

CHAPTER 190
DYNAMIC NUMERICAL MODELS FOR SAND WAVES
AND PIPELINE SELF-BURIAL

by

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ABSTRACT

Two independent numerical models for prediction of sand wave heights and migration rates and for calculation of pipeline/cable seabed interaction have been developed. Both models are fully dynamic and respond to detailed and comprehensive input describing environmental conditions, seabed and soil characteristics and pipe/cable data. The models are based on state-of-the-art knowledge and include theoretical developments, model tests, field data and advanced sediment transport formulations. The models have been validated and calibrated against field data of sand waves and buried pipelines.

1. SAND WAVE MODEL

1.1 Introduction

Erodible sea beds will not usually remain plane when exposed to changing environmental conditions. A variety of different bed forms may emerge and continuously modify and adjust in response to varying wave/current conditions. The bed forms and their response to environmental conditions are also highly dependent on the bed material. Dimensions of wave/current generated bed forms observed in the offshore environment vary considerably from small ripples of a few centimetres height to the large tidal banks of heights up to 50 metres and many kilometres long.

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The most important bed form in relation to marine pipelines is the offshore sand wave. Sand waves can at the same time be large enough and migrate fast enough to cause bed level changes, which are significant compared with the diameter of the pipeline. Furthermore, the sand wave is a widespread form which is observed in a large number of tidal areas but may also occur in deep water where density differences are responsible for relatively high current velocities near the bed.

1.2 Description of the model

The numerical model for sand waves is based on state-of-the-art theories for sand waves and for sediment transport in combinations of current and waves.

Basic theory

Deigaard and Fredsøe (1986) combined a theory for fully developed sand dunes in rivers by Fredsøe (1982) with a detailed model for sediment transport in current and waves developed by Fredsøe et al. (1985). The result was a calculation model for equilibrium sand waves in an offshore environment with waves and a unidirectional current perpendicular to the sand wave front.

This model was taken as a basis in the development of the present sand wave model.

The basic elements of the theory are:

- continuity equation for sediment
- constant form sand waves assumed
- sediment transport model
- description of deposition "downstream" of sand wave crest.

The equations to determine migration rate and height of equilibrium sand waves thus become (see e.g. Deigaard and Fredsøe, 1986)

$$a = - \frac{1}{1-n} \frac{dq}{dD} \quad (1)$$

$$A = \frac{q_D}{(1-n)a} \quad (2)$$

$$q_D = q_B + q_S (1 - \exp(-6A/L_S)) \quad (3)$$

The following notation is used:

- a : Sand wave migration rate
- A : Sand wave height
- q_D : The part of the sediment transport which is deposited downstream of the sand wave crest
- n : Porosity of the sea bed
- D : Water depth at sand wave crest
- q : Total sediment transport rate at sand wave crest
- q_B : Bed transport rate at sand wave crest
- q_S : Suspended transport rate at sand wave crest
- L_S : Length scale for the lag of the suspended sediment

The length scale of the lag of the suspended sediment, L_S , can be estimated from the concentration profile as:

$$L_S = \frac{U_C \cdot y_C}{w_s} \quad (4)$$

where y_C is the height of the centroid of the concentration profile above sand wave crest level, U_C is the flow velocity at the height, y_C , and w_s is the settling velocity of the suspended sediment.

A key assumption for the prediction of sand wave height is that the deposition of sediment at the front, and thus the migration rate of the sand wave, can be calculated directly on the basis of the sediment transport at the crest of the sand wave. The basis for this assumption is that flow separation occurs at the (instantaneous) sand wave front, leading to a vanishing sediment transport capacity in the separation zone. Thereby all the bed load and some of the suspended load which passes the sand wave crest will be deposited downstream of the crest. The deposition rate described by (3) assumes that the separation zone has a length of 6 times the sand wave height. Note that (3) has been slightly modified compared with the corresponding expression by Deigaard and Fredsøe (1986).

