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Consistent unconfined contaminated disposal facilities dike tidal flow and transport model

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Abstract

A two-dimensional time dependent coupled free-boundary numerical model for tidally-induced flow and transport through the dike of a confined disposal facility (CDF) for contaminated sediments is presented in this paper. The model simulations demonstrate the major effect that tidal fluctuations have on the dynamics of the discharge flux. Sensitivity analyses demonstrate that tidal fluctuation hastens the rate of plume migration near the exit face because of the relatively high advective and dispersive fluxes induced by tides. With or without tides, the rate of contaminant migration increases with higher CDF area hydraulic gradient.
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1. Introduction

Confined disposal is the placement of dredged material into a secure area where the sediment is physically contained. Confined disposal facilities (CDFs) are diked structures that have been built for the disposal of dredged material where in-water placement and beneficial use are not feasible or environmentally acceptable. The size, shape, design and level of complexity of these facilities vary widely depending on dredging quantities, methods of disposal, sediment contamination levels, state and local requirements and site characteristics. Confined disposal is the most commonly used management practice for contaminated sediments dredged for navigation and environmental remediation (IJC, 1997; USACE, 1997; USEPA/USACE, 1998a,b) and several computer models have been developed to support CDF design and operation (Dorkin et al., 1989; Myers, 1991; Myers and Brannon, 1991; Su et al., 2003; Su, 2004).

Many in-lake facilities have stone dikes constructed with layers of stone of increasing size. The center of the dike (the core) typically contains sand or gravel. The outer layers of the dike have stone with sizes increasing from several pounds to several tons to protect the facility from waves. Most existing, in-lake CDFs have no liners. The stone dikes are permeable upon construction. As dredged material is placed into the CDF, water is moved passively through the dike. The sand or gravel in the core of the dike functions as a filter and retains much of the suspended sediments. As the in-lake CDF becomes filled, portions of the dike become clogged as the sediments are pressed against it. The stone dike becomes progressively less permeable.

The primary objective of a CDF is retention of suspended solids, which are expected to contain the vast majority of the contaminants. The porous dike walls

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eliminate water retention but provide a pathway for untreated contaminated water to return to the fresh water body. Evaluation of water exchange and its implications for contaminant exchange is the main focus of our study.

The focus of the current work is development of a computer-modeling tool capable of evaluating the environmental significance of contaminant losses from in-lake CDFs. Data from a particular CDF near Chicago, IL and constructed in the manner described above were employed in the development of the model. Previous results of computer modeling had suggested that a CDF was efficient in retaining PCB's (Dorkin et al., 1989; Myers, 1991; Myers and Brannon, 1991). Here, we focus on the effects of tidal fluctuations on the temporal and spatial distributions of a contaminant plume within and through the dike wall. Lake waters infiltrating inward during a high-tide period and discharging outward during a subsequent low-tide period may significantly change the concentration levels of a plume as well as the rule of plume migration into the lake water. The model can also be used to examine the influence of low frequency lake water level fluctuation. Time-dependent variable water level changes in the lake produce time-variable dike-water flow into the CDF and contaminant transport outward through the dike.

This study presents a numerical solution of a two-dimensional porous flow model coupled with an advection–dispersion equation for contaminant transport in a flow domain with simultaneous CDF elevation control. This study also provides sensitivity analyses for the physical parameters that affect the transport processes of a contaminant plume within a tidally influenced zone of an aquifer.

2. Problem formulation

Fig. 1 shows an idealized version of the cross-section of a confined disposal facility dike. This idealization is made to render the problem for two-dimensions.

