

HOW STRONG ARE OUR LEVEES? HYDRAULIC ANALYSIS BASED ON POLDER2C'S PROJECT IN SITU TESTING

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KEY WORDS

Overflow, in-situ testing, models comparison, models calibration, polder2C's.

ABSTRACT

In the framework of climate change, and especially sea level rise, the Polder2C's project has been set up to improve, among others, knowledge of external erosion processes on seashore earth dikes. Within the project, overflow, overtopping and breaching in-situ tests are carried out in the Hedwige- and Prosperpolder, while numerical models are set up to improve technical analysis of testing results. This paper describes blind predictions of water depth and flow velocity on the slope of a levee during an overflow event prior, before the test is conducted, using a range of different state-of-the-art numerical models. This model predictions were performed by several partners of the Polder2C's project. No in advance calibration of the numerical models was carried out, all models considered the same levee geometry, roughness of the grass cover layer and upstream boundary conditions. The aim of the comparison is to identify best numerical tools to predict flow pattern for different hydraulic loadings, since it takes part of the erosion process, which is to be analyzed as a second step. Homogeneous results have been found with the different deployed tools, with some spread partially linked to numerical uncertainties, and calculation methods. Next, numerical modelling steps within the project are calibration of the hydraulic models (e.g. the roughness of the grass cover layer) and modelling of the erosion process under calculated hydraulic loading conditions. In order to calculate the erosion of the cover layer, the computed erosion will be compared with the erosion measured during the field experiments within the coming years.

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1. THE HEDWIGE AND PROSPER POLDER: A PLAYGROUND FOR HYDRAULIC ENGINEERS

Worldwide, floods affect more people each year than any other natural hazard [1]. Global warming causes the sea level along European coasts to rise [2], and increasing autumn and winter rainfall has led to an increase of river discharges and flooding in Northwestern Europe [3]. This poses an increasing threat to flood disasters for the coming years. Therefore, an ongoing effort is required to improve current flood risk management practices in order to keep flood risk at a minimum.

At the border between Belgium and the Netherlands, the Hedwige- and Prosperpolder are to be transformed into new tidal nature. Before their removal, the existing levees offer a unique chance for in-situ strength and emergency response testing. The Polder2C's project, part of the INTERREG programme, is a joint collaboration between (the coastal regions of) France, Belgium, the Netherlands and the United Kingdom, which aims at keeping these countries safe by adapting to climate change. Protection against, and adaptation to the consequences of climate change, are the main topic of this project. To achieve that objective, we have a large study area at our disposal: the Hedwige- and Prosper polder (see Figure 1). These polders will be turned into tidal nature areas, and a new levee is built more inland. The 'old' levee becomes redundant and will be partly removed, which means that over 3 km of levee becomes available for testing current and innovative techniques, processes, methods and products for practical validation, in an area already facing climate change effect, especially maritime-related, such as sea level rise and increasing storm impact on its coasts. Between 2020 and 2022 a range of experiments is planned, varying from testing the levee's resistance against extreme loading conditions (i.e., wave overtopping and overflow), levee breaching, to repairing damage in an innovative way, inspection and emergency response.

To gain more insight in the resistance of the levee cover layer against overflowing water, a series of continuous overflow tests have been performed in the period October-December 2020, and February-March 2021. These tests contribute to improving predictive hydrodynamic and erosion models, which in turn leads to better predictions of levee failure and reduced uncertainty in levee safety assessments. The tests could also lead to valuable insights for levees that are specifically designed to overflow once the river water level exceeds a certain threshold value, to reduce the hydraulic loads on the flood defense system further downstream (see for example [4]).

This paper aims at comparing hydrodynamic predictions of several overflow models with outcomes of the field tests. The set of models that are used in this paper is representative for the models that are currently used by Polder2C's partners, and includes a range of models with a varying level of complexity.

