regular wave (small wave flume)
Kajima et al. (1982)	Regular wave (large wave flume)
Mimura et al. (1986)	Irregular wave (small wave flume)
Sato and Horikawa (1986)	Asymmetric regular oscillation
	Asymmetric irregular oscillation (2-D ripples)
	Asymmetric irregular oscillation (3-D ripples)
Watanabe et al. (1986)	Regular wave (small wave basin)
Maruyama et al. (1982)	Beach evolution around a harbour
Shimizu et al. (1990)	Beach evolution around a harbour entrance
Watanabe et al. (1991)	Deposition rate in a fishing harbour
	Fluorescent sand tracer experiments in the surf zone
	Deposition rate in a power station basin

in the surf zone. It is combined with another sub-model of beach profile change for the analysis of profile response to the incident waves. The validity of the model was examined through observation of profile changes in ordinary and prototype-scale flumes.

The basic inputs required are the incident waves, bed material and initial profile. The process of profile change is simulated through repetitive calculation of two-dimensional wave transformation and net sediment transport due to waves. In essence, it is a two-dimensional model.

The numerical simulation of wave transformation in a shallow region is based on the law of wave energy conservation. In the determination of the variation of wave energy, a steady field of unidirectional waves is assumed. The effects of bottom friction and internal turbulence produced by breaking are evaluated separately (as formulated by Izumiya and Horikawa, 1984). The effect of turbulence in the surf zone is represented in their formulation by a coefficient. The breaking point is determined by use of the breaker indices given by Goda (1970).

The radiation stress accompanying monochromatic waves is evaluated from the energy density. The surface elevation due to wave setup is given through a numerical integration of the equation which balances the gradient of the stress with the gravitational component of water surface slope.

The Ursell number U_γ represents the skewness of the water particle velocity profile and the Hallermeier (1982) parameter θ (dimensionless sediment parameter) indicates the intensity of sediment movement, where

$$U_\gamma = HL^2/d^3 \text{ and } \theta = (a_o\omega)^2/\gamma^1 gD \tag{8.1}$$

in which L = the wavelength, d = the water depth including wave set-up, a_o = the near-bottom orbital diameter, ω = the wave angular frequency of waves, D = the grain size of the sediment, γ^1 = the specific gravity of immersed sediment.

A formula derived (experimentally) from the relationship between the Ursell number and the Hallermeier parameter gives the rate of net sediment transport (Q) as a function of local wave height and water depth, but does not include any factor to represent the effect of bottom slope. As a consequence, a model based simply on the above formula may allow the generation of unrealistic local irregularities in the bottom topography. The model is also incapable of describing the reduction of net sediment transport which is normally observed as the beach profile approaches an equilibrium slope.

Therefore a term proportional to the excess bottom slope was introduced to give the effective sediment transport rate Q' to allow for slumping:

$$Q' = Q \pm \left(\frac{dz}{dx} - \beta^*\right) Q^* \tag{8.2}$$

where z is the bottom surface level, x is the horizontal distance and the modification is performed only when the absolute bottom slope exceeds the critical slope β_*. The scale of the modification Q_* as well as β_* are determined empirically.

Finally the bed mass continuity equation is solved (over time).

Simulation of the beach profile change is achieved by repetitive computations of the wave height, sediment transport rate and bottom surface levels.

Data

Calibration and testing of the model were done by using data collected in small and large (prototype) scale wave flumes.

The ranges of the data are as follows:

$1.8\ \mathrm{cm} < H_o < 0.85\ \mathrm{m}$

$0.85\ \mathrm{s} < T_p < 3.0\ \mathrm{s}$

$0.18\ \mathrm{mm} <$ median grain size $< 2.9\ \mathrm{mm}$

2.9. *The Steetzel model (Steetzel, 1987, 1990, 1993; Delft Hydraulics, 1990)*

Theory

Steetzel (1987, 1990, 1993) and Delft Hydraulics (1990) developed a time-dependent cross-shore transport model to simulate profile changes of unprotected and protected (partially or totally with a revetment) dunes and nearshore profiles. The basis of the model is the following simplified equation which yields the local time-averaged cross-shore transport rate:

$$S_y(x) = \int_{z=0}^{\eta_{max}} \bar{u}(z) \cdot \bar{C}(z) \mathrm{d}z \tag{9.1}$$

in which $S_y(x)$ = cross-shore sediment transport rate at a distance x, $\bar{u}$ = time-averaged cross-shore current velocity, $\bar{C}$ = time-averaged sediment concentration, z = vertical co-ordinate, and η_{max} = maximum instantaneous water level.

Steetzel (1987) used different formulations for velocity and sediment concentration compared with his latest (1990) version. Only the formulations by Steetzel (1990, 1993) will subsequently be discussed.

The suspended sediment concentration is found from the wave-averaged diffusion equation after assuming a linear distribution of the diffusion coefficient $\epsilon(z)$ above the bed:

$$\epsilon(z) = \epsilon_o + \mu z \tag{9.2}$$

in which ϵ_o = reference mixing (diffusion) coefficient (at the bed), and μ = vertical mixing gradient.