We will restrict our investigation to incompressible flow with constant density ρ and a homogeneous medium. In this case, we can define the hydraulic head h as a potential function for the flow:

$$h = \frac{P}{\rho g} + y.$$

The flow equation, assuming Darcy's Law, is of the form:

$$S\frac{\partial h}{\partial t} = \frac{\partial}{\partial x}\left(K_{xx}\frac{\partial h}{\partial x}\right) + \frac{\partial}{\partial y}\left(K_{yy}\frac{\partial h}{\partial y}\right); \qquad (1)$$

where h is hydraulic head (potentials), K_{xx} is the hydraulic conductivity tensor's x (horizontal) component and K_{yy} is the y (vertical) component and S is the specific storage coefficient. The storage coefficient S expresses the compressibility effects of the saturated volume element; P is the pore water pressure in the flow, ρ the fluid density, and g is the gravitational constant.

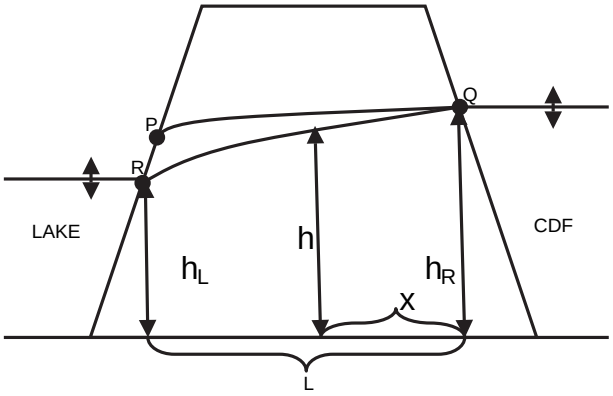

Fig. 1. Simplified dike cross-section. *PQ* and *PR* in-dike flow boundary. *PR* possible sink area.

The velocities can be estimated as:

$$V_x = -K_{xx}\frac{\partial h}{\partial x}; \quad V_y = -K_{yy}\frac{\partial h}{\partial y}.$$

The dispersion tensor D_{ij} is the sum of molecular diffusion and mechanical dispersion:

$$D_{ij} = S_0 D_M \delta_{ij} + \alpha_T |V| \delta_{ij} + (\alpha_L - \alpha_T)\frac{V_i V_j}{|V|};$$

additionally,

$$D_{xx} = \frac{\alpha_L V_x^2 + \alpha_T V_y^2}{V} + S_0 D_M;$$

$$D_{yy} = \frac{\alpha_T V_x^2 + \alpha_L V_y^2}{V} + S_0 D_M; \qquad (2)$$

$$D_{xy} = D_{yx} = (\alpha_L - \alpha_T)\frac{V_x V_y}{V}; \quad V = \sqrt{V_x^2 + V_y^2};$$

where D_M is the molecular diffusion coefficient, α_L the horizontal longitudinal dispersivity, α_T the vertical transverse dispersivity, V the magnitude of the velocity vector and S_0 is the porosity of the aquifer [dimensionless].

With these definitions, contaminant transport can be described in terms of its concentration $c(x, y, t)$ satisfying:

$$S_0\frac{\partial c}{\partial t} = \frac{\partial}{\partial x}\left(D_{xx}\frac{\partial c}{\partial x} + D_{xy}\frac{\partial c}{\partial y}\right) + \frac{\partial}{\partial y}\left(D_{yx}\frac{\partial c}{\partial x} + D_{yy}\frac{\partial c}{\partial y}\right) - V_x\frac{\partial c}{\partial x} + V_y\frac{\partial c}{\partial y}. \qquad (3)$$

To solve Eqs. (1) and (3), the initial and boundary conditions must be specified in the area of interest in the

domain (Fig. 1). In the most common investigations, the boundary (vU) of the fluid region is fixed. This is not true in this problem because we have an unconfined free surface in the dike, which is critical to our investigation.

In an unconfined aquifer, the physical problem is different for the upper "free" boundary (G_{up}). On this "free" boundary, we need to satisfy both a dynamic condition, for the flow pressure to be equal to the outside atmospheric pressure, i.e. $h = y$, and a kinematic condition, which represents the mass conservation law on the free surface (see Appendix):

$$S_y \frac{\partial f}{\partial t} + \bar{u}\frac{\partial f}{\partial x} - \bar{v} = 0: \qquad (4)$$

Here, $\bar{u}; \bar{v}$ are the flux vectors for flow for free boundary $y = f(x; t)$ and S_y is the effective porosity of the media. In our study, we did not assume the hysteresis effect in saturated–unsaturated boundary movements.