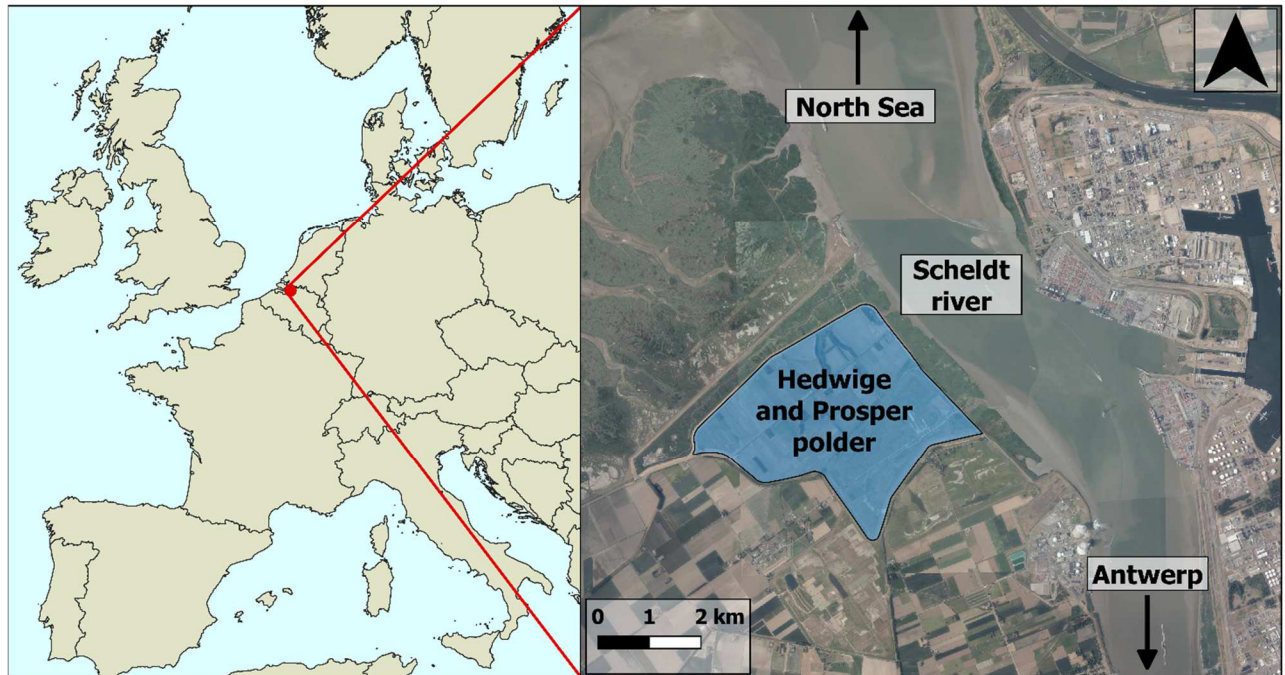


Figure 1. Left: overview of the location of Hedwige and Prosper polder. Right: a close-up.

2. THEORETICAL BACKGROUND

General consideration

Overflow over a levee is similar to overflow discharge on dam spillways, being better documented regarding hydraulic theories. The theoretical background in this chapter is mainly based on the reference work regarding grassed spillways: Design of reinforced grass waterways [5].

Hydraulic analysis of a spillway usually involves four conditions of flow (see Figure 2) [5][6]:

- Subcritical flow in the slope approach, initially at a low velocity, accelerating, however, as it approaches the crest.*
- Critical flow as the water passes over the spillway crest.*
- Supercritical flow on the slope below the crest; uniform flow conditions are achieved when terminal velocity is reached and energy loss due to friction is matched by energy gain from hydrostatic head.*
- Transitional flow at or near the terminus of the chute where the flow must transition back to subcritical generally with a hydraulic jump with the tailwater.*

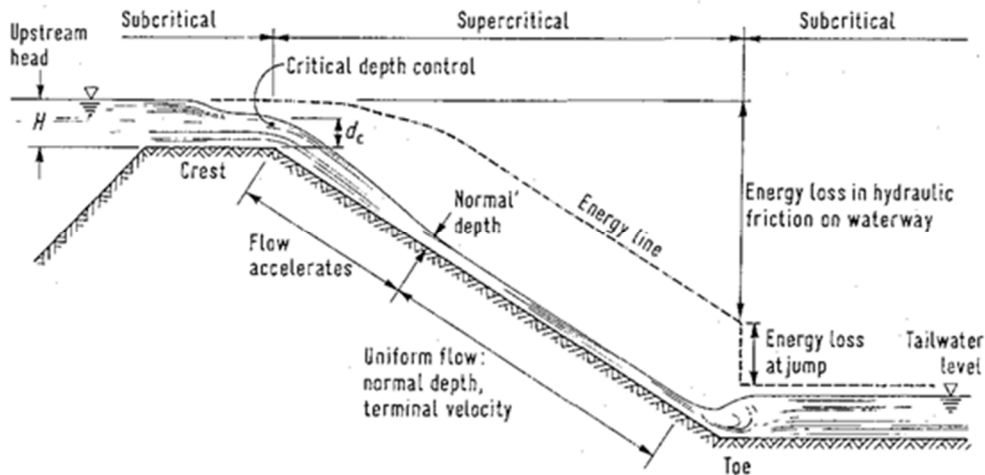


Figure 2. Elevation showing flow down typical reinforced grass waterway [5].

The velocity and flow depth in a waterway depends on the design discharge, the shape of the waterway cross section, the longitudinal slope of the waterway and the hydraulic roughness of the grass of the armour layer. To describe quasi-unidimensional flows into a channel or a river, shallow water equation has been developed by Saint-Venant, derived from depth-integrating the Navier–Stokes equations for cases where the horizontal length scale is much greater than the vertical length scale. The system of partial differential equations which describe the 1-D incompressible flow in an open channel of arbitrary cross section is:

$$\frac{\partial A}{\partial t} + \frac{\partial (Au)}{\partial x} = 0, \quad \text{and} \quad (1)$$

$$\frac{\partial u}{\partial t} + u \frac{\partial u}{\partial x} + g \frac{\partial \zeta}{\partial x} = - \frac{P \tau}{A \rho} \quad (2)$$

Where x is the space coordinate along the channel axis, t is the time, $A(x,t)$ is the cross-sectional area of the flow at location x , $u(x,t)$ is the flow velocity, $\zeta(x,t)$ is the free surface elevation, $\tau(x,t)$ is the wall shear stress along the wetted perimeter $P(x,t)$ of the cross section at x , ρ is the (constant) fluid density and g is the gravitational acceleration.