The model of Deigaard and Fredsøe (1986) has been extended to include:

- non-equilibrium sand waves
- arbitrary angles between waves and currents and sand waves (including reversing tidal currents)

- effects of short-crested sand waves
- effects of high water waves
- effects of wave generated ripples

The extensions are further described below.

Sand_wave_length

The sand wave length, λ , is in the model determined by a purely empirical relation

$$\lambda = 64A \quad (5)$$

The factor 64 was determined from field observations with significant variation in length/height ratio.

In pure steady current experiments show that the bed shear stress after a step attains a local maximum 16 times the step height downstream of the step indicating that for situations with dominant bed load transport the dune length would be 16 times the dune height. Presence of suspended load would cause the dunes to become longer but for the current and depth conditions prevailing in off-shore areas this effect would be very limited.

There is presently no satisfactory explanation of why offshore sand waves are approx. 4 times longer than dunes of the same height in rivers.

Effect_of_high_waves

Flow separation downstream of the sand wave crest is essential for the formation of sand waves. Deigaard and Fredsøe (1986) described briefly the separation suppression effect of relatively high water waves. When waves are present together with a current, the tendency towards separation of the mean current is reduced. A crude criterion for suppression of flow separation was taken to be that the oscillating part of the shear velocity, U_{fw} , associated with the wave motion is larger than 7 times the shear velocity, U_{fo} , characterizing the mean current in the wave boundary layer.

In the present model the above criterion has been applied to form a correction factor by which the equilibrium sand wave height is reduced to zero (smoothly) when U_{fw} exceeds 7 times U_{fo} . The effect of waves on flow separation has been described in more details recently by Deigaard (1990).

Non-equilibrium sand waves

The wave and current conditions are normally highly variable, and it is therefore of interest to predict the behaviour of sand waves that are not in equilibrium under the actual hydraulic conditions. The adaptation time of a sand wave will depend on the sediment transport rate and on the volumes of the equilibrium versus the actual sand waves, i.e. the amount of sediment that has to be shifted in order to form the equilibrium sand wave.

Fredsøe (1979) and (1981) presented a model for the change of dune dimensions subject to a change in hydraulic conditions. The rate of change in dune height is found assuming a given shear stress distribution evaluated on the basis of an idealized triangular dune shape. A simple expression for the rate of change of dune height is readily derived from Fredsøe's formulas

$$\frac{dA}{dt} = \frac{a}{\lambda} (A_{eq} - A) \quad (6)$$

where

- A : Actual sand wave height
- A_{eq} : Instantaneous equilibrium sand wave height
- a_{eq} : Actual sand wave migration rate
- λ : Actual sand wave length
- t : Time

The above equation (6) has been applied in the present model to calculate the transient behaviour of the sand waves.

Directions of waves and current

The Sand Wave Model has been developed to deal with arbitrary directions of both waves and currents relative to the direction of the sand wave crest. The migration rate of a sand wave is calculated in a direction perpendicular to the sand wave crest. When the direction of the total sediment transport forms an oblique angle with the sand wave crest, only the sediment transport component perpendicular to the crest is taken into account in the calculation of sand wave migration and development.

It is assumed that long-crested sand waves will not affect the direction of a current approaching at an oblique angle i.e. streamline refraction has not been taken into account in the model.

Tidal flow conditions with reversing flow direction characterize many sand wave areas. A reversing tidal current may significantly change the shape of the sand wave crest region during a tidal cycle and thereby affect the migration and development of the sand wave. No effects of the sand wave profile shape have, however, been included in the model.

Effect of short-crested sand waves

Field data show some systematic differences in sand wave morphology from one area to another. One of the most pronounced is the difference in relative crest length. The present model is not able to explain this difference, but the effect of short-crestedness on sand wave development has been included in the model in a simplified way as described in the following:

If the sand wave crest lengths are not large compared to the sand wave length then the three-dimensional structure of the bed will result in a three-dimensional flow pattern. There will be a decrease in the water discharge over the crest peaks. The decrease is partly due to the energy loss in the separated flow downstream of the crest and partly due to the decreased water depth at the crest of the sand wave.