The reference concentration at the bed (C_o) was assumed to be related to the kind of breaking (F_k) and the dissipation of the turbulent kinetic energy (Diss). Furthermore, an exponential downward decrease in turbulence level and a characteristic penetration depth below the mean water level of $0.5H_{rms}$ (H_{rms} = local root-mean-square wave height) are supposed.

Following Stive and Wind (1986), a constant vertical gradient in the time-mean shear stress is supposed below the trough level. In addition, the assumption that the diffusion (mixing) coefficient is equal to the sediment diffusion coefficient is made in order to derive the mean velocity profile. The shear stress and the mass flux at trough level are calculated by relations given by De Vriend and Stive (1987).

The cross-shore transport rate is calculated in two parts, that is, below and above the mean trough level. Steetzel (1990, 1993) and Delft Hydraulics (1990) integrates Eq. (9.1) below the trough level by using the derived relations for u and C. Above the trough level, C is assumed to be constant and given by the concentration at the mean water level.

The continuity equation is then solved to obtain bedlevel changes.

The breaker decay model of Battjes and Janssen (1978) is used to provide local wave input.

Data

Calibration of the model was carried out by first verifying the prediction of the different variables, for example, the velocity, concentration and the transport rates. About 150 simultaneously measured velocity and concentration profiles, from small- and large-scale model tests (from Delft Hydraulics, 1987, and presumably also Vellinga, 1986) were used. The diffusion coefficient $\epsilon(z)$ was determined from 68 data points contained in Delft Hydraulics (1987). Verification of the transport rate with 22 small-scale model results yielded an accuracy of $\pm 20\%$ for most of the points.

The model as a whole was then tested by comparing the predicted results (dune erosion quantities) with the response from 43 model tests and three prototype data sets. The latter data sets are the 1953 storm on the Dutch coast (Vellinga, 1986), erosion by hurricane Eloise on the USA coast (see e.g. Vellinga, 1986) and an extreme erosion event at Oranjemund, Namibia (Möller and Swart, 1988). Good results were achieved. The original model results (Steetzel, 1987) compared well with the erosion of a protected dune during two small-scale tests. Reasonable agreement was also found between measured and computed profiles where revetments were present; that is, in the version of the model reviewed here (Steetzel, 1993). No special calibration was done to improve predictions for revetment cases. Both small scale and prototype size laboratory tests were used for this comparison. For both the cases of high and low (including hidden) revetments, six tests each were employed.

2.10. The Larson and Kraus model (SBEACH) (Larson, 1988; Larson et al., 1988, 1990a; Larson and Kraus, 1989; Kraus et al., 1991; Kriebel et al., 1991)

Theory

A deterministic numerical model was developed to predict beach profile change resulting from cross-shore sand transport, focusing on the main morphologic features of bars and berms. Many of the assumptions and relationships used in development of the model are founded on observations made from large wave tank data (Saville, 1957; Kajima et al., 1983). Changes in the beach profile are assumed to be produced by breaking waves; therefore, the cross-shore transport rate is determined from the local wave, water level, and beach profile properties, and the equation describing the conservation of beach material is solved to compute profile change as a function of time.

The total model consists of separate modules. The wave module calculates the wave height H and other wave-related quantities, such as wave angle θ and wave- and wind-induced setup, at grid points on a profile line. Cell opening (wetting) and closing (drying) occurs according to the wave and wind conditions and the tide level. Assuming locally plane and parallel depth contours the wave height is determined by linear wave theory in regions of non-breaking and by the Dally et al. (1984) model in the surf zone. Either regular or random wave height can be specified. The wave module reproduced wave breaking and reformation over bar and trough profiles measured in a large wave tank (Kajima et al., 1982), as well as wave height and setup in laboratory experiments with a plane beach and oblique wave incidence. This module only requires input time series of wave height, period, and direction (and wind speed and direction if wind is important).

A general derivation of the equilibrium profile form with a sloping beach-face has been given by Larson and Kraus (1989). The profile shape is assumed to result from uniform wave energy dissipation in the surf zone; however, unlike Dean's (1977) derivation, wave breaking is not restricted to spilling breakers with a constant breaker height-to-depth ratio. Instead, wave energy dissipation per unit volume is assumed to be given by the dissipation model of Dally et al. (1984).

Based on this wave height description, the equilibrium wave energy dissipation equation may be integrated over the surf zone, with the boundary condition, the water depth (h) = 0 at the shoreline ($x=0$; $x=$ horizontal distance). The resulting form of the equilibrium beach profile is

$$x=\frac{2}{K}h+\frac{5\rho g^{3/2}\Gamma^2}{24D_e}h^{3/2} \tag{10.1}$$

where D_e is the equilibrium value of wave energy dissipation per unit volume and where Γ is the stable wave height-to-depth ratio. Eq. (10.1) can be re-written in the form

$$x=\frac{h}{m}+\left(\frac{h}{A^*}\right)^{3/2} \tag{10.2}$$

In Eq. (10.2), the profile at the shoreline (and above the shoreline by inference) has a linear slope, $m=K/2$, while the profile offshore is defined by both a linear term and a term that is similar to Dean's original solution. The A_* value is not the same as the A value used in the usual Dean equilibrium profile equation due to the additional linear slope dependence. According to Kriebel et al. (1991), no re-analysis of Moore's data (Moore, 1982) has yet been carried out to quantify the new A_* values as a function of grain size or of wave conditions.

