CHAPTER 146

NUMERICAL MODELING OF BREACH GROWTH IN A SANDDIKE

by

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ABSTRACT.

On October 7, 1994 a full scale breach growth experiment has been carried out at the Dutch-Belgium border, in the Zwin at the west-end of the Dutch-Belgian border. A small breach was initiated in a sanddike with a height of 3.25 m and the breaching process was monitored. The present paper focusses on the numerical modeling (hindcasting) of the breach growth process.

1 INTRODUCTION; GOAL OF THE INVESTIGATIONS.

De Looff at al.(1996) give a summary of the current breach research in the Netherlands. The present paper reports on a part of these investigations, i.e.:

- . the full scale field experiments of 1994 (Zwin);
- . the present research, which should lead to a comprehensive numerical model for breaching processes.

On October 7, 1994 a full scale breach growth experiment has been carried out at the Dutch-Belgium border, in the Zwin [1], [2]. Fig. 1 shows the site of the experiment at the west-end of the Dutch-Belgian border.

A small breach was initiated in a sanddike with a height of 3.25 m and the breaching process was monitored. The experiment was carried out by Rijkswaterstaat and Delft University of Technology, with aid of Delft Hydraulics Laboratory.

Ultimate goal of the investigations, which are reported on is extrapolation of the behaviour of the Zwin breach to other circumstances:

- . Construction of the dike: use of other materials (clay instead of sand);
- . Larger areas of the submerged basin;
- . Other bottom level of the submerged basin.

The present paper focusses on the numerical modeling (hindcasting) of the breach growth process.

2 MEASUREMENTS.

The Zwin experiment has been reported before (Visser et al., 1995a,b): an experimental closure of a tidal basin near the Dutch-Belgian border, which is a historic residual of the former tidal entrance to the Belgian city of Brugge, called "the Zwin" (fig.1). Twice in 1994, on October 6 and 7, the inlet was closed off by a sanddam; breaching was simulated by digging a small trench in it at High Tide.

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Fig. 1. Situation

Fig. 2. Site of measuring poles

Fig.2 shows the measuring site with the artificial dam.MP 2..6 are measuring poles in which the water level and the current velocities were measured. Maximum velocities were of the order of 4.2 to 4.4 m/s.



Fig. 3. Trillo's

An array of burglar alarms was placed in and underneath the dike in order to monitor the breaching process.

These alarms, specially developed for these tests were called: "Trillo". Fig.3 shows the instrument, with a PVC bar fixed to a watertight PVC cylinder (length 20 cm). The alarm consists of a piezo-electric element, reacting on the motion of a bolt, which is attached to it with a spring. When the PVC cylinder, buried in the sand, turns loose the bolt will tremble, which is monitored by an electrical signal in the piezo-electric element.

Fig.4 shows the sites, where the burglar alarms were placed in and underneath the dike and furthermore the dimensions of the dike.

³Dutch: translated in English: "Tremblo",

It reaches up to a level of 3.30 m above NAP (which is about OD). The high water level at which the breach was initiated was NAP 2.75 m. An initial trench in the dike was dug to a level of NAP +2.50 m. Poles had been placed in a 5mpattern underneath the dike. The Trillo's were attached on these poles at a mutual distance of 75 cm (4 Trillo's per pole). This paper reports on the second day of experiments. At that time the Trillo's in the body of the dike were not present any more because of the experiments of the first day. Also the Trillo's in the front of the dike disfunctio-



Fig. 4. Site of the trillo's

ned the second day. Before looking to some results of the measurements it is useful to



Fig.5. Phases

the experiment. The dip "a" in the water level on the seaside ofthe dike is caused by the Bernouilli term. Measuring gauges in deeper water show a convex curve in this hour. The thin lines (with the scales on the right) show the measured velocities in course of time on the seaward and landward side (maximum velocities 4.3 m/s). remind to the various phases of breaching, as indicated by Visser (1995).

Successively, one observes the steepening of the inner slope, backscour of the inner slope and removal of the sill at the seaside of the dike. After this happens, the breaching process reinforces itself and a quick growth of width takes place.

Refer to fig.5 and the photo's 1 to 8. The thick lines in fig.6 show the water level at the seaside and the inner side of the dike as function of time.

Indicated is the moment of initiation of the breach and the moment of the turning of the tide; furthermore the end of



Fig. 6. Measured waterlevels and velocities



Photo.1 Initial trench

Photo.3 Current velocities after 6 min.





Photo.2 Current velocities after 1 min.

Photo.4 Current velocities after 7 min.







Fig.7 . Trillo registrations

Fig.7 shows the 8 registrations of two vertical rows of each 4 Trillo's, somewhat landward of the center of the dike. The larger the deviations between the lines "a" and "b", the larger the motion of the Trillo's.





From visual analysis of the video's and slides and from the Trillo's figure 8 resulted concerning the relation between the water motion and the scour. In the lefthand figures the dike and the watersurface at various times are indicated. No attention to the bottom is given. In the righthand picture the bottom development is shown The black and white band show the rate of

band show the rate of inaccuracy. The white band was caused by malfunctioning of the Trillo's; the black band by inaccuracy of interpretation. In both cases the upper side of the band is more likely than the lower extreme (in the white area this lower extreme is even very unlikely).

It doesnot seem likely, that much dee-

per scour holes occurred than 2m below the original bottom level. These scour holes were formed not before the phase of the breach, in which the sill at the seaside was removed.

COMPUTATIONS.

Numerical computations were made with the 3-D Trisula model of Rijks waterstaat and Delft Hydraulics. Fig.9 shows a 2DH-current pattern.