For the purposes of this work, the zone of partial saturation is neglected. The practical problems of interest involve relatively high permeability, low capillarity media for which a sharp boundary between saturated and unsaturated regions is a good approximation.

To solve this problem, the Finite Element Method (FEM) will be used with a discretization of the original problem using Eqs. (1) and (3) (Cook et al., 1989; Zienkiewicz and Taylor, 1994; Schiesser, 1991; Burnett, 1988; Lapidus and Pinder, 1982; Pinder and Gray, 1977). The finite element solution for transient analysis introduces a function that is time dependent $H = H(t)$ for the head. Typically, time integration is performed with a finite difference approximation. We used variable-order variable-stepsize backward differentiation formulas with a preconditioned Krylov iterative method (Saad and Schulz, 1986; Brown et al., 1994).

The objective of solving the finite element equations is to compute the head at each node in the domain. When the material properties are constant, the nodal head can be computed directly by linear analysis. However, when the material hydraulic conductivity is a function of the head, the correct material properties are not known at the beginning of the analysis; consequently, an iterative scheme is required to solve the equations, which are nonlinear.

The simplest technique uses a repeated substitution in the iterative process. For the first iteration, the specified initial heads are used to define the material properties. The material properties are updated in subsequent iterations using the computed head from the previous iteration. For transient analyses, the head at the midpoint of the time interval is used to define the material properties, that is, the material properties are defined at the average of the previous and current computed heads. The iterative process continues until the iteration number reaches a maximum specified number or the results satisfy the convergence criteria. The Euclidean norm of the change in the total head vectors H between consecutive iterations is used as a measure of convergence. The vector norm of the residual changes is defined as:

$$\|\Delta H\| = \left(\sum_{j=1}^{n} \Delta H_j^{\,2} \right)^{1/2}:$$

This residual is a measure of the total difference between iterations. In a normal convergence process, this residual will decrease and approach zero. The solution is assumed to have converged when the residual is less than a pre-specified tolerance ε_H.

Frind and Pinder (1982) and Frind (1988) proposed a method where the natural boundary term is solved; then, the dispersive and advective mass fluxes are determined at the exit boundary. This result is referred to as a free exit boundary. We used this method to solve the natural boundary term.

With a free exit boundary, the nodes along the exit boundary are not like ordinary boundary nodes. Instead, they behave like internal nodes, making it possible to compute the concentration at these nodes without assuming a known concentration for the mass flux at these nodes. The free exit approach does not force the concentration gradient to zero as with a zero dispersive mass flux. The solutions for these two methods with different exit boundary conditions are practically the same, except very near the exit boundary, i.e. specified on the (left) side of the flow problem.

3. Results

3.1. Hydraulic head and Darcian flow problem

As an example of the application of the described methodology, the self-consistent estimation of the flow region and flow characteristics governed by Eqs. (1) and (4) is considered. First, we considered a flow boundary development without the tide; this produced no changes in the level of either reservoir. Next, a dynamic transformation and repositioning of the interior in-dike flow boundary due to the presence of a tide and corresponding changes of the free water levels in the surrounding reservoir are considered.

The iterative solution procedure follows: fix the current in-dike interior flow boundary position, $y = f(x; t_0)$, and formulate (apply) appropriate boundary conditions on this surface (in this case—a curved line) at the atmospheric pressure exposed boundary. This keeps the porous water pressure zero $P|_G = 0$ on this boundary or the total head equal to elevation y

(boundary PQ). This assumes an impermeable flow boundary $\partial V/\partial n = 0$. On the wetted portions of the dike boundary (Fig. 1), we have known values for the total hydraulic heads $h = H_L$ on the left side and $h = H_R$ on the right side. The additional seepage area of the boundary QP remains at $P = 0$. These flow boundary descriptions are significantly different from the fixed problem. In this problem, there is an additional highly nonlinear dependence for the head on the dynamic flow area. Given the geometry and boundary conditions, the hydraulic head problem (1) is solved yielding the time dependent head $h(x, y, t)$.

