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An Excavator Bucket

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AIMS & SCOPE OF THE JOURNAL

The *Journal of Dredging* is published by the Western Dredging Association (WEDA) to provide dissemination of technical and project information on dredging engineering topics. The peer-reviewed papers in this practice-oriented journal will present engineering solutions to dredging and placement problems, which are not normally available from traditional journals. Topics of interest include, but are not limited to, dredging techniques, hydrographic surveys, dredge automation, dredge safety, instrumentation, design aspects of dredging projects, dredged material placement, environmental and beneficial uses, contaminated sediments, litigation, economic aspects and case studies.

RECONSTRUCTION OF AND NUMERICAL SENSITIVITY ANALYSIS ON WILSON MODEL FOR HYDRAULIC TRANSPORT OF SOLIDS IN PIPELINES

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ABSTRACT

The Wilson model for the hydraulic transport of solids in pipelines is a widely used model. A theoretical background of the model has been published piece by piece in a number of articles over the years. A variety of information provided in these publications makes the model difficult to reconstruct.

A good understanding of the model structure is inevitable for the user who wants to extend or adapt the model to specific slurry flow conditions. An aim of this article is to summarise the model theory and submit the results of the numerical analysis carried out on the various model configurations. The numerical results show some differences when compared with the nomographs presented in the literature as the graphical presentations of the generalised model outputs. Model outputs are sensitive on a number of input parameters and on a model configuration used. A reconstruction of the nomographs from the computational model outputs is a subject of discussion.

INTRODUCTION

This article contains an overview of a theory for the Wilson two-layer model as it has been published in a number of articles over the years. Results are presented from the model computation. The results provide an insight to the behaviour of the mathematical model. The computation has been completed out using the MathCad document described by the authors in (van Riet et al., 1995).

GEOMETRY OF TWO-LAYER MODEL

A schematic cross section of a pipe is illustrated in figure 1 as it is defined in the two-layer model for the fully-stratified flow and for the heterogeneous (partially-stratified) flow.

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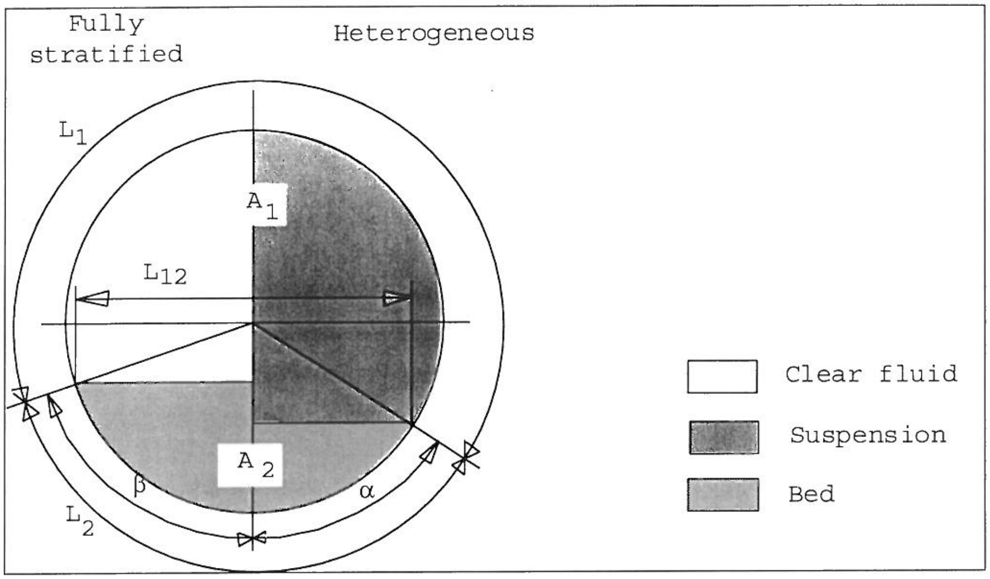

Figure 1: Schematic cross-section for two-layer model.

The geometry of the pipe cross section is defined by the following equations.