At the crest of the levee, where the flow accelerates from the relatively static conditions upstream of the crest, the flow condition within the fluid will be laminar (which is characteristic of viscous fluid flow). This condition is inherently unstable and turbulent flow conditions will develop. Turbulent conditions at the water surface promote air entrainment, which, in turn, will increase the specific volume of the water and will result in an increased depth of flow. The flow depth typically increases up to 40% over the unaerated water depth in highly turbulent flow conditions [5]. This is shown in Figure 3. Air-water mix is only considered with Navier-Stokes's equations, which is less used than the derived shallow water equations within hydraulics engineering.

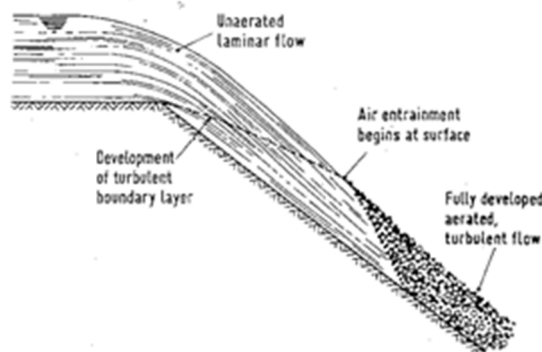


Figure 3. Development of fully aerated flow [5].

Energy grade line equations

The flow in a prismatic channel with a constant slope can be easier described by the equation of Bresse, derived from the Bernoulli equation [7][8]:

$$\frac{dh}{ds} = \frac{S_0 - S_f}{\cos \theta - \frac{Q^2 B}{g A^3}} \quad (3)$$

where ds denotes the distance between two infinitely close sections [m], dh is the loss in piezometric head between these two sections [m], S_0 is the bottom slope [-], θ is the angle between the slope of the channel and the horizontal bottom [rad], S_f is the friction loss [-], Q is the discharge [m³/s], B is the top width of the channel [m], A is the wetted section of the channel [m²] and g is gravitational acceleration [m/s²].

For solving equation 3 the section of the overflow levee is considered as an infinite width section, the friction loss is computed using the Manning equation (equation 4) and the parameter $\cos \theta$ can be replaced by $\sqrt{1 - S_0^2}$. Consequently, the following function is derived:

$$\varphi(h) = \frac{dh}{ds} = \frac{S_0 - n^2 \frac{q^2}{h^{10/3}}}{\sqrt{1 - S_0^2} - \frac{q^2}{g h^3}} \quad (4)$$

where $\varphi(h)$ is the function to be solved [-], n is the Manning roughness coefficient [s/m^{-1/3}], q is the discharge intensity [m²/s], h is the water depth [m] and S_0 is the bottom slope [-].

Manning formula

The empirical Manning's equation is commonly derived to describe the type of flow, and link velocity to river bed roughness:

$$V = \frac{R^{2/3} S^{1/2}}{n} \quad (5)$$

Where V is the mean velocity of flow (m/s), R is the hydraulic radius (m) which equals cross-sectional area of flow divided by wetted perimeter, S is the slope of the energy line and n is the Manning's roughness coefficient.

For a given waterway slope and discharge, if uniform flow conditions (Manning's formula) are supercritical then the slope is termed hydraulically 'steep', which is the case for the overflow that is modelled here. Various features of supercritical flow should be noted [5]:

- *Unless a hydraulic jump occurs and the flow becomes subcritical, flow is controlled by conditions upstream and not by conditions downstream, meaning downstream conditions do not affect the flow pattern.*
- *The velocity head is large in relation to flow depth. On steep embankment slopes, the Froude Number could be as high as 8.0 and the specific energy of the flow will be principally contained in velocity head as opposed to flow depth.*
- *With all supercritical flow, any change in cross-section is likely to give rise to a standing wave pattern throughout the water surface downstream, and this will have corresponding effects on local velocity and flow depth.*

It is emphasized that Manning's equation is empirically derived and simply provides an approximate relationship for hydraulic resistance over a range of flow conditions commonly encountered in civil engineering applications. The roughness coefficient is not dimensionless and furthermore varies with hydraulic loading.

Hydraulic resistance for turbulent flow on steep slopes may be more accurately described over the complete range of laminar transitional and turbulent flow regimes by the Colebrook-White equations.

Roughness of vegetated slope

The hydraulic roughness of a grassed surface depends on the physical characteristics of the grass sward such as its height stiffness and density, and its interaction with the flow. This interaction can be divided into three basic regimes according to the hydraulic loading:

1. The flow depth is significantly less than the height of the vegetation, which is not deflected and the velocity at the soil surface is low due to the interference effect of the vegetation;
2. The combined effect of increasing flow velocity and depth causes the vegetation to deflect and oscillate in the flow;
3. The velocity is high enough to lay the vegetation down and a relatively smooth, stationary surface is presented to the flow. The effective height of the vegetation is then considerably less than its natural height. This is the regime that will generally be encountered in steep waterways or other applications with high velocity flow.