Fredsøe (1989) calculated analytically the equilibrium height of sand waves as a function of their length/width ratio, λ/w . He assumed a double periodic bed topography, did not include wave effects and found that a short crested sand wave will have a larger equilibrium height, A_{3D} , than the height, A_{2D} , of a sand wave with infinitely long crest.

$$A_{3D} = A_{2D} (1 + (\lambda/w)^2) \quad (7)$$

Eq. (7) has been included in the present sand wave model as a correction applied to the equilibrium sand wave height.

Effect of wave generated ripples

The detailed sediment transport model originally developed by Fredsøe et al. (1985) is applied as an important submodule for the Sand Wave Model. This transport model was developed for plane sea bed conditions i.e. without wave generated ripples. This situation will occur during storm conditions which are very important in relation to

sand wave development and migration. More calm wave conditions where ripples are present are, however, also important. The sediment transport module has therefore been further developed to handle both situations without and with wave generated ripples. The modifications to the sediment transport model include descriptions of the processes involved as and are briefly outlined as follows:

- estimates of ripple dimensions as function of wave-current conditions.
- estimates of contribution to eddy viscosity due to ripples.
- estimate of mean concentration at the level of the ripple crests.
- estimate of the roughness due to the influence of ripples on the current.

The ripple dimensions, the eddy viscosity and the mean concentration are calculated from empirical formulas presented by Nielsen (1979). The ripple roughness is calculated by a formula of Raudkivi (1988).

1.3 Example of calculation

In a simulation of sand wave development the equilibrium sand wave height is first calculated i.e. the height that would exist after a long time with the same wave/current conditions. Sand waves will form for a certain range of current velocities and the height may theoretically be up to about 30 percent of the mean water depth. Wave action generally tend to increase the sand wave height (except when they totally dominate the current) and particularly when the waves travel perpendicular to the sand wave crests.

Fig. 1 shows part of the input and the output time series for a 10 days Sand Wave Model simulation. The example which is taken from a location in the Southern North Sea illustrates the effect a stormy period has on sand wave development and migration in an area dominated by tidal currents. Directions of waves and currents which are also part of the input are not shown in this figure.