The cross-sectional perimeters:

$$L_1 = D(\pi - \beta) \quad (1)$$

$$L_2 = D\beta \quad (2)$$

$$L_{12} = D \sin\beta \quad (3)$$

The cross-sectional areas:

$$A = \frac{1}{4}\pi D^2 \quad (4)$$

$$A_2 = \frac{1}{4} D^2 (\beta - \sin\beta\cos\beta) \quad (5)$$

$$A_1 = A - A_2 \quad (6)$$

The equivalent hydraulic diameter of the non-circular waterway section above the bed is a function of the bed height (Wilson, 1984).

$$D_{eq} = \frac{4A_1}{L_1 + L_{12}} \tag{7}$$

FORCE BALANCE TO DETERMINE THE MAXIMUM DEPOSIT VELOCITY CURVE AND THE RESISTANCE CURVE FOR FULLY-STRATIFIED FLOW

The Wilson model uses the following important parameters for the slurry pipeline design and operation:

- The maximum deposit velocity V_{aMDV} (at the MDV curve). V_{aMDV} is the maximum average velocity of slurry flow in a pipe at which a stationary bed still occurs. The MDV curve depicts V_{aMDV} as a function of the bed height in a pipe.
- The friction loss (at the resistance curve). This curve depicts the pressure drop as a function of the flow rate in a pipe for slurry of the constant delivered concentration of solids.

Both curves can be plotted in one system of coordinates.

The MDV curve and the resistance curve are calculated from a force balance of four main forces (per unit length of the pipe) acting on the stationary or moving bed, which is formed by particles in mutual contact and contact with a pipe wall (Wilson, 1970, 1974, 1976). The force balance is written for forces and shear stresses averaged over the perimeters of flow boundaries.

The shear stresses on the flow boundaries are determined using the Nikuradse friction equation for turbulent flow in a hydraulically-rough pipe (Wilson, 1984):

$$\tau = \frac{V^2 \rho}{\left(2.5 \ln\left(\frac{30 D_{eq}}{8r}\right)\right)^2} \tag{8}$$

The MathCad document solves a set of the model balance equations.

The equilibrium average velocity V_{eq} in the upper layer is obtained by solving V_1 in the force balance while the following quantities are kept constant:

- Bed height
- Bed velocity
- Physical properties of the fluid and solids.

V_{eq} is velocity V_1 for which the force balance is found by iteration in the MathCad document.

The following procedure is used for a model computation:

1. The driving shear force on the bed surface is calculated using the Nikuradse equation multiplied by an empirical constant for shear stress on the bed surface. This constant was originally assumed to be equal to the value 2 (Wilson, 1976).

$$F_{12} = 2L_{12}\tau_{12} \tag{9}$$

   Shear stress τ_{12} is calculated for velocity equal to the difference between the velocity in the upper and in the lower layer.

2. The driving force caused by the pressure gradient over a pipe section of a unit length is determined from the pressure gradient

$$\Delta P = \frac{F_{12} + \tau_1 L_1}{A_1} \tag{10}$$

   and the driving force is

$$F_2 = \Delta P \cdot A_2 \tag{11}$$

3. The resisting mechanical friction force between bed and pipe wall is the normal force exerted by the bed against the pipe wall multiplied by the mechanical friction coefficient μ (Wilson, 1970, Wilson et al, 1992).

$$F_{2d} = g\rho\mu(S_s - S_f)C_b \frac{D^2}{2}(\sin\beta - \beta\cos\beta) \tag{12}$$

4. The viscous friction force between the bed and the pipe wall is calculated

$$F_{2v} = L_2\tau_2 \tag{13}$$

5. The force balance is

$$F_{12}(V_1,...) + F_2(V_1,...) = F_{2d}(...) + F_{2v}(...) \quad (14)$$

6. The relative delivered concentration of solids in slurry flow is determined as

$$C_{del} = \frac{V_2 A_2}{V_{eq} A_1 + V_2 A_2} \quad (15)$$

Relative delivered concentration is a ratio of the absolute delivered concentration and concentration of solids in a loose-packed bed.