Fig.9 . Trisula model

With a flat sloping bottom (dotted line in fig.10), the water surface (in the center line of the breach)according to the line, indicated with diamonds was computed. The stars show the measured values after 12.5 minutes. If scour holes were introduced (lines with triangels; fig.10), the water surface as indicated by blocs was the result.



Fig.10. Water surface

Fig.11. Velocities

Current velocities (for the case withflat bottom) are as given in fig. 11. Maximum velocity found was 4.3 m/s.

IMPRESSIONS.

From the investigations, the following impressions resulted concerning the hydraulic aspects:

- . The maximum velocities occurred shortly after washout of the dike in the breach;
- . At that time the water level in the breach is mainly determined by the Bernouilli term;
- . The Froude number of the maximum velocity occurring in the breach was of the order of 1.5;
- . The current has a jet-like character; a Bernouilli-trough with a depth of ca. 1 m could be observed, ending in an undular jump.

Concerning morphology the following observations c.q. impressions seem relevant:

- . The dike washed away with a velocity of 2 m/minute; the sand consisted at some places of coarse sand, originating from a former artificial sandsupply with a grain diameter of .315 mm and at other places of local sands with a diameter of .185 mm.
- . Scour started to develop at the landward side of the dike; this might have triggered an undular jump.
- . The bottom changes show some similarity with the dynamics of antidunes.

SEDIMENT TRANSPORT MODEL (under development).

A pick-up transport model (appendix B) is developed with to the following characteristics The acting forces are: self-weight; shear stress on the surface and the horizontal pressure gradient. Hindered entrainment is implemented, caused by:

percolation resistance.

This is the resistance which occurs if a mass of grains is moved in vertical sense through a mass of ambient fluid.

Percolation resistance does not imply changes in the grain structure.

Suction forces in the contact points between the grains.

Additionally to the percolation resistance, there should be accounted for viscous suction forces, caused by opening of the grain structure, at the sites, where originally touching grains are quickly separated.







Fig.13. Virtual reduction self-weight by horizontal pressure gradient.

The feature to be simulated is called: "zip-erosion" (fig.12).

In the course of the breaching process, the career of a grain is as follows. First it makes part of the soil, then of fluffy soil, then of dense slurry and it ends as suspension transport. When the thick horizontal line in fig.12 is the bottom, the curved line shows the mutual distance between the grains, i.e.: an average gap width. The mutual distance between the grains is negative-exponential in downward direction.

During "Zip-erosion" the process of detaching propagates (as a zipper) with constant velocity in downward direction.

In the area just below the bottom line dilatation takes place. In the model, this process is simulated because the grains play leap-frog, triggered by gravity and pressure. Instability occurs, when the layers play leap-frog; this doesnot lead to infinite transports.

The model starts from the presumption of perfect spheres, which gives instability under an

angle of repose of 30° in the case of a horizontal water surface, but 15° in case of a water level, parallel to the slope. This because of the effect of the horizontal water pressure. Based upon hydrology and soil mechanics considerations, essentially the same mechanism was indicated by Edelman (1960).



Fig.14. Direction motion grains for horizontal water level and for water level parallel to slope.

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7 APPENDIX A. Hydraulic Model

MODEL EQUATIONS

The numerical model is based upon approximations for continuity, momentum, and transport equations for salinity, heat, turbulent kinetic energy k and its dissipation rate ϵ . The equations are based upon the hydrostatic pressure assumption. In cartesian coordinates they are given by:

continuity equation:

$$\frac{\partial u_i}{\partial x_i} = 0 \tag{1.a}$$

momentum equations in the horizontal:

$$\frac{\partial u_i}{\partial t} + \frac{\partial u_i u_j}{\partial x_j} + \frac{1}{\rho_0} \frac{\partial p}{\partial x_i} + \frac{\partial R_{ij}}{\partial x_j} = f_i, \quad i=1,2$$
(1.b)

hydrostatic pressure assumption:

$$\frac{\partial p}{\partial x_3} = -\rho g \tag{1.c}$$

transport equations:

$$\frac{\partial c}{\partial t} + \frac{\partial F_i}{\partial x_i} = P \tag{1.d}$$

where u_i is the velocity in x_i direction, p is the pressure, ρ is the density, ρ_0 is a reference density, g is the acceleration due to gravity, c is the concentration, f is the external forcing, e.g. wind, F is the concentration flux due to convection and diffusion, R is the Reynolds stress tensor, and P is the dissipation and/or production function. The Reynolds stresses are modeled via the well-known eddy viscosity concept. The density ρ is supposed to be constant i.e. $\rho = \rho_0$.

The equations are transformed via a transformation given by:

$$\xi = \xi (x_1, x_2) , \quad \eta = \eta (x_1, x_2) , \quad \sigma = \frac{x_3 - \zeta}{h}$$
⁽²⁾

in which ζ is the free surface elevation, and h is the water depth. In the horizontal the transformations are orthogonal, in the vertical it is the well-known sigma transformation.

For the numerical approximation a "C grid", i.e. a fully staggered grid has been chosen. As far as the spatial derivatives are concerned the approximation method is based upon second order central differences for the continuity equation and for the pressure terms in the momentum equation. For the advection terms some weighted average is being used of second order upwind differencing and second order central differences. The integration in time is based upon a second order ADI and AOI (alternating operator implicit) type of factorization technique. Details are given by Leendertse (1989), Stelling and Leendertse (1992).

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8 APPENDIX B. Sediment transport model

The principles of the sediment transport model, which will be used, are described in Bakker & van Kesteren (1986), Bakker, van Kesteren & Klomp (1990), Bakker & van Kesteren (1994) and Bakker & van Kesteren (1996). A comprehensive report concerning the theory will be published in 1997.

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