These regimes are illustrated in Figure 4.

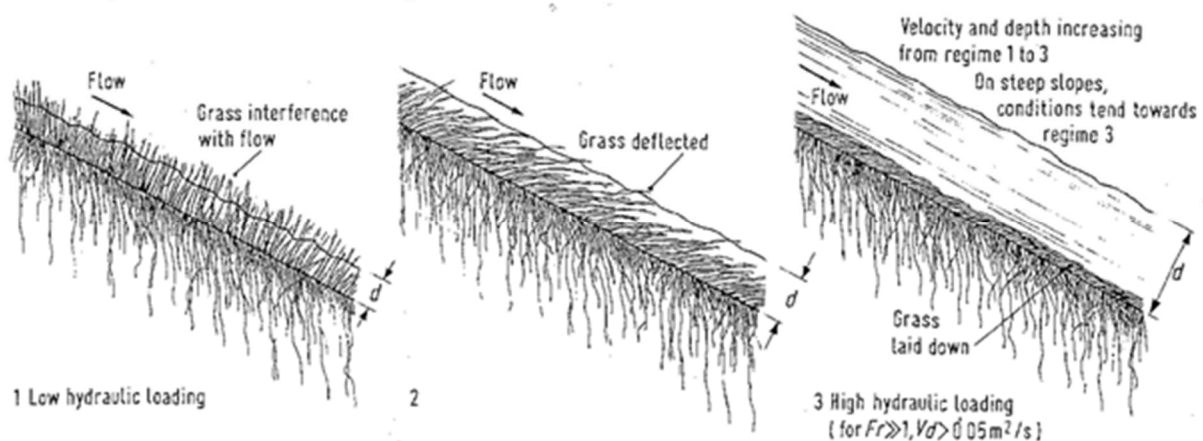


Figure 4. Effect of hydraulic loading on grassed surface [5].

For slopes flatter than 1/10 commonly the VR-method is used [5]. This method relates the Manning roughness coefficient to flow in the form of a hydraulic loading parameter VR. The left panel of Figure 5 presents the relation between the Manning roughness and the parameter VR for difference grass retardance categories. Note that the flow parameter VR in this figure is equal to the discharge intensity q in a hydraulically wide channel. For slopes steeper than 1/10 the grass is laid down during overflow. Consequently, the Manning roughness value is independent of the hydraulic loading and the grass length, but varies only with the slope. Based on the CIRIA project, [5] recommends a Manning value of $0.030 \text{ s/m}^{1/3}$ for slopes of 1/10 and a value of $0.020 \text{ s/m}^{1/3}$ for slopes of 1/3 and steeper, with intermediate values as indicated in the right panel of Figure 5. Note that that these values are not applicable for low hydraulic loading ($q < 0.05 \text{ m}^2/\text{s}$).

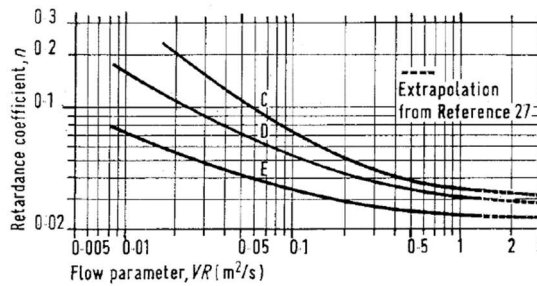


Figure 7 Hydraulic roughness of grasses

Table 2: Grass retardance categories

Average grass length	Retardance
150 mm to 250 mm	C
50 mm to 150 mm	D
Less than 50 mm	E

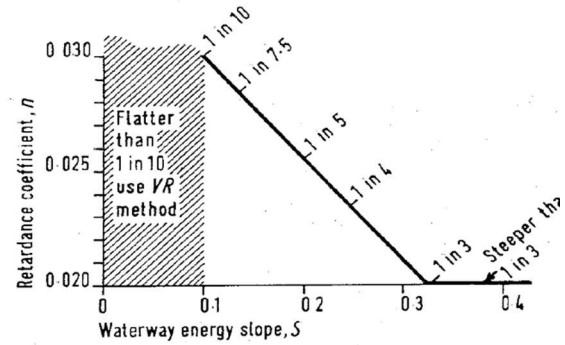


Figure 8 Recommended retardance coefficients for grassed slopes steeper than 1 in 10

Figure 5. Manning roughness values; left panel: slopes flatter than 1/10; right panel: slopes steeper than 1/10 [5].

3. FIELD TESTS

3.1 Continuous overflow tests

During an overflow test the overflow generator (left panel of Figure 6) is positioned on the riverside slope of the levee near the crest. The overflow generator, having a width of 2.0 m, generates a continuous adjustable discharge that is uniformly divided over the width of the section. The water is pumped into the overflow generator using a frequency modulated pump. Depending on the pump capacity a maximum discharge until 0.500 m³/s/m can be reached, according to a maximum overflow water depth above the crest of 0.50 m. On the crest and the levee slope at polder side wooden plates form the edges of the test section (right panel of Figure 6).