One solution step solves the 1-D moving-boundary problem and determines a new boundary position, which is consistent with Eq. (4) and corresponds to an initially "fixed" normal flux $\partial V/\partial n = 0$. By its nature, Eq. (4) is the 1-D transport equation that governs the transport of the boundary $y = f(x, t_0)$ into its new "position" $y = f(x, t)$. The function $f(x, t)$ is unknown, and will be the consistent solution at time t_0 of the coupled problems (1) and (4). The solution of this coupled problem involves the calculation of the movement of the boundary to compensate for the normal velocity and corresponding mass flux through the boundary. This new boundary redefines a new head distribution and when internal stabilization is achieved, the time step Δt is completed.

The numerical solution of the 1-D transport problem is based on a 1-D finite element method (FEM) approximation of Eq. (4) with boundary conditions as follows: fix the function $f(x, t)$ value at the side where head is independent of time and use the slope of the previous iteration. We used second order Lagrange elements as the FEM shape functions. The formulated problem has some additional numerical difficulties; its coefficients, the boundary values of Darcian velocities $\bar{u}, \bar{v}$, were obtained after differentiation of the hydraulic head with respect to x and y. We can guarantee the existence of these derivatives for an approximated FEM solution, but in general, they do not have continuity. This results in a hyperbolic partial differential equation with discontinuous coefficients. We used piecewise third degree polynomial spline approximations for the normal Darcian velocity field to smooth the predicted boundary. We estimated the mass flux across the previous boundary at the previous time step (when this boundary was consistent with the geometry) to reposition the boundary.

The next iteration step interpolates the previous head solution into a new flow area and solves the head distribution problem with a new flow boundary that has the pore water pressure equal to zero $P = 0$. This is the condition on the in-dike moving boundary PQ. This iteration is repeated until acceptable convergence of the algorithm occurs.

Fig. 2 represents consistent solutions of coupled flow-boundary problem for porous unconfined flow. In this case, with predefined geometry for the dike and hydraulic parameters for the moving-boundary, the results are significantly different from the case when the boundary for the flow is given by the Boussinesq approximation. As a comparison, the Boussinesq approximation solution in a Dupuit–Forschheimer approach (Bear, 1979; Freeze and Cherry, 1979) was used to determine an unconfined aquifer boundary. In the Boussinesq approximation, the change in water volume associated with change in aquifer thickness (or water table level) follows from the definition of the apparent specific yield, and with a horizontal flow prevalence governed by the nonlinear Boussinesq equation for aquifer thickness h:

$$S_y \frac{\partial h}{\partial t} = K_{xx} \frac{\partial}{\partial x}\left(h \frac{\partial h}{\partial x}\right) \quad (5)$$

The steady-state solution of this equation in 1-D immediately follows and has a parabolic profile:

$$h^2 = h_R^2 + \frac{h_R^2 - h_L^2}{L}(L - x)$$

where h_L and h_R are the lake and CDF open water levels and L is the horizontal distance between point R and point Q.