Wilson and his co-workers have published the nomographs (Wilson, 1976, 1978, Wilson et al., 1992), the tools to predict the slurry flow parameters without handling the computational two-layer model. The nomographs are based on the computational model outputs. A comparison of the nomographic values with those from a computational model is of interest since it is not always clear for which slurry characteristics (C_b, μ, S_s) and model configuration the nomographs are proposed. The outputs of the computational model have been found very sensitive to the input parameters and a chosen model configuration.

The Resistance Curve

Any point on the resistance curve (i-V_a curve for constant C_{del}) is obtained by a numerical solution of the force-balance equations for the following conditions:

- constant bed velocity
- constant physical properties of the fluid and solids.

The bed height is a variable in a numerical iteration procedure. The bed height is determined for which two criteria are satisfied simultaneously:

- the force balance in pipe section is found
- the calculated delivered concentration equals the C_{del} required by the constructed resistance curve.

The resistance curve computed is presented in the same plot as a nomograph in the literature (Wilson et al., 1992).

The following dimensionless parameters are used in the nomograph:

- relative velocity V_a/V_{max}
- relative concentration C_{del}
- relative excess pressure gradient which is defined as

$$\Delta P_{ex} = \frac{\Delta P - \Delta P_{clear}}{\Delta P_{plug}} \tag{16}$$

when $\Delta P_{clear} = \frac{4.\tau}{D}$ is the pressure gradient of equivalent clear water flow and $\Delta P_{plug} = 2\mu(S_s - S_f)C_b g\rho$ is the pressure gradient for equivalent plug flow.

The MDV Curve

Any point of the MDV curve is obtained by solving the force balance for a given bed height and V_2=0. The curve is produced by solving the balance for an array of bed heights. A maximum at the MDV curve gives V_{max}.

The MDV curve and the resistance curve are plotted in figure 2. This figure is a product of the MathCad document described in (van Riet et al., 1995).

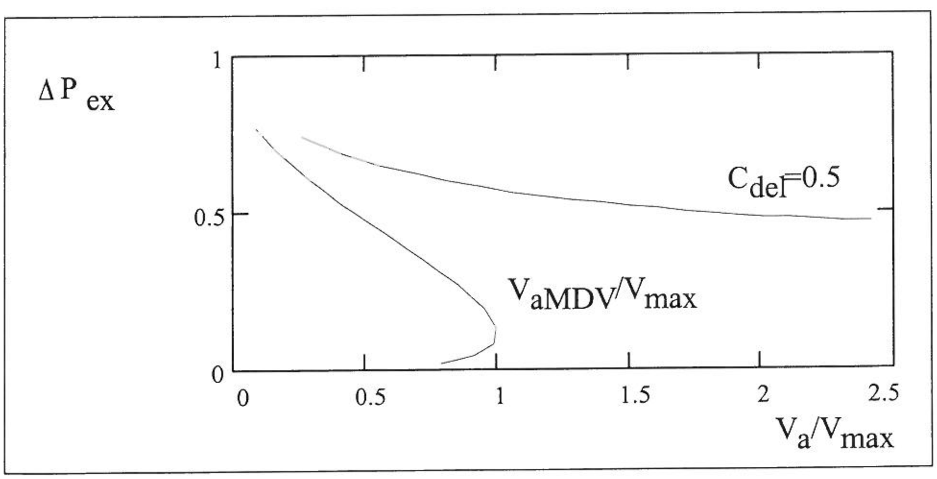

Figure 2: Non-dimensional MDV curve and resistance curve (fully stratified flow).

THE INCORPORATION OF SUSPENSION, HETEROGENEOUS MODEL

An adaptation of the two-layer model has been proposed (Wilson, 1976) for the partially-stratified flow, i.e. flow in which a part of transported solid particles is suspended in the stream above the bed (see figure 1). Suspension of particles due to carrier turbulence causes an increase in the density (and viscosity at the highest concentrations) of mixture flow in the upper layer (Wilson et al., 1980). This change in the physical properties of flow should explain a significant decrease of the V_{max} with decreasing particle size (for particles smaller than approximately 0.7 mm) provided by the curve of the demi McDonald nomograph (V_{max}=f(d, D, S_s)) (Wilson et al., 1978, 1992). Although this decreasing trend can be produced by a numerical simulation of the model (van Riet et al., 1995), it appears impossible to reproduce such a large drop in the V_{max} values as the demi McDonald nomograph gives.