At four locations on the levee slope, in this paper indicated with crest, upper, middle and lower (Figure 7), the water depth and flow velocity are measured. At two other locations on the levee slope, a camera setup holding two cameras is present, one camera looking into the upstream and the other into the downstream direction of the slope. These cameras, thus covering the entire levee slope, regularly record the flow pattern on the levee slope during a test and record the possible damage of the cover layer during and after a test.



Figure 6. Left: side view of test location; right: test section during overflow test (source: Stephan Rikkert).

3.2 Test scenarios

The reference scenario considers a smooth levee geometry and good grass cover quality. This scenario was tested using different overflow discharges. Besides this reference scenario, also other scenarios were included in the test program. These other scenarios comprise amongst others:

1. *varying grass heights (varying mowing or maintenance regime),*
2. *the presence of a cliff,*
3. *the presence of anomalies like animal burrows or a tree on the levee slope,*
4. *the presence of emergency repair measures (covering the eroded part with EPDM foil),*
5. *the presence of a stair*
6. *the presence of asphalt on the crest or a road down the slope*

To give an impression, some of these scenarios are included in Figure 7.



Figure 7. Top left: Test section with a tree at the bottom end of the landside slope, prior to overflow test, picture taken by Stephan Rikkert on February 16, 2021. Top right: Overflow test on an emergency repair measure: damaged landside slope was covered with EPDM foil, picture taken by André Koelewijn on February 7, 2021. Bottom left: Prepared test section with fox holes present at the landside slope, picture taken by André Koelewijn on November 23, 2020. Bottom right: Test location with stairs, to study soil-structure interaction, prior to overflow test, picture taken by Stephan Rikkert on March 30, 2021.

4. OVERFLOW MODELS

4.1 Introduction to the overflow models

4.1.1 Hydraulic grade line (FHR)

The model calculates the hydraulic grade line on the landward slope of the overflow levee. Equation 3 is solved using the trapezoidal method with a constant water height difference of 0.001 m in the vertical direction and the water depth at crest level as upstream boundary condition.

The calculation of the hydraulic grade line is carried out in Matlab and results into water depth and flow velocity on the slope of the overflow levee. Aeration of the flow is not taken into account. Any possible erosion of the cover layer can only be assessed by comparing the computed flow velocity with limiting values for flow velocity [5][9]. The extent of the erosion cannot be predicted with this model.

4.1.2 1D Saint-Venant Equations (Cerema)

The computer code used for the case study simulations is the 1D version of the Saint-Venant equations (also called shallow water equations). The approximation of these equations uses the finite volume method. Several second order numerical flux calculation schemes have been implemented in the code, such as Rosanov and HLL (Harten, Lax and van Leer) schemes. To reconstruct the variables in the numerical flux calculation, the code offers the choice between the second method MUSCL (Monotonic Upwind Scheme for Conservation Law), ENO (Essentially Non Oscillatory) and modified ENO. This reconstruction method has been proposed to preserve equilibrium states in the presence of source terms (well-balanced scheme) and to guarantee the positivity of water level (h) at the dry/wet interfaces. Likewise, to maintain second-order precision, various limiters (minmod, superbee, van Leer, van Albada) can be coupled with these reconstruction methods to preserve the maximum principle. The Heun scheme was implemented for time discretization in order to ensure a second order accuracy. Finally, different types of boundary conditions which cover all the flow regimes governed by the Saint-Venant equations have been implemented for a more flexible use of this computer code.

4.1.3 Star-ccm+ CFD model (Cerema)

The Star-ccm+ software is used to simulate the overflow events. The CFD model used to perform this work is verified and validated previously for different problematics including turbulent flows with free surface. The selected CFD software is based on the Reynolds Averaged Navier-Stokes (RANS) equations in Cartesian coordinates. The (SST) $k-\omega$ turbulence model is chosen to close the Navier-Stokes's equations and to predict the hydrodynamic forces at near – wall regions. The flow was defined by an inlet boundary condition at the crest of the dike (corresponding to a water level and mass flow rate) while the pressure-outlet boundary condition is used at the outlet boundary of the domain leaving the water height as a variable to be calculated.

4.1.4 SVE2D-BF (UCLouvain)

This finite-volume model is under continuous development by PhD students and researchers of the UCLouvain Hydraulics team. It solves the shallow-water-Exner equations, i.e. water flow and sediment continuity [11], with different possible closure equations for predicting sediment transport. The equations are solved using an explicit finite-volume method on unstructured grids, with different flux calculation schemes available (HLLC, Roe, different topographic source terms treatments). The code also has a bank-failure operator [12], based on simple geotechnical considerations, to account for the failure of local steep slopes. This simulation tool was used several benchmarks tests with other researchers, and sometimes some consultancy activities. It should therefore be able to predict the progressive development of an initial breach in a dike, and the erosion of the downstream slope of the dike.

4.1.5 Ansys CFX (ISL)

Ansys CFX is a general Computational Fluid Dynamics (CFD) software that works by solving the governing equations of fluid flow. Navier-Stokes equations are discretized and numerically solved basing on the finite volume method (see section 2). It delivers solutions across a wide range of CFD and multiphysics applications. The model is built on 2D-dimension (X,Z) along the defined cross section, and results are extracted directly all along the profile, in terms of water depth and velocities.