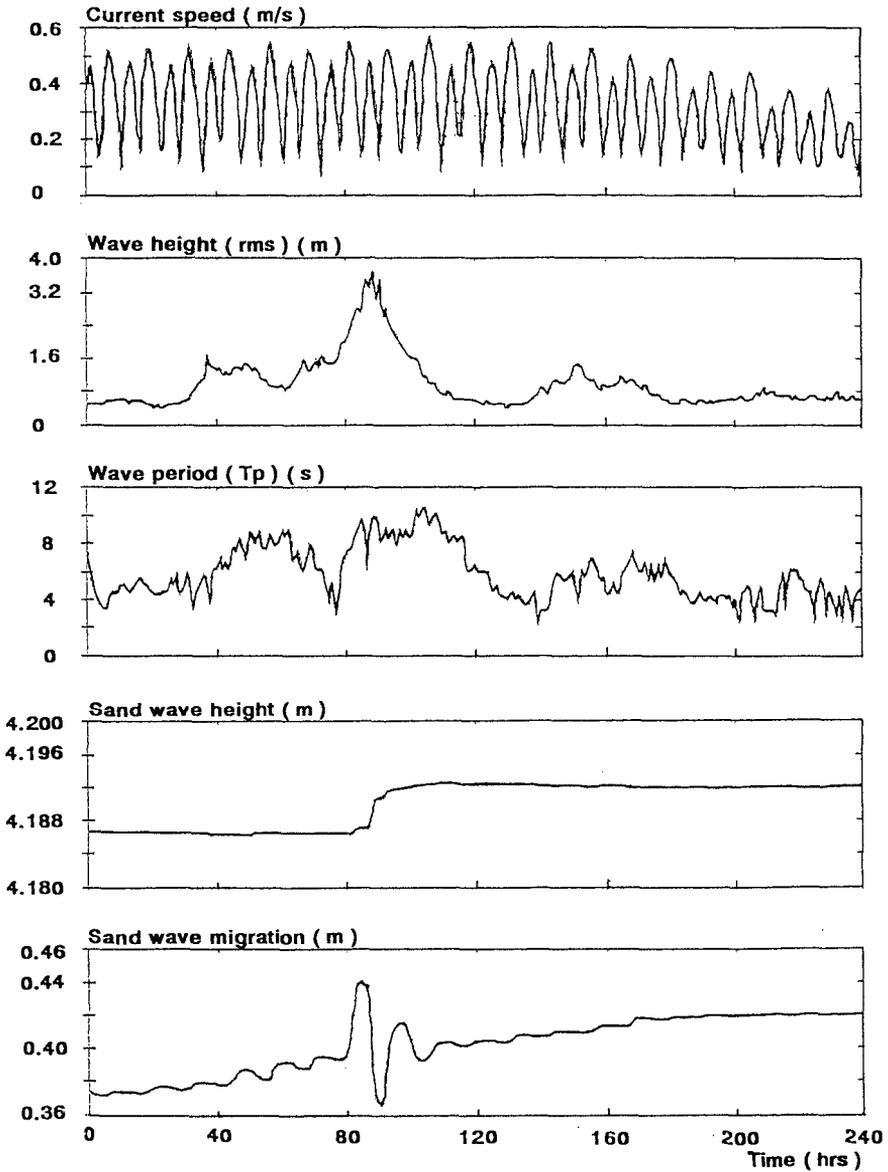


Fig. 1 Sand wave development and migration during a 10 days period with a storm. Example from Southern North Sea. Water depth: 35 m, Sediment: $d_{50}=0.35$ mm.

2. SELF-BURIAL MODEL

2.1 Introduction

Self-burial of submarine pipelines has been subject to study since 1978. At that time, a sudden, and unexpected, self-burial of a just-laid pipeline triggered extensive research programs.

The Dutch authorities almost immediately recognized the potential of self-burial as a cost-effective burial method and approved a large number of pipelines to become self-buried. As result of this attitude, significant experience with self-burial of submarine pipelines has been gained. Fig. 2 shows the self-buried pipelines in the Dutch sector. However, a reliable method to predict the self-burial of submarine pipelines did not yet exist.

The aim of the present development has been to achieve a model which is able to predict the self-burial behaviour of submarine pipelines as a function of environmental conditions (waves and currents), seabed and sediment characteristics and pipeline dimensions.

2.2 Description of the model

The concept of the Self-Burial Model is based on the observation of two dominant erosion processes under and around the pipeline: tunnel erosion and leeside erosion.

Tunnel erosion is a short term process with a time scale of the order of hours or days. After initiation of scour through seabed irregularities or vigorous water movements, a tunnel will be eroded under the pipeline.

Leeside erosion is an erosion process with a time scale of the order of weeks or months. It affects a much larger area of the seabed than tunnel erosion. A pipe protruding from the seabed disturbs the flow causing turbulence on the leeside of the pipe and thus increases the capacity for sediment transport. When the flow reverses, the leeside erosion now takes place at the other side of the pipe and some backfilling occurs at the upstream side. During a tidal cycle, however, the leeside erosion generally dominates over this backfilling effect. As a consequence the leeside erosion lowers the seabed near the pipe and at a certain moment the pipe will again be exposed to tunnel erosion.

Both tunnel and leeside erosion have been clearly observed in large scale two dimensional flume tests (scale 1:1 to 1:2).

Small scale flume tests and field data confirmed the existence of the third process which determines the actual burial of the pipeline: the natural backfilling.

As the local seabed around the pipe and the pipe itself sink relative to the original seabed, the capacity of the upstream eroded hole to capture sediment increases, so that backfilling becomes more and more dominating over leeside erosion.

The environmental conditions (waves and currents) determine to a large extent the equilibrium position of the pipe. In fact, each set of environmental conditions result in a different equilibrium position. It means that for instance a storm may trigger further lowering of the pipe after a period of stable pipeline position.