The net cross-shore sand transport rate on a cross-shore line is then calculated. The profile is divided into four zones of different wave and transport properties, with the magnitude of the transport mainly governed by energy dissipation in the surf zone. SBEACH has been verified with data both from large wave tank experiments and the field (Larson and Kraus, 1989; Larson et al., 1990b). Bars and berms can be predicted, but at present it is restricted to cross-shore transport related to breaking waves. The wave module supplies the main input to this module, which also requires an initial profile shape and sediment grain size.

Changes in the bottom topography are obtained from the mass conservation equation after the cross-shore transport rates have been calculated at each grid point for the particular time step. By adopting the general equilibrium profile form in Eq. (10.2), the equilibrium profile response may be derived by balancing eroded and deposited sand volumes.

The equilibrium response is insensitive to the exact characteristics of the surf zone for a given surf zone width, as also suggested by the Bruun Rule (Bruun, 1962). Avalanching (slumping) is initiated in the model if the local slope exceeds 28° at any point on the grid, and the process continues until an angle of 18° is reached.

Data

A large data set comprising of 42 cases from two independent prototype-scale tank experiments was used to develop the model. The model reproduced bar formation and growth in the tank experiments, which involved monochromatic waves. It performed well in a severe test to reproduce measured bar movement in the field over five separate simulations, each encompassing events of 3- to 12-day duration. The field comparisons were severe as all profile change, wave, and water level data were used directly as measured, and only one model parameter was adjusted in calibration.

Larson et al. (1988) state that the model was subjected to extensive sensitivity analysis with input conditions and model parameters varied beyond the range of values available in the data set. Reasonable trends in predictions were always found. In addition, the model was run for several thousands of time steps; the calculated profile always reached a physically reasonable shape at earlier times and did not change in subsequent thousands of time steps. Thus the model is very stable and can be expected to reproduce the correct temporal rate of profile change.

The ranges of the data for the two sets of large wave tank experiments are as follows:

0.29 m < wave height < 1.80 m

30 s < wave period < 16.0 s

022 mm < median grain size < 0.47 mm

3.5 m < maximum water depth < 4.57 m

0.0011 < deep-water wave steepness < 0.108

The model has also been employed to simulate the profile evolution for events taken from a number of good US East Coast data sets. These were collected at Point Pleasant, NJ (Larson et al., 1990b), Ocean City, MD (Wise and Kraus, 1993), Duck, NC (Howd and Birkemeier, 1987; Lee and Birkemeier, 1993) and Myrtle and Debidue Beaches, SC (Glover and Hales, 1991).

2.11. The Danish Hydraulic Institute (DHI) model (LITCROSS) (DHI, 1991; Brøker Hedegaard et al., 1991; Skou et al., 1991)

Theory

The DHI model is a deterministic model for the morphological evolution of a coastal profile. It describes the variation of wave heights across the coastal zone, current profiles, sediment transport and the corresponding morphological evolution. The interaction between hydrodynamic conditions and bed-level evolution is reflected by the model. Firstly, the hydrodynamic conditions, wave and current profiles, across the coastal zone are calculated corresponding to the initial bathymetry. Afterwards, the cross-shore sediment transport rate and the bed-level changes are determined and then the calculation is repeated.

In more detail the model is described as follows:

The hydrodynamic model includes a description of propagation, shoaling and breaking of waves. An empirical cross-shore wave height distribution is applied.

Outside the surf zone the conditions in the near-bed wave boundary layer and its influence on the mean flow are modelled using the Fredsøe (1984) wave current boundary layer model. The effects of streaming, the non-linearity of the waves and the Lagrangian wave drift are taken into account. In the surf zone the production of turbulence and energy dissipation are dominant. These processes are described by following the method by Deigaard et al. (1986). The undertow profile which is valid for spilling breakers and the inner surf zone only, is also dependent on the shoreward flux of water in the surface roller.

When a satisfactory hydrodynamic description has been obtained, the sediment transport is determined as bed- and suspended load (including the vertical distribution). The vertical sediment diffusion equation is solved on an intra-wave period grid, for breaking and non-breaking waves and current.

The cross-shore profile changes are described by solution of the bottom sediment continuity equation, based on the sediment transport rates calculated previously. Being a time-domain model, it includes the effects of changing morphology on the wave climate and transport regime. This enables a detailed simulation of profile development towards a new equilibrium for a time-varying incident wave field.

The bed-level changes are described by the continuity equation for the sediment. The numerical solution of the continuity equation is explicit. A modified Lax–Wendroff scheme has been applied to reduce the numerical diffusion and to obtain a stable solution.

The cross-shore sediment transport model gives the relation between the local hydraulic parameters (wave height, energy dissipation etc.) and the sediment transport. The sediment transport model is, however, formulated for quasi-uniform conditions, and the actual sediment transport rate cannot be expected to adjust itself to abrupt changes in the hydraulic conditions, as for example at the point of wave breaking for regular waves, where the present models predict a large discontinuity in the energy dissipation and the radiation stress gradient.