4.1.6 TELEMAC-2D (ISL)

TELEMAC-2D is a library of the TELEMAC-MASCARET freeware dedicated to free surface flow computation, initially developed the National Hydraulics and Environment Laboratory (LNHE) of the Research and Development Directorate of the French Electricity Board (EDF – R&D), and is now managed by a consortium of other consultants and research institutes².

² More informations can be found in the website www.opentelemac.org

TELEMAC-2D code solves depth-average free surface flow equations as derived first by Barré de Saint-Venant in 1871 (also called the shallow water equations – see section 2). The main results at each node of the computational mesh are the depth of water and the depth-averaged velocity components. The main application of TELEMAC2D is in free-surface maritime or river hydraulics.

Turbulent viscosity may be given by the user or determined by a model simulating the transport of turbulent quantities k (turbulent kinetic energy) and Epsilon (turbulent dissipation), for which the equations are the following:

$$\frac{\partial k}{\partial t} + \vec{u} \cdot \vec{\nabla}(k) = \frac{1}{h} \operatorname{div} \left(h \frac{v_t}{\sigma_k} \vec{\nabla} k \right) + P - \epsilon + P_{kv} \quad (6)$$

$$\frac{\partial \epsilon}{\partial t} + \vec{u} \cdot \vec{\nabla}(\epsilon) = \frac{1}{h} \operatorname{div} \left(h \frac{v_t}{\sigma_\epsilon} \vec{\nabla} \epsilon \right) + \frac{\epsilon}{k} (c_{1\epsilon} P - c_{2\epsilon} \epsilon) + P_{\epsilon v} \quad (7)$$

Where: h (m) is the depth of water, u, v (m/s) are velocity components, g (m/s²) is the gravity acceleration, v_t, v_T (m²/s) are momentum and tracer diffusion coefficients, Z (m) is the free surface elevation, t (s) is the time and x, y (m) are horizontal space coordinates.

The right-hand side terms of these equations represent the production and destruction of turbulent quantities (energy and dissipation). A complete description of the theory is given in the Telemac-2D user manual (Telemac modelling system) and in the book “Hydrodynamics of free surface flows”, by Jean-Michel Hervouet (Wiley, 2007).

The two options regarding turbulence have been tested for these first hydraulic calculations, meaning:

- *The use of a constant viscosity coefficient representing either the molecular viscosity the turbulence viscosity and the dispersion,*
- *The use of the k-Epsilon equations, in which the energy dissipation is calculated by the hereinabove equations*

The second option requires a mesh thin enough to being able to solve the k-Epsilon equations. The model is built in 2 dimensions (X,Y), and the result is then extracted on a cross section.

4.2 Model settings and input

In this study we focus on one of the test locations, of which the cross-section can be seen in (Figure 8). Water depth measurements are performed at the crest (C), and 3 positions along the levee slope: upper (U), middle (M), and lower (L). The levee has a slope of approximately 1:2.5. Flow velocity measurements have been performed at these locations as well, but seemed too inaccurate at this stage of post-processing of the data, and have therefore not been included in this article.

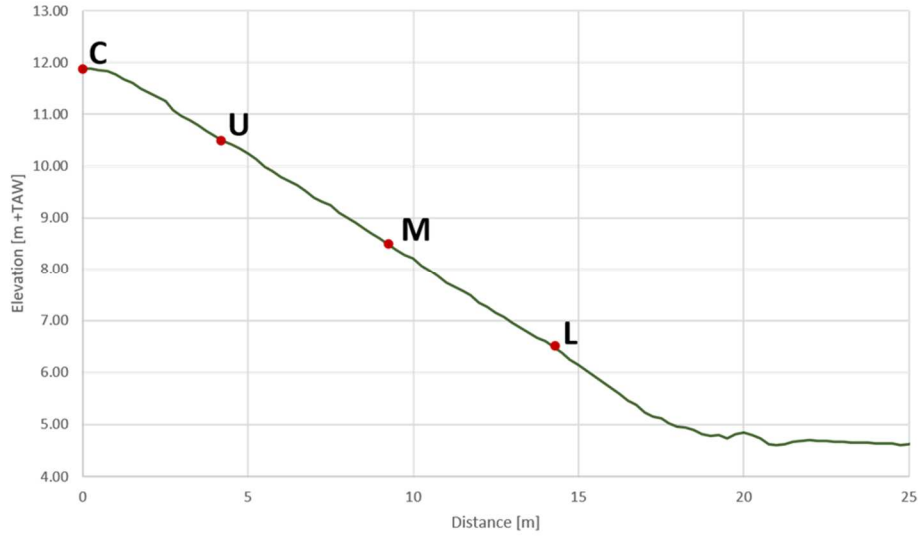


Figure 8. Cross-section of the tested levee. The locations of the measurement devices are presented as C (crest, $x=0\text{m}$), U (upper, $x=4.19\text{m}$), M (middle, $x=9.25\text{m}$) and L (lower, $x=14.31\text{m}$). The actual pump discharge and measured water level at $X=0$ are used as boundary condition.