Wilson's (and his co-workers') investigation of the sheet flow has led to a further development in a structure of the two-layer model. Description of the flow in the shear layer, i.e. of the bed-load motion at high shear stress, has provided a new formulation of the friction law for an interface between bed and waterway.

A transition zone between a packed granular bed and water above the bed is called the shear layer. The model may be called 'three layer model' when the shear layer is implemented to its structure. At present the shear layer effect on the model structure is expressed only by an implementation of the new interfacial friction law to the two-layer model so not by changes in the model geometry.

THE THREE-LAYER MODEL

Publications (Nnadi et al., 1995, Wilson et al., 1966, 1984, 1990, 1995, 1995) deal with a description of the shear on the bed-fluid interface. Originally it was assumed (Wilson, 1984) that the hydraulic roughness of the interface is equal to one half of the shear layer thickness. The shear layer thickness is a function of the shear stress at the real/virtual interface. Thus shear stress was determined from a theoretical implicit equation in which the hydraulic diameter D_{eq} was one of the variables.

Later Wilson and Nnadi (1990) derived that the hydraulic diameter can be cancelled from the equations and that the friction factor at the bed surface depends only on i/(S_s-1) providing the following relationship

$$\frac{\delta_s}{R_b} = \left(\overline{C}\tan\phi\right)^{-1}\left(\frac{i}{S_s - 1}\right) \qquad (17)$$

R_b should be determined using a method from (Wilson, 1966). An application of the eq. (7) has led to the following semi-empirical formula expressing a friction law for sheet flow (Wilson et al., 1990)

$$f_{12} = 0.088\left(\frac{i}{S-1}\right)^{0.22} \tag{18}$$

revised in (Wilson et al., 1995) as

$$f_{12} = 0.87\left(\frac{i}{S-1}\right)^{0.78} \tag{19}$$

Eq.(17) has been also implemented in the general friction equation for a rough-wall boundary, that is expressed as:

$$\frac{V}{\sqrt{\frac{\tau_{12}}{\rho}}} = \sqrt{\frac{8}{f_{12}}} = \frac{1}{\kappa}\ln\left(\frac{R_b}{\delta_s}\right) + B \tag{20}$$

Empirical constants in eq. (20) have been determined by a calibration of eq. (20) by the experimental data. Different constants have been published for different data (characterised here by different ϕ):

- equation published in (Nnadi et al., 1995) for ϕ=24°

$$\frac{V}{\sqrt{\frac{\tau_{12}}{\rho}}} = 2.7 - 2.5\ln\left(\frac{i}{S_s-1}\right) \tag{21}$$

- equation published in (Wilson, 1995) for ϕ=18°

$$\frac{V}{\sqrt{\frac{\tau_{12}}{\rho}}} = 2.5\ln\left(2.2\left(\frac{S_s-1}{i}\right)\right) \tag{22}$$

The equations (19, 21, 22) give similar f_{12} values but the eq. (18) differs.

When the recently published value ϕ=14° (Wilson et al., 1995) for a tested material is used in the eq. (20) the following equation can be written

$$\frac{V}{\sqrt{\frac{\tau_{12}}{\rho}}} = 1.2 - 2.5\ln\left(\frac{i}{S_s - 1}\right) \tag{23}$$

Recently, Wilson has proposed a correction of the demi McDonald nomograph based on analytical results from the three-layer model. This has the form of a fit function (Wilson et al., 1992, 1995). The three-layer model outputs have shown that V_{max} is not dependent on the particle diameter when the friction law for sheet flow is used for the interface between layers

$$V_{max} = \sqrt{2gD(S_s - 1)}\left(\frac{0.018}{f_f}\right)^{0.13} \tag{24}$$

Wilson and Pugh (1995) have recommended to use this equation instead of the curve in the demi McDonald nomograph when the value of V_{max} obtained from the demi McDonald nomograph exceeds that from the fit function. The three-layer model has been tested in the MathCad document (van Riet et al., 1995). The V_{max} outputs for various friction equations are compared with the fit function in figure 3. The following input parameters to the model are used: μ=0.4, r=10-5, C_b=0.6, S_s=2.65 and f_f according to Nikuradse.