The calculated sediment transport is therefore smoothed before being used for the morphological calculations. Firstly, the calculated transport rates are smoothed by a running average, and secondly, a response function is introduced:

$$\frac{\mathrm{d}q_{\mathrm{m}}}{\mathrm{d}x}=\frac{q_{\mathrm{s}}-q_{\mathrm{m}}}{L_{\mathrm{r}}} \tag{11.1}$$

where q_s is transport capacity as a function of local parameters smoothed by a running average and q_m is actual transport. The length scale, L_r, used for the running average and for the above given response function, is proportional with the local water depth.

Data

Simulated evolutions of a breaker bar have been compared to measurements for two cases: Saville (1957) and Dette and Uliczka (1986). Both tests were carried out in a large wave flume with regular waves. From the comparison with these measurements it appears that the model can be calibrated to simulate the evolution of a breaker bar under regular waves. However, there are still discrepancies between the calculated and measured evolutions. Further, the simulated evolution does not reach a stable profile. Brøker Hedegaard et al. (1991) discuss the influence on the calculated morphological evolution of parameters applied in the model and phenomena described by the model. They investigated the sensitivity of the morphological model to the effect of the length scale, effect of density-driven currents and gravitation on bed load or bed concentrations, effect of description of wave heights after breaking, effect of onshore transport, effect of irregular waves and effect of tidal variation on bar formation.

They conclude that the model is able to simulate the growth of a breaker bar formed by regular waves and to give indications of the modifications of the evolution due to phenomena such as irregularity of the waves and tide.

The ranges of their test data are as follows:

$0.5 \text{ m} < H < 1.68 \text{ m}$

$6.0 \text{ s} < T < 11.3 \text{ s}$

$0.2 \text{ mm} < \text{grain size} < 0.33 \text{ mm}$

The DHI model forms part of DHI's integrated modelling system for littoral processes and coastline kinetics (LITPACK).

3. Comparison of the model characteristics

In the introduction a number of minimum basic requirements for cross-shore models are given. Table 2 contains a summary of the characteristics of each model. Included are also remarks concerning the basic requirements as stated by the authors in Section 1.2, as well as the capability of the models with regard to the following aspects:

- dune slumping
- dune overwash
- revetments/seawalls or rock profiles
- long waves
- incipient motion criterion
- limit of where significant beach profile variations ends (that is, the closure depth)
- incorporation of water-level variations

In particular, the pros and cons of the theoretical basis of each cross-shore transport model are listed together with conclusions regarding the extent to which the models were verified. The latter is based on the data ranges for each model given in Section 2. Table 3 lists whether the above-mentioned aspects are accounted for by the models, with the aim of facilitating easy comparison of them. For example, from Table 3 it is easy to see which models can predict the formation of sand bars. The contents of these tables are then used to classify the models into three categories with regard to theoretical basis and the extent of their verification. The same amount and detail of information was not available for all the models. Thus although we have attempted to be consistent throughout, a uniform level of detail and sophistication in comparing the models could not be absolutely maintained for all models.

Table 2
Theoretical basis and extent of verification

Model	Sediment transport modelling approach	Theoretical basis: pros	Theoretical basis: cons	Verification data	Remarks
1. Swart	Equilibrium beach profile; geometric; empirical	• The basis (concept) of the Swart model is appealingly simple. • The method accounts for 3-dimensional effects by adjusting the bed shear for wave and current action. • Water-level variations (e.g. due to tides and storm surges) can easily be incorporated. • The model can predict both accretion and erosion. • The limit up to where significant profile variations occur is computed. • In later versions of the model (CSIR, 1988), the effect of a rock profile (or revetment) was incorporated.	• This model, being the first real mathematical model for the prediction of time-dependent cross-shore transport, is empirical (Swart, 1986). Even though it has been used extensively it should be applied with care. • The parameters used in the model have to be calculated from complicated empirical formulae. • Dune slumping, dune overwash and the effect of long waves have not been taken into account. • Bars cannot be reproduced. • Intensive calibration is sometimes necessary to predict realistic profiles. • Smoothing is applied in the model.	• It should be borne in mind that the original data consist only of erosion events caused by regular waves. • Prototype verification was done at 3 sites with reasonable results. In addition, large wave flume tests were also used. • Extreme conditions are also included in the verification.	• No incipient motion criterion is applied. • The model is commercially or otherwise available to users other than the model developer(s).
2. Dally and Dean	Suspended sediment flux ($u \cdot C$)	• The theory is not empirical and is based mainly on physical parameters. • Sand bars and both accretion and erosion can be modelled. • Water-level changes (including storm surge) can easily be handled.	• Bedload has been disregarded (Seymour, 1987; Zeidler, 1987). • For simplicity, it has been supposed that the bottom reference concentration, the dimensionless value of the stream function at the	• The model has not yet been tested extensively, especially not against	• Neither an explicit incipient motion criterion nor a closure depth is used.