Two overflow situations were modelled, as can be seen in Table 1. For the roughness a Manning coefficient of $0.020 [\text{s/m}^{-1/3}]$ has been used. Since some models use the sand roughness instead of the Manning coefficient, these models used a value of 0.0156 m . This value was derived, following the relation between the Manning coefficient n and the Nikuradse's equivalent sand roughness K_s , given in [10]:

$$n = 0.04 (K_s)^{\frac{1}{6}} \quad (8)$$

Table 1. Modelled overflow situations.

Water depth river – levee crest height [m]	Discharge [l/s/m]	Water depth at $x=0$ [m]
0.10	43	0.053
0.25	170	0.14

5. RESULTS

In this section we show the observed water depths, and the modelled water depth and flow velocity.

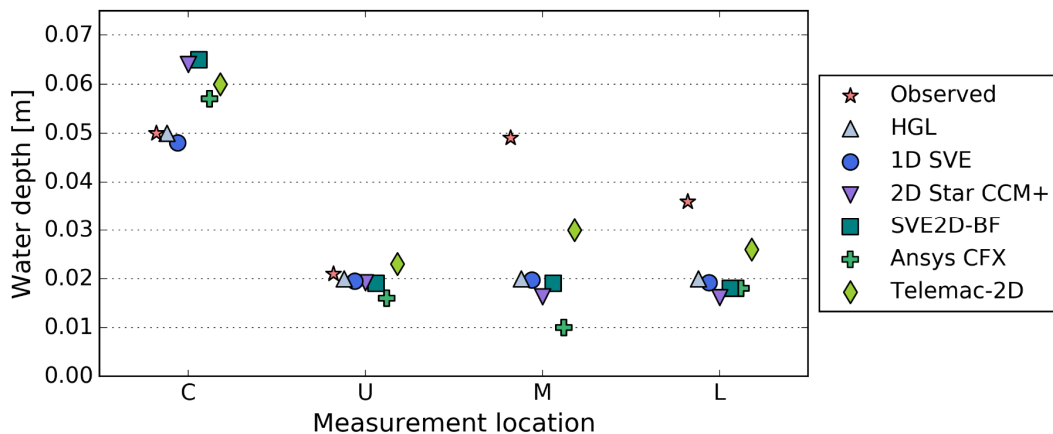


Figure 9. Observed and modelled water depth for the test where the river water level exceeds the levee crest height by 10 cm.

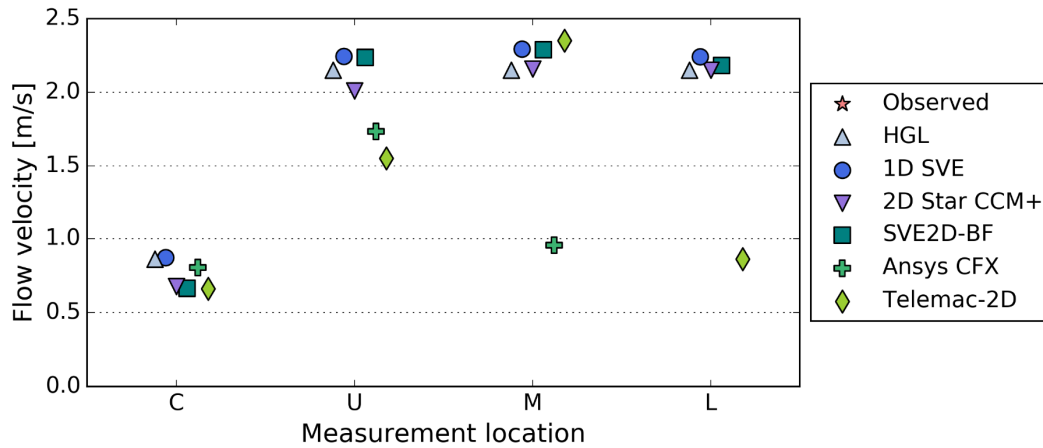


Figure 10. Observed and modelled flow velocity for the test where the river water level exceeds the levee crest height by 10 cm.

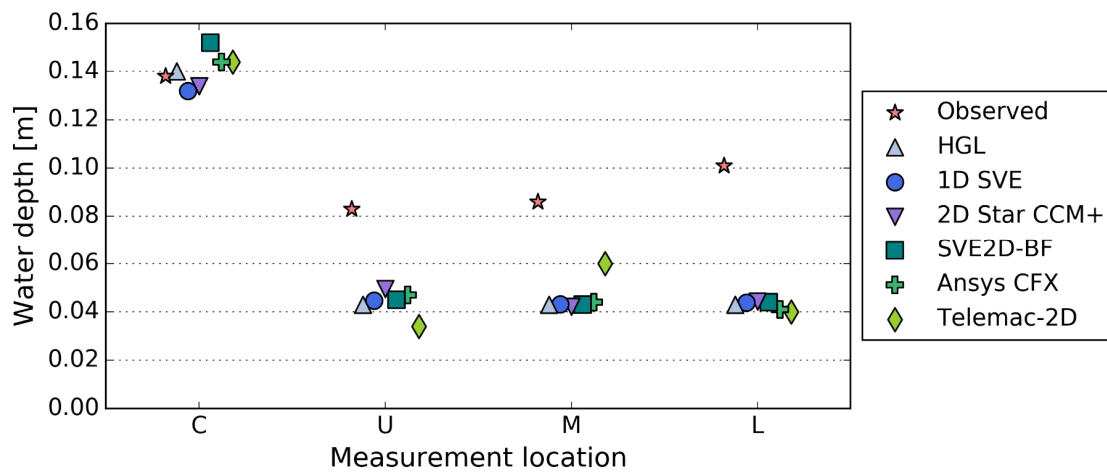


Figure 11. Observed and modelled water depth for the scenario where the river water level exceeds the levee crest height by 25 cm.