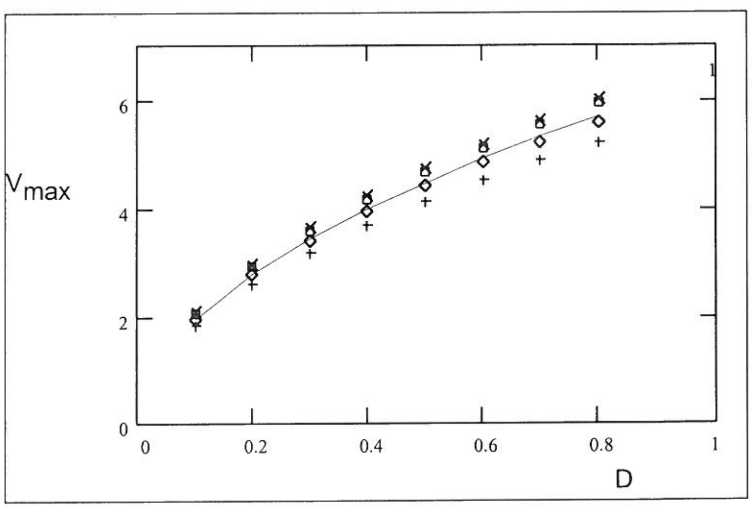

Figure 3: Maximum deposit velocity. Comparison of the fit function with the outputs of the three-layer model for various interface-friction equations. [*Legend*: line = fit function see eq. (24); diamonds = output of the three-layer model with eq. (22); squares = output of the three-layer model with eq. (19); crosses = output of the three-layer model with eq. (21); and plusses = output of the three-layer model with eq. (23)]

The fit function eq. (24) matches reasonably the three-layer model outputs for all tested friction equations. The best fit is reached by the eq. (22). A decrease in ϕ from 29 to 14 degrees causes a decrease in V_{max} of 15 - 20%.

DISCUSSION AND CONCLUSIONS

The theoretical background of the Wilson model for fully-stratified flow, heterogeneous flow and stratified flow with a shear layer has been examined. Model configurations can be numerically analysed in the MathCad document. Examples of the analysis are presented in figures 2 and 3. Issues from an extensive testing are generalized to the following remarks regarding a configuration and an application of the computational model and the nomographs.

The viscous bed-wall friction and horizontal asymptote of resistance curves

It was assumed originally that viscous friction between bed and pipe wall was that for clear water at the pipe wall for the average velocity equal to the velocity of the sliding bed (Wilson, 1976). Then, the graph given in (Wilson et al., 1992) can be reproduced by the outputs of the two-layer model as shown in figure 2.

Wilson and Brown (1982) have later published a method for a determination of viscous friction between sliding granular bed and pipe wall. They compared the viscous friction between a sliding bed and a pipe wall to the friction between a capsule and a pipe wall. According to their analysis the viscous friction factor and wall shear stress should be determined according to the following procedure.

If $Re_2 = \frac{V_2 d \rho}{\eta} < 335$ then

$$f_2 = \frac{22}{Re_2} \qquad (25)$$

If $Re_2 > 335$ then

$$f_2 = 0.033\left(1 + \frac{138}{Re_2}\right)^2 \qquad (26)$$

The shear stress is:

$$\tau_2 = \frac{f_2 \rho V_2^2}{8} \tag{27}$$

When this method is implemented in the computational model, the resistance curve no longer has a horizontal asymptote as shown in figure 4.