			free surface and the kinematic eddy viscosity are constant across the surf zone. ● By smoothing, some empiricism has also been introduced. ● Lack of reaching equilibrium profiles even after long runs. ● The influence of long waves, dune slumping and dune overwash have not been included. ● The effect of a sea wall is not included.	prototype data. ● Comparison of predicted results with one test in a large wave flume was done with qualitative success.	
3. Bailard (and improvements by Stive, Nairn and Roelvink)	Energetics; efficiency of the water to transport sediment	● The Bailard model has a reasonably sound theoretical basis. ● The model has been improved by Stive (1986), Nairn (1990) and Roelvink (1993). ● The effect of long waves can and has been incorporated. ● Dune slumping is included in the morphological model into which the transport model is incorporated. ● The effects of auto-suspension (Bailard, 1981; Nairn, 1990) were investigated. ● A seawall can be handled and sand bars can be predicted. ● It is easy to incorporate tide and storm surges. ● The model includes wave-asymmetry transport. ● It includes the effect of the bottom slope. ● Both erosion and accretion can be simulated.	● Assumptions include the following: constant (temporal and spatial) efficiency factors (ϵ_B and ϵ_s) and a constant drag coefficient (C_f); only valid for a planar bed; ignore turbulence due to wave breaking; assumes sheet flow. ● Smoothing of the calculated bed-level changes is required in order to simulate lateral mixing and obtain realistic profile variations. ● Dune overwash has not been considered.	● The model has been tested extensively, especially by Nairn (1990). ● The data cover a large range of conditions at a number of different locations pertaining to both erosion and accretion events. ● Small-scale test results were not well produced.	● The closure depth is not explicitly calculated. ● Nairn (1990) found bad correlation between predicted and measured profile variations in small-scale tests. He accounted it to a lack of an incipient motion criterion, the lack of a threshold condition for the onset of suspended transport, and a failure to account for mechanisms in the vicinity of the swash zone. These shortcomings may, however, be less important in prototype conditions. ● The model is commercially or otherwise available to users other than the model developer(s).

(*Continued on p. 26*)

Table 2 (continued)

Model	Sediment transport modelling approach	Theoretical basis		Verification data	Remarks
		pros	cons		
4. Kriebel and Dean	Equilibrium beach profile; wave energy dissipation	• The Kriebel and Dean model is in essence a simple model. • Wave characteristics, water-level variations such as tides and storm surges and grain size are taken into account. • Reasonably extensive calibration has shown that the model is not overly sensitive to the values of the input. • Later developments include the effects of wave run-up (dune slumping), dune steepening, dune overwash, volume of material in the dune and the capability to include seawalls. • Accretion as well as erosion can be predicted by the model.	• The offshore limit of cross-shore transport on the profile is taken to be the breaker depth of each particular wave condition. It is, however, well-known that significant beach profile variations occur beyond the breaker depth (for example, Swart, 1974a and Hallermeier, 1982). • The model is not capable of predicting sand bars. • The influence of long waves is not incorporated in the model. • Assumptions were made on the applicability of shallow-water wave theory in the surf zone and the use of a breaker index of 0.78 for spilling breakers.	• Extensive calibration using prototype data has been carried out at a number of sites. • Lack of calibration of the model with respect of accretion events is a disadvantage.	• No incipient motion criterion is applied. • The model is commercially or otherwise available to users other than the model developer(s).

5. Shibayama	Shields parameter and the ratio (maximum orbital velocity at the bed)/fall velocity	• It is a numerical model based mainly on theory. It is relatively simple, but takes account of a reasonable number of parameters. • It is capable of simulating bars/troughs. • The wave and velocity field calculations can be based on either the cnoidal or linear wave theory. In the surf zone the wave field is based on an energy dissipation model. • The transport rate formula includes ripple effects (vortices). • The model includes a check on the stability of the underwater sand bed slopes (and adjusts the slopes if necessary). • Water-level changes can be incorporated in the model. • Both accretion and erosion events can be simulated.	• The model for calculation of the velocity field does not include return flows or wave reflection. • The relatively small time steps and grid intervals mean that to model a large area or long time span, extensive computing facilities are required. Furthermore, the physical processes which may dominate medium-term evolution are not included. • Further work on vortex effects (created by wave breaking) was done but the results were not satisfactory. • Shibayama et al. (1988) state that more precise modelling of the sand pick-up rate and the velocity field is required. • The model does not include dune overwash or seawall effects. • The model considers only monochromatic short waves.	• In laboratory tests, the model could simulate overall patterns reasonably well but local effects like bars were poorly simulated. • The model was tested only under small-scale laboratory conditions and comparisons with field data are especially needed.	• The closure depth is not explicitly computed. • No incipient motion criterion is used.

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Model	Sediment transport modelling approach	Theoretical basis: pros	Theoretical basis: cons	Verification data	Remarks
6. Watanabe	Wave power (excess shear stress multiplied with the mean velocity)	• The model is largely based on theory although some aspects are empirical. • It is comprehensive — takes account of many parameters/ processes. • It is capable of simulating sand bars (and structures like offshore breakwaters as part of a 3D model). • The offshore limit of cross-shore transport is not taken to be the breaker line but estimates a true limiting depth. • Dune slumping and water-level variations are taken into account. • Both erosion and accretion can be simulated.	• The model is relatively complicated. • It requires extensive calibration data (empirical coefficients). • The prediction of direction of net on/off shore movement is still problematic. This is seen as an important shortcoming especially in non-standard situations. • Transport due to undertow and the effect of turbulence due to wave breaking on the bottom shear stress need further investigation. • It has not been demonstrated to simulate shore-flooding, seawalls or medium-term beach evolution and does not approach equilibrium. • It requires extensive computing facilities; long computation time and thus is expensive to run. • The effects of grain size distribution or of grading, on the sediment transport rate are not incorporated. • Dune overwash and the effect of long waves are not incorporated in the model.	• A wide range of verification data (including field data at a number of sites) was used. • Actual cross-shore transport and predicted beach profiles need more field verification. Also, storm data are needed.	• It is incorporated in a 3D model. • The critical shear stress is used as an incipient motion criterion. • Presumably, the model is commercially or otherwise available to users other than the model developer(s).