In the numerical solution, flow conditions satisfy boundary conditions $P = 0$ on PQ and then remain as a flow line with the condition $\partial V/\partial n = 0$ on PQ. In the Boussinesq approximation, the head equipotential lines are not normal to the boundary, so a nonzero normal flux $\partial V/\partial n = s \neq 0$ exists on the boundary. It is impossible to meet both dynamic and kinematic conditions on the boundary. The Boussineq approximation

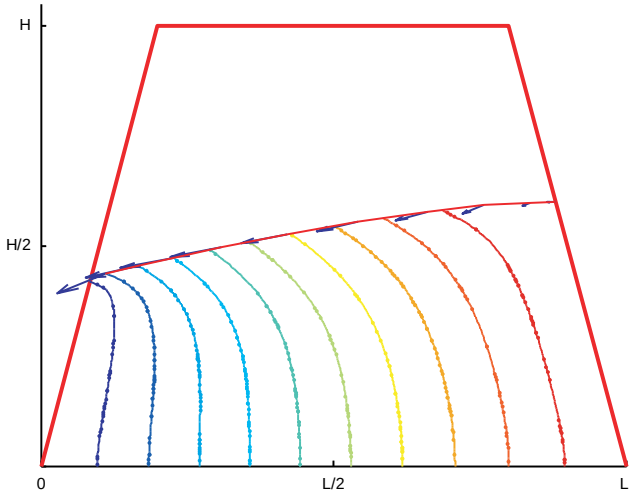

Fig. 2. Head distribution equipotential lines in a consistent moving-boundary approximation. On the in-dike boundary we have relatively accurate approximation for condition $\partial V/\partial n = 0$.

solution also incorrectly estimates the area incorporated within the flow.

3.2. Tide

The effects of the tidal fluctuation were evaluated in the Chicago CDF for which water levels inside and outside the retaining dike have been measured (USACE, 1989). This motion has a characteristic delay in the CDF level with respect to the lake. The amplitudes are practically the same in both reservoirs. The major results are in the behavior at near equilibrium; these results will be discussed further.

3.3. Soluble contaminant transport

The contaminant transport process analysis is a separate calculation from the hydraulic head with moving boundary for each time step. For every time step, a flow domain $U(t)$ and hydraulic head distribution $h(x, y, t)$, $U(t)$ are consistent and form a coupled problem. With non-stationary modeling, a dynamic flow domain $U(t)$ depends on time, and non-stationary Darcian velocity field $V(t)$, $K_{xx}\partial h/\partial x$; $K_{yy}\partial h/\partial y$. Additional iterations need to be performed for each time step to solve the transport Eq. (2). To solve this problem the steps followed were: first, the geometry was fixed, then a time-step Δt with this geometry was computed so that calculated time-dependent head distribution was consistent within acceptable tolerance. This problem with Eq. (2) is a dispersive–convective problem with a time- and geometry-dependent convective velocity field $K_{xx}\partial h/\partial x$; $K_{yy}\partial h/\partial y$. The concentration variation for this time step is then determined. When we begin the next time step, the previous concentration distribution is treated as the new step initial concentration distribution for the dispersive–convective problem. The mass balance between the two main phenomena – substance dispersive flux $Q_d = -S_0 D \operatorname{grad} C$ and convective flux $Q_a = C V$ in general govern the underlying physics for the moving media transport process. Convective flux in uncompressible flow with $\operatorname{div} V = 0$ produces a non-divergent advective term $V \operatorname{grad} C$ in the mass balance Eq. (2). The flux Q_a is defined by the flow characteristics and is responsible for the direct transport of concentration. Its contribution to the total mass balance is proportional to the concentration and is important in the high concentration gradient areas of the system. Diffusive flux Q_d depends on the concentration gradient and in the mass balance this appears as a negative divergence of the dispersive flux $\operatorname{div}(-S_0 D \operatorname{grad} C)$. In this, Eq. (3) is a non-divergent form of the convective–dispersive equation. The diffusion-like process depends on the dispersion tensor D in Eq. (3). In Eq. (3), both the dispersion tensor's symmetric and antisymmetric parts depend upon the Darcian velocities. So, in the flux term $S_0 D$, Darcian velocity V Eq. (3) for the porous media was used instead of the water velocity $U = V/S_0$.