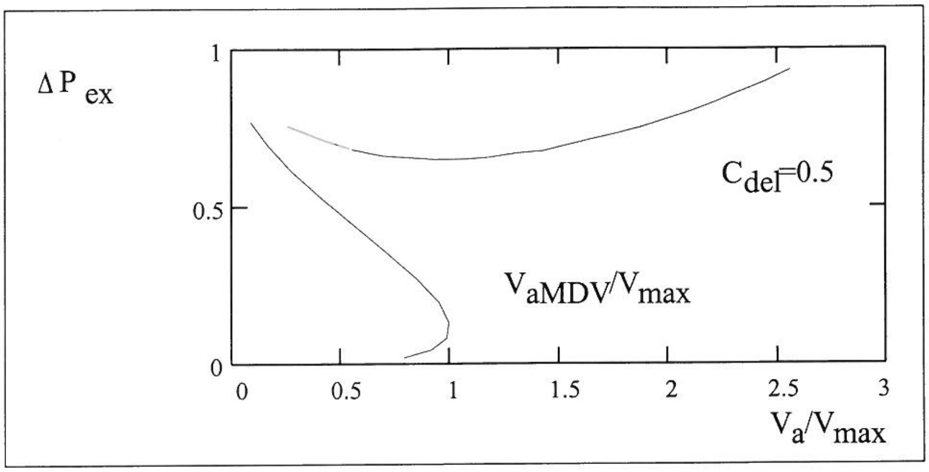

Figure 4: Non-dimensional MDV curve and resistance curve from the model with implemented viscous friction f_2 according to (Wilson et al., 1982) (fully-stratified flow).

Thus, an implementation of this method is not appropriate for the two-layer model. An absence of the horizontal asymptote in figure 4 can be explained from the following. The proposed method provides higher viscous shear stress between bed and pipe wall than is that for fluid. Therefore the ratio V_{eq}/V_2 increases with increasing V_a when the slurry flow is simulated for a given bed height. This results in a decrease in the delivered concentration because all solids are delivered by the lower layer according to the model structure. To maintain a constant delivered concentration (as required by a resistance curve of constant C_{del}), the bed height must increase with increasing V_a. A thicker granular bed provides more resistance and so a higher pressure gradient exists in a pipe.

The cross section between the MDV curve and the resistance curve - zero delivered concentration at the MDV curve

A determination of the MDV curve and the resistance curve in the plot Δp_{ex} vs. V_a/V_{max} (see figures 2 and 4) is based on the fully-stratified flow pattern. It is assumed that no particles are delivered until the average velocity in a pipe exceeds the critical value determined by the MDV curve. In most real flow situations some portion of solids is delivered also at the average velocities below the critical value for which granular bed starts to slide. This is caused by a suspension of particles due to high fluid velocity in the upper layer and/or by a development of a shear layer at the top of a granular bed. Therefore the resistance curves for the low delivered concentrations should cross the MDV curve.

Empirical constant for a determination of the friction factor at the layers interface

Numerical simulations have shown that the multiplication coefficient proposed for the Nikuradse equation to determine the interfacial friction factor does not reproduce the demi McDonald curve. The coefficient equal to 2.75 (instead of 2.00) provides model outputs matching the demi McDonald curve for particle sizes for which the fully-stratified flow is expected (approx. d > 0.7 mm). Even higher value of the coefficient would have to be used to reproduce the demi McDonald curve for heterogeneous flow (a curve section for approx. d < 0.7 mm).

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NOMENCLATURE

A	cross-sectional area of pipe	m^2
A_1	cross-sectional area of upper layer	m^2
A_2	cross-sectional area of lower layer	m^2
B	empirical coefficient	-
$\overline{C}$	volumetric concentration of solids in shear layer	%
C_b	volumetric concentration of solids in the loose-packed bed	%
C_{del}	relative delivered concentration of solids	%
d	particle diameter	mm
D	inside pipe diameter	m
D_{eq}	equivalent hydraulic diameter	m
f_f	Darcy-Weisbach friction factor for fluid flow	-
f_{12}	Darcy-Weisbach friction factor at stratified-flow interface	-
f_2	Darcy-Weisbach friction factor for bed flow	-
F_{12}	driving force on the surface of contact layer	N
F_2	driving force to contact layer due to pressure gradient	N
F_{2d}	mechanical friction force of contact layer against pipe wall	N
F_{2v}	viscous friction force between lower layer and pipe wall	N
g	gravitational acceleration	m/s^2
i	hydraulic gradient	-
L_1	perimeter of pipe between upper layer and pipe wall	m
L_{12}	perimeter of interface between upper layer and lower layer	m