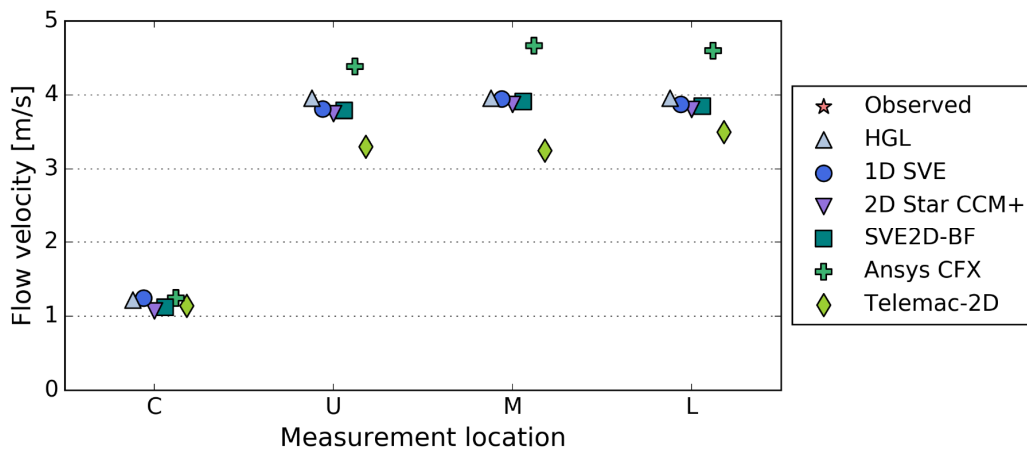


Figure 12. Observed and modelled flow velocity for the scenario where the river water level exceeds the levee crest height by 25 cm.

Comparison between the CFD model and the Shallow Water Equation (SWE) model for depth-averaged velocity in Figure 13 shows that the results are very close on the slope, when a reduction factor (here of 0.875) is applied on the surface velocity, that is obtained by CFD model.

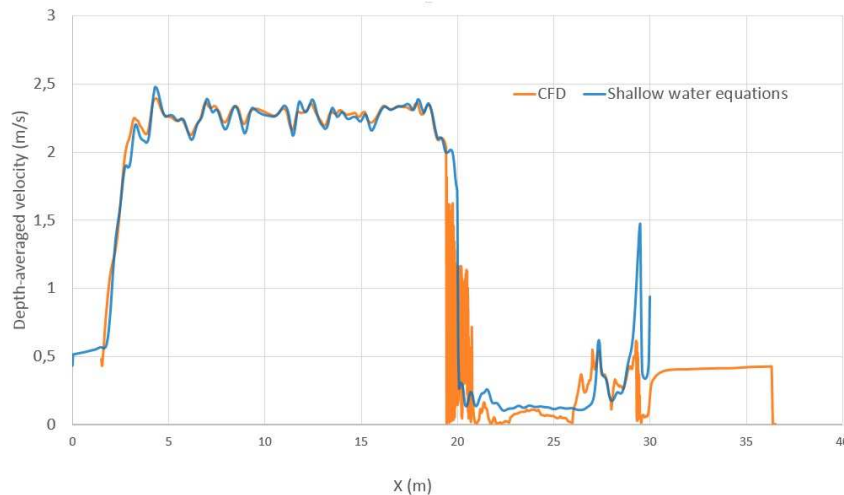


Figure 13. Comparison between CFD model and SWE model velocity calculation.

6. CONCLUSIONS

Within the framework of the Polder2C's project, continuous overflow tests, wave overtopping tests and breaching experiments are carried out in the Hedwige- and Prosperpolder. The project also comprises numerical modelling of continuous overflow and wave overtopping over a levee. This paper describes blind predictions of water depth and flow velocity on the slope of a levee during an overflow event using a range of different state-of-the-art numerical models. No in advance calibration of the numerical models was carried out, all models considered the same levee geometry, roughness of the grass cover layer and upstream boundary conditions.

The water depth and flow velocity on the slope of a levee computed with the different numerical models were compared with the water depth and flow velocity measured during the field experiments.

An explanation of the variety in results of the different numerical models is the absence of calibration of the models, being the requirement of the blind predictions. A next numerical modelling step within the Polder2C's project is the improvement of the results by calibrating some numerical model parameters, e.g. the roughness of the grass cover layer. The erosion of the grass cover layer will also be calculated and compared with the erosion measured during the field experiments.

ACKNOWLEDGEMENTS

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