2.3 Calibration of the model

The model is primarily based on flume tests and numerical models of the two-dimensional situation of the behaviour of a cross-section of the pipeline. As a consequence, the pipeline diameter and the pipeline configuration (plain or fitted with piggy-back or spoiler) are in fact the only pipe related parameters taken into consideration in the formulations. Pipeline stiffness and weight are not (yet) explicit input parameters.

The calibration of the model has therefore been very important. Fortunately a relatively large number of pipelines in the Dutch sector of the continental shelf have self-buried and regular surveys showing the lowering process were available.

The shortcomings of a 2D model concept has been compensated by extensive calibration of the model against actual field (3D) survey data.

Actual waverider buoy measurements covering the survey periods of the calibration pipelines have been used to ensure correct representation of the wave conditions. The importance of this is clearly demonstrated in Figure 3. It shows the measured and the computed position of pipe and seabed of the Mobil P6AB pipeline. The effect of storms is evident after approximately 90 and 160 days.

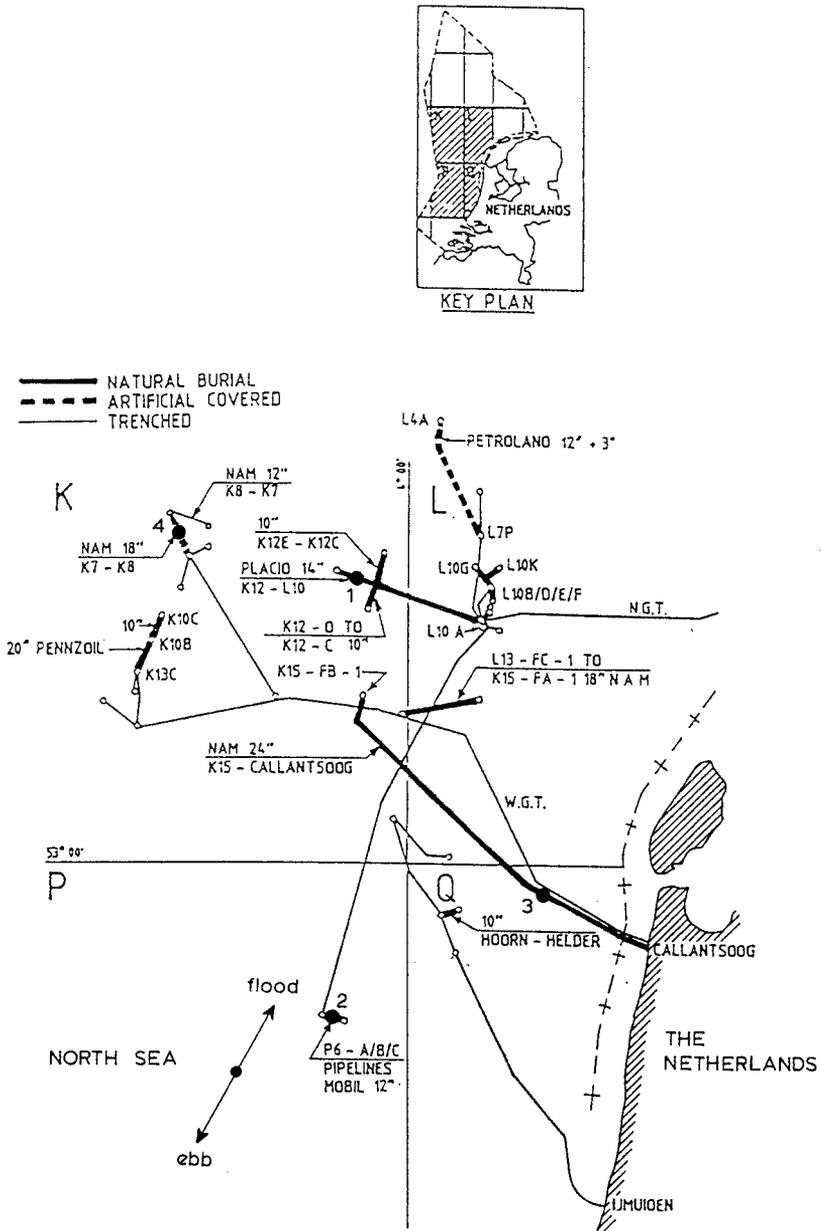


Fig. 2 Pipeline locations in North Sea.