3.4. Boundary conditions specification

The "free" boundary condition $(-S_0 D \operatorname{grad} C)/n = 0$ was employed which implies that the transport by diffusion in the direction perpendicular to the boundary can be neglected, i.e., transport in or out of the system is dominated by convection. This boundary condition was applied to the dike walls exposed to both the lake and CDF. The dike foundation and at the in-dike free surface boundary are treated as impermeable boundaries with the dynamic condition $(n \cdot V)|_S = 0$ and no dispersive flux $(-S_0 D \operatorname{grad} C)/n = 0$ through the boundary. Fig. 3a and b provide a comparison of the influence of the surrounding reservoir level changes with respect to interior dike dynamics. Fig. 3a presents the concentration distribution profiles for the localized initial contaminant at the CDF side. Fig. 3b presents

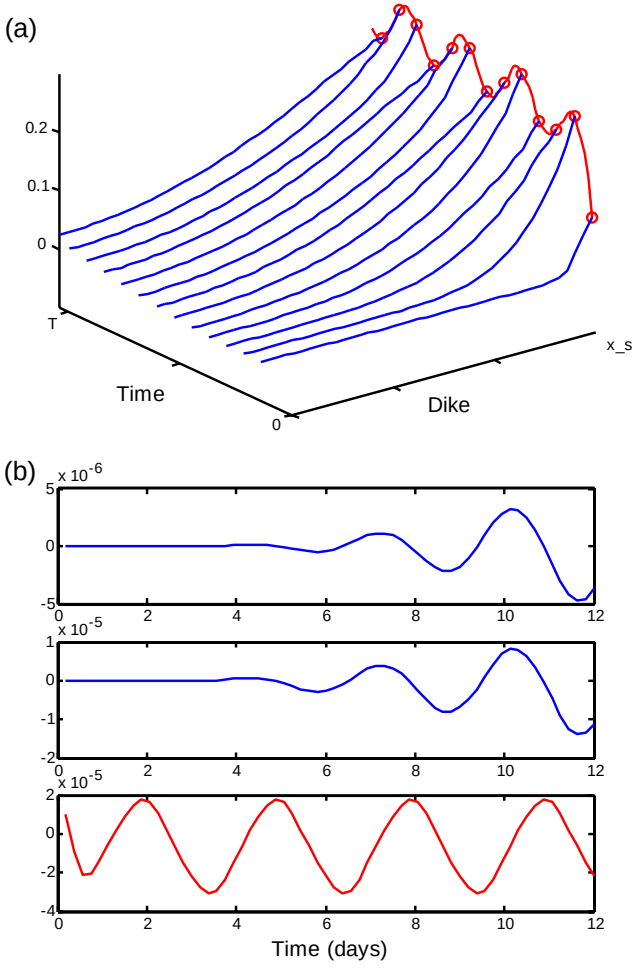

Fig. 3. Time dependent concentration distribution (a) mass fluxes (b) at the lake boundary, dike center and CDF boundary, respectively, (for a 1% tidal amplitude) $R_{bnd}(C \cdot V \cdot n)$.