7. Nishimura and Sunamura	Ursell and Hallermeier parameters	• Although it is a numerical model (based partly on theory), it is relatively simple. • The wave model takes the effect of bottom friction, and turbulence produced by breaking, into account separately. • The model is capable of predicting the formation and development of sand bars; this includes predicting correctly the generation of a secondary bar in the trough behind a main bar. • Dune slumping has been accounted for. • The effect of tides (and mean water-levels) is included in the model. • Erosion and accretion can be modelled.	• The local net transport rate is formulated empirically. The experimental data showed considerable scatter. • It only predicts profile changes due to waves; currents are not specifically (or separately) taken into account. • Nishimura and Sunamura (1986) state that the present model involves a net sediment transport rate formula of which the accuracy can as yet (1986) not be so high and that more study is needed to determine the value of the coefficient. • The wave model assumes a steady field of unidirectional, monochromatic short waves and does not take account of random waves or wave reflection. • The formulas include a number of empirical coefficients. • The location of sand bars is not correctly predicted. Nishimura and Sunamura say that the incorrect prediction of sand bar location and position of the peak sediment transport rate imply inadequacy of the basic assumption that the rate of net sediment transport can be evaluated solely from local wave and topographic conditions. In fact, the intense near-bottom flows, induced by plunging waves, largely contribute to the bar and trough formation. • The effects of dune overwash, shore flooding, grain-size distribution and structures (for example, seawalls) are not considered.	• The model has been tested and calibrated in prototype scale flumes. • A wide range of grain sizes was used. • It has not been tested under field conditions (especially longer wave periods and high waves).	• The boundary conditions (parameters) are not well determined (for example, the maximum off-shore depth for sediment movement). • The model does not specifically include a threshold of movement criterion.

(Continued on p. 30)

Model	Sediment transport modelling approach	Theoretical basis		Verification data	Remarks
		pros	cons		
8. Steetzel	Suspended sediment flux ($\overline{u}\cdot\overline{c}$)	• Although the Steetzel model (essentially a dune erosion model) is basically simple, it simulates the most important coastal processes to obtain short-term time-dependent profile changes. • The model incorporates water-level changes and the influences of revetments. Tests have shown that fair agreement was found between measured and computed profiles if a revetment is present. • Sand bars can be handled and predicted in an erosive setting. However, sand bar growth under accretive conditions cannot be predicted. • Overtopping and break-through of dunes are in the process of being included in later versions of the model (Steetzel and Visser, 1992).	• Processes that are presently omitted, include long wave effects. • Calibration with a number of accretion events is necessary to establish the model as sufficiently versatile to predict beach profile envelopes. • The model is not suited to predict long-term evolution.	• Elements of the model have been calibrated individually and as a whole against a comprehensive data set (small and large scale flume data). Three prototype cases in which extreme erosion occurred, were also used in the verification. Steetzel (1990) recommended further testing against a variety of prototype cases. Furthermore, all the calibration data concerns beach erosion (as the model was aimed at dune erosion). • Extreme conditions are also included in the verification.	• No incipient motion criterion is explicitly used. The same applies for the closure depth. • The model is commercially or otherwise available to users other than the model developer(s).

9. Larson and Kraus	Wave energy dissipation; equilibrium beach profile	• The model simulates the dynamics of macroscale profile change, such as the growth and movement of berms and breakpoint bars. • Model development was founded on data from large wave tank experiments. • A bimodal character of beach sediment can be accommodated in the model. • A sensitivity analysis showed that reasonable trends in predictions were always found and that the model is very stable and can be expected to reproduce the correct temporal rate of profile change. • Breaking waves are the sole driving force causing sand transport in the model in its present state. However, since the model operates in a general way using a transport rate combined with the material conservation equation, in principle other sand transporting mechanisms can be incorporated if their transport rate relations are available. These mechanisms can include wave reflection from the beach or from seawalls, and transport induced by long-period wave motion. • The model is economical to run. • It has performed well in test applications to simulate month-long adjustment of beach-fill involving storm and recovery wave and water-level conditions. • Boundary conditions are easily implemented. A vertical seawall, tides and dune retreat can be accommodated in the model. • Avalanching (slumping) is employed in the model.	• The model is empirically based. • Slumping is initiated if the local slope exceeds 28° at any point on the grid, and the process continues until a single angle of 18° is reached. • The present model does not specifically cater for current-induced sediment transport. • Berm processes and the bar trough are less well reproduced, also due in part to a lack of accretion data. • Transport in the breaker transition zone and in the swash zone is less well modelled. • Although profiles are adequately predicted, actual cross-shore transport needs to be verified. • Long waves and dune overwash are not incorporated in the model.	• Large wave tank data consisting of 42 cases with different incident wave conditions, median grain size, and initial beach shape were used. Model predictions were tested with field data, and reasonable agreement was found. • Field testing of the model necessitated re-calibration of the transport coefficient. • The model is mainly based and verified on erosion events and verification is needed with respect to accretion events. • The data on which the empirical net transport direction is based, showed considerable scatter.	• SBEACH is incorporated in a working 3D model (3D BEACH). • The closure depth was not calculated but based on data, an exponential decay in cross-shore transport rate was supposed. • An incipient motion criterion is employed in the model. • The model is commercially or otherwise available to users other than the model developer(s).