L_2	perimeter of pipe between lower layer and pipe wall	m
MDV	maximum deposit velocity	m/s
ΔP	pressure gradient for mixture flow	Pa
ΔP_{clear}	pressure gradient for clear water flow	Pa
ΔP_{ex}	relative excess pressure gradient	-
ΔP_{plug}	pressure gradient for plug flow	Pa
r	absolute roughness of flow boundary	mm
R_b	hydraulic radius associated with bed	m
Re_2	Reynolds number	-
S_f	relative density of fluid	-
S_s	relative density of solids	-
V	average velocity in waterway	m/s
V_a	average slurry velocity in full cross-sectional area of pipe	m/s
V_{aMDV}	value of V_a at limit of deposition	m/s
V_{eq}	average velocity in upper layer for which force balance is found	m/s
V_{max}	maximum value of V_{aMDV}	m/s
V_1	average velocity in upper layer	m/s
V_2	average velocity in lower layer	m/s
α	angle defining position of surface of real/virtual interface	°
β	angle defining position of surface of contact-load layer	°
δ_s	thickness of the shear layer	mm
η	dynamic viscosity of fluid	Pa.s
κ	von Karman constant	
μ	mechanic friction coefficient of solids against pipe wall	-
ρ	density of fluid	kg/m_3
τ	shear stress at waterway boundary	Pa
τ_1	shear stress between upper layer and pipe wall	Pa
τ_2	shear stress between granular bed and pipe wall	Pa
τ_{12}	shear stress at stratified-flow interface	Pa
ϕ	angle of internal friction of particles (dynamic)	°

MODELING APPROACHES FOR ESTIMATING DREDGED MATERIAL SETTLEMENT – A COMPARISON OF THE PSDDF AND CONDES MODELS

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ABSTRACT

This paper presents a comparison of two modeling approaches for simulating the consolidation and desiccation behavior of dredged material using a case study of Poplar Island in the Chesapeake Bay. As part of the Poplar Island Site Development Plan, it was necessary to accurately predict the initial and final (stable) material elevations in the wetland cells. The U.S. Army Corps of Engineers model PSDDF was first executed for six cases and subsequently, comparative analyses were performed using the computer model CONDES, developed at the University of Colorado, Boulder. The results clearly indicate that both models, PSDDF and CONDES, are able to predict the consolidation and desiccation processes of dredged material. When the same conditions are imposed in both models, the results are quite similar. This is not a surprise for the consolidation part of the process, as both models are based on the same theory. For the desiccation analysis, the crucial parameter is the effective desiccation rate. As long as that rate is the same for both models, similar results should be obtained. The effective desiccation rate in PSDDF is obtained as a result of water balance calculation with a number of empirical factors affecting the outcome. In CONDES, the effective desiccation rate is the top boundary condition and must be specified as input data. The porewater pressure profiles demonstrate that in the CONDES analyses the groundwater table location is obtained as a part of the output and is not an input data, as is in the PSDDF analysis. For CONDES analyses, the site conditions affect only the boundary conditions for the layer, while the quantities within the layer (void ratio and porewater pressure) are obtained in the solution process. Thus, it is shown that CONDES and PSDDF are both numerical models that can complement each other, and may be used as a planning tool in estimating the behavior of dredged material fills.

INTRODUCTION

Poplar Island is located in Chesapeake Bay about 32 miles southeast of Baltimore-Washington International Airport and 35 miles east of Washington D.C (Figure 1). The Poplar Island restoration project was recommended by then Maryland Governor Schaefer's Task Force and involves restoration of habitats lost through the erosion of Poplar Island by the beneficial use of dredged materials from the approach channels to the Port of Baltimore.