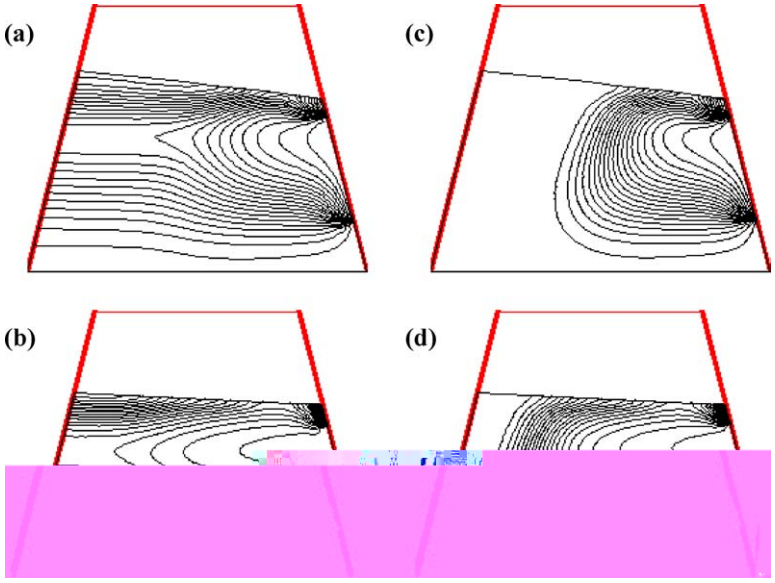

Fig. 4. Dynamic concentration distribution in a several 10%-tide phases: (a) and (b) soluble transport model (grad $C \neq 0$ on the contact boundary); (c) and (d) model the undissolvable case ($C = 0$).

time versus dispersive flux, $\int_l \delta S_0 D \operatorname{grad} C \cdot n \, dl$, through the lake boundary, center and CDF boundary.

Fig. 4 presents predicted concentration distributions with the tidal fluctuations shown in Fig. 3. As seen on Fig. 4a–d, these distributions differ significantly. The time-averaged solution is not identical to the case of average tide (i.e. no tide) because there is a velocity field distribution with the time-changing geometry. The moving boundary behavior inside the dam has a complicated non-symmetric dynamic profile in time. There is no reasonable linear flow approximation, even in the area where the disturbance in the velocity field is not large and the flow is practically horizontal. The time dependent behavior of the dispersive and convective fluxes as shown in Fig. 3 can be used to estimate the time lag in the seepage on the lakeside and compared to the average flux values on the lakeside. The convective flux estimation and the average behavior of the system

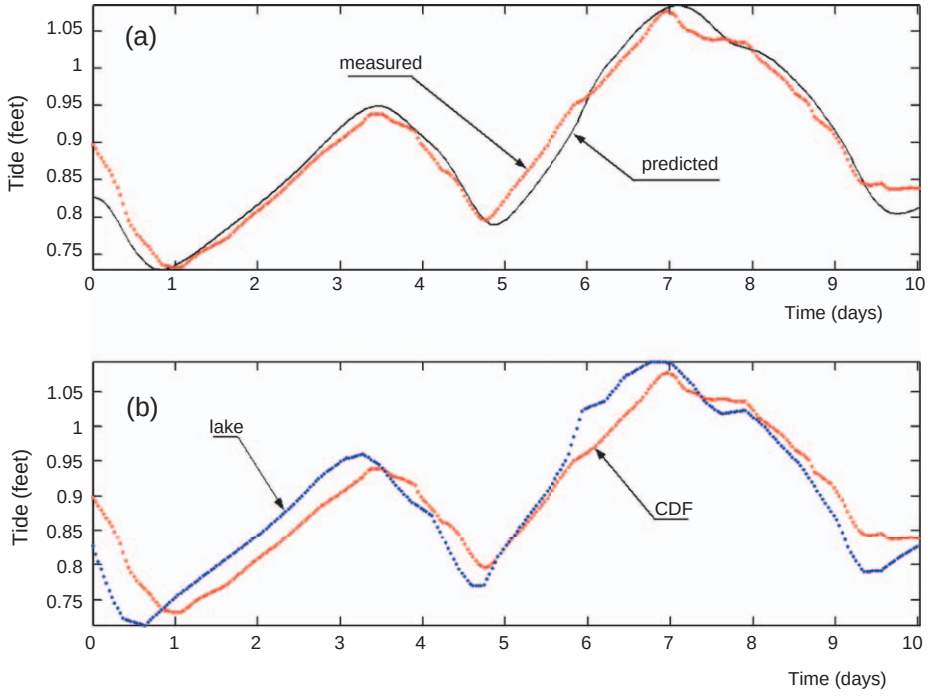

Fig. 5. Measured and predicted flow behavior in a dike. (a) shows CDF level's prediction versus experimental measurements. (b) compares averaged lake measurements and CDF measurements levels behavior.