(Continued on p. 32)

Model	Sediment transport modelling approach	Theoretical basis: pros	Theoretical basis: cons	Verification data	Remarks
10. DHI (Litcross)	Bedload plus suspended sediment flux (includes wave- and current-borne transport on an intra-wave period scale)	• The model applies a deterministic approach which allows consideration of many, and sometimes dominating, factors which are not available to semi-empirical formulations. • It can thus handle the simulation of a complex multi-barred profile, with varying grain size distribution. • The minimum hardware requirements to run the model on a computer are not excessive. • The present model takes into account the mechanism according to which suspended sediment increases the density of the fluid which means that in case of a sloping bed there is a downhill contribution to the near-bed mean current. • Further, the model includes an adjustment to the bed concentrations and bed load in the case of a sloping bed. • The effects of irregular waves, bar formation and tides are included. • Both erosion and accretion can be modelled.	• The method by which wave heights are calculated across the profile is empirical and probably only a valid approximation for moderate slopes ($<1:30$). • The presence of wave ripples, which can give a large phase lag between the maximum of the flow velocity and the near-bed sediment transport, is not included in the cross-shore sediment transport model (1992 version). • A smoothing and a response function is applied to the calculated sediment transport. • Dune overwash and the effect of long waves and revetments have not been accounted for.	• Verification was done by modelling profile response from two cases in large wave flumes.	• Neither an incipient motion criterion nor the calculation of the closure depth has been used explicitly. • The cross-shore model forms part of an integrated modelling system for littoral processes and coastline kinetics. In essence it is a two-dimensional model although combined with a longshore transport model it is used in a quasi-three-dimensional model. • For transport in the surf zone, the model is valid for spilling breakers and the inner surf zone, but the complex flow pattern near the plunge point of plunging breakers is not described. This is also true for most of the other models. • The use of a running average and a response function is similar to the effect of accounting for dune slumping. • The model is commercially or otherwise available to users other than the model developer(s). • Effects of a bed slope are (among others) that the wave boundary layer flow becomes convergent/divergent during each wave period and that the wave profile is deformed by the shoaling. These two effects are not included in the model (1992 version). However, this is also true for most of the other models.

Table 3
Summary of model characteristics and abilities (for comparison)

Aspects	Model number and name									
	1 Sw	2 D&D	3 B	4 K&D	5 Sh	6 W	7 N&S	8 St	9 L&K	10 L
Simulates erosion and accretion	2	2	2	2	2	2	2	1	2	2
Bars (macro profile change)	0	2	2	0	2	2	2	2	2	2
Dune slumping	0	0	2	2	?	2	2	0	2	1
Dune overwash	0	0	0	2	0	0	0	2	0	0
Revetments/seawalls, rock profiles	2	0	2	2	0	0	0	2	2	0
Long waves	0	0	2	0	0	0	0	0	0	0
Incipient motion	0	0	0	0	0	2	0	0	2	0
Closure depth	2	0	0	0	0	2	0	0	1	0
Water-level variations	2	2	2	2	2	2	2	2	2	2

Key: Sw = Swart; D&D = Dally and Dean; B = Bailard; K&D = Kriebel and Dean; Sh = Shibayama; W = Watanabe; N&S = Nishimura and Sunamura; St = Steetzel; L&K = Larson and Kraus; L = Litcross (DHI). 2 = yes/can simulate this/takes account of this aspect. 1 = possibly/takes account of this aspect to some degree. 0 = no/cannot simulate this/does not take account of this aspect. ? = cannot evaluate this/not enough information available.

4. Conclusions and recommendations

Of the available, time-dependent, mathematical cross-shore transport models, 10 were chosen and evaluated with regard to their theoretical basis (re. mainly sediment transport) and the associated verification data (re. mainly morphodynamics). In doing so, it was considered how well each model meets the requirements for such models in terms of short and medium-term applications.

According to the judgement of the authors the models were classified on their **theoretical basis** into the following three groups:

1. *Best group*
   Bailard
   Steetzel
   DHI
2. *Acceptable group*
   Watanabe
   Larson and Kraus
   Nishimura and Sunamura
   Shibayama
3. *Group with a less suitable theoretical basis*
   Swart
   Dally and Dean
   Kriebel and Dean

Because there are no hard and fast rules by which to judge completely objectively the theoretical base, it is not possible to classify the models unambiguously. However, although some readers may not agree with the above evaluation, it is felt that the information in

Sections 2 and 3 leads to this classification. Furthermore, it must be kept in mind that this evaluation is based on the models as described in the references given for each model. Rather than become involved in arguments or cause a lot of wasted time we would like to see a more positive outcome. At the least we believe this paper provides some insights or some basic guidelines to those new to the field. At best we hope to stimulate fruitful discussion (e.g. on how models may be improved).