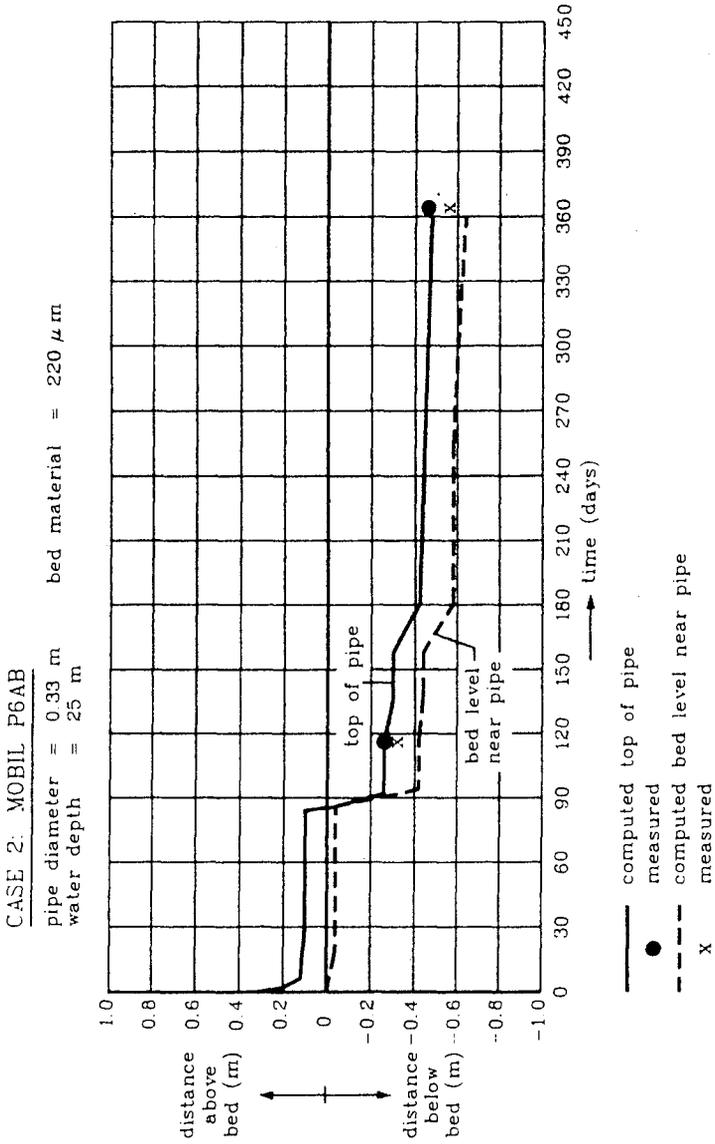


Fig. 3 Computed and measured pipe position of CASE 2: MOBIL P6AB

3. CONCLUSIONS

Two independent PC based computer models have been developed. Both models require comprehensive environmental input data in the form of time series of wave/current data. By the new developments existing theoretical models have been integrated to useful tools that are operational in engineering practice.

The Sand Wave Model:

- the model has been developed from existing theoretical models.
- modifications and extensions of the existing theory has been made with respect to:
 - . non-equilibrium sand waves
 - . varying directions of waves and currents
 - . effects of short-crested sand waves
 - . effects of high water waves
 - . effects of wave generated ripples.
- the model does not include the presence of any structure.
- future extensions of the existing model should focus on including the observed natural variability of sand waves.

The Self-Burial Model:

- the Self-Burial Model integrates the effects of the physical processes of tunnel erosion, leeside erosion, pipe lowering and backfilling.
- the model has been based on flume tests and numerical models.
- effects of pipeline stiffness is not included directly in the (2D) model but enters through extensive calibration against actual (3D) pipelines.
- future extensions of the existing model should focus on modelling of the development of free spans and pipe lowering in the scour holes.

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