In terms of the development potential, the more theoretical models (especially LITCROSS and the Bailard-orientated models) offer relatively better possibilities to incorporate new insights into the relevant hydrodynamics than the more empirical approaches. This gives these models (theoretical) the potential of becoming more generic (i.e. more widely applicable) than the others (empirical). Some of the models are being further developed and there is considerable scope for improving the models, whereafter some of them could easily be placed in a higher group. For example, the Bailard model does not take turbulence due to wave breaking into account and smoothing is needed in a number of the abovementioned models. However, the danger also exists that by taking more and more processes and details into account, a model will become not only over sensitive for input parameters and numerical solutions, but also cumbersome to apply and/or expensive to run. It is of little use having an excellent theoretical model which is impractical to apply to engineering problems. Not surprisingly, the simpler and more empirical models, are in general more easily applied. However, predictions which may not resemble the actual beach changes are obviously also not very useful. Furthermore, some of the relatively more simple models still require extensive calibration before practical results are obtained (e.g. the Swart model). On the other hand the relatively more complex nature of the models with a more theoretical base, in general may make them impractical to apply to large areas (e.g. the Watanabe model) or longer time spans. Other physical processes may also be more important in long-term modelling.

In practice, field data are often of relatively poor quality or altogether lacking for a particular site. Normally in such cases better results will not be achieved by using theoretically ''superior'' models. Depending on what is required, those models which do not require too extensive inputs but which nevertheless give proven realistic results are of most practical use. All the models except the Swart and the Kriebel and Dean models can simulate the dynamics of macro-scale profile change (such as bars) to some degree. Overall the different parts of a model should be balanced; meaning that all parts should be equally good, as the model can only be as good as its worst part.

Models which are commercially or otherwise available to other users than the model developer, in general require more reliable and tested calculation algorithms. Thus it may be argued that this should be regarded as an advantage in the model evaluation. However, the authors have tried to objectively evaluate the models solely on the known theoretical basis and the extent of documented verification.

From the available references it could mostly not be determined whether the models approached equilibrium. Thus this aspect is not discussed in the comparison of the models. However, it is known that models which do not approach equilibrium might distort the time scale of profile evolution, even though a specific, measured profile could be well reproduced.

In evaluating the ability of models to reproduce observed profile behaviour, the amount of calibration needed should be considered. A good agreement with measured data is not enough for quality assessment; a practical model should not be overly sensitive to realistic uncertainties in the model input parameters.

To evaluate the cross-shore transport models with regard to the **extent to which they were verified** (re. mainly morphodynamics), the following criteria were used:

- the number of test cases, especially the prototype and prototype-scale tests used in the verification
- the range of the verification data
- the number of sites at which the data were collected so as to get an idea of how representative the test cases are.

The result was the following:

1. *Best group*
   Bailard
   Watanabe
   Kriebel and Dean
   Larson and Kraus
2. *Acceptable group*
   Swart
   Steetzel
3. *Group with a less suitable verification*
   DHI
   Nishimura and Sunamura
   Shibayama
   Dally and Dean

Generally speaking, data of accretionary events have not been used to calibrate cross-shore models. It is recommended that such data of prototype cases be collected for a range of parameters and used for verification. If the data ranges are examined, it is clear that few data sets (of erosion) are available where significant wave heights exceeded 2.5 m. Such data sets should be collected and properly documented. However, a number of events with high storm surges and long durations have been recorded. Among these are some good data sets from the US East Coast (Larson et al., 1990b; Wise and Kraus, 1993; Howd and Birkemeier, 1987; Lee and Birkemeier, 1993; Glover and Hales, 1991). The models may thus be tested to some extent against these field data sets.

It may be argued that the Nishimura and Sunamura, the Shibayama and the Dally and Dean models were more "developmental" in nature and less intended for practical application. Thus it can be expected that the practical validation of these models is not as extensive as for some of the other models.

Final conclusion

Presently, it is not possible to conclude without doubt which model(s) is (are) superior to the others. This is partially due to the fact that the models have not been verified against the same data sets and not all the models have been extensively validated. It is recommended that this be done. The European MAST G6 and MAST G-8 M research programme (see De Vriend, 1991) has started such testing by using experiments carried out in the large wave flume in Hannover, Germany (Roelvink and Brøker, 1993). It is important that this comparison of models continue and that a wide range of prototype data is used for this purpose. Many of the models are also undergoing constant development and improvement.

It is also very important to keep in mind that each model may be the best for a specific case (purpose) or under specific conditions. For example, the transport mechanisms under spilling or plunging waves are very different and some models may be best for spilling waves while others are better for plunging waves.

Dean (1991) discusses future aspects to be considered in cross-shore transport modelling. These are offshore bars, overwash processes, seawalls, effect of wind, the role of infragravity waves, etc. In addition, we think, it is important to be able to handle accretion, dune slumping, sand feeding and, varying sediment characteristics.

In conclusion, it can be stated that a number of promising models have been developed but that they need further verification. Depending on the application and the accuracy of the available input and the required precision of the results, a number of these models are suitable as they are.

Acknowledgements

The authors gratefully acknowledge the helpful comments by Dr. D.H. Swart and Mr. P. Huizinga, CSIR, Stellenbosch, South Africa as well as the reviewers.

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