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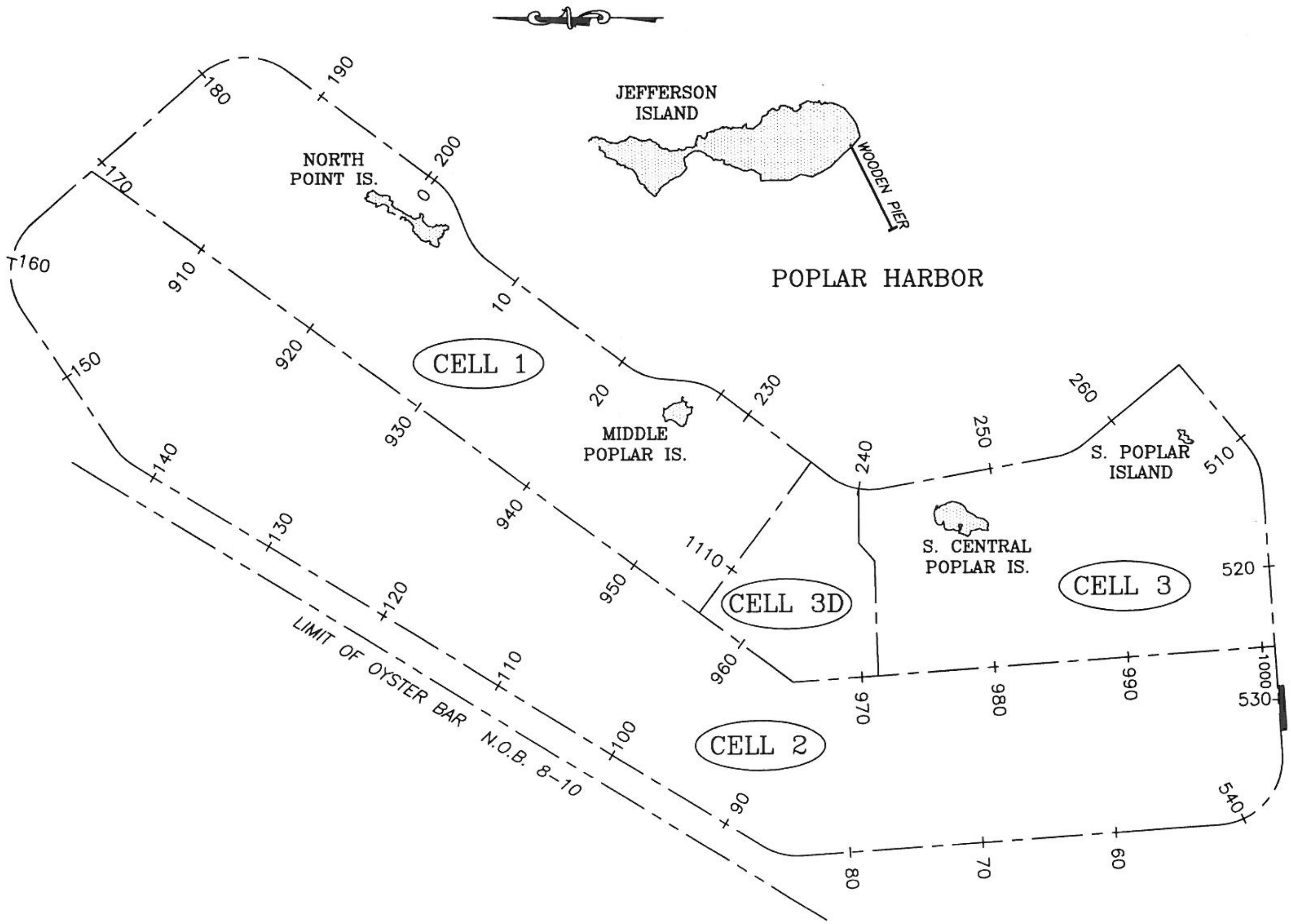

Figure 1. Poplar Island Site Layout

The U.S. Army Corps of Engineers, Baltimore District (CENAB) maintains more than 125 miles of federal navigation channels in providing access to the Port of Baltimore. The concept to reconstruct Poplar Island (to it's near 1890 footprint of 1,142 acres) using clean dredged material was developed by Maryland Port Administration (MPA) in cooperation with CENAB, Maryland Environmental Service (MES), state agencies, federal agencies and private organizations. The restored island will be used as a placement site for clean dredged material from the outer approach channels to the Port of Baltimore. Maintenance dredging of these reaches will generate an average of 2 million cubic yards (mcy) of material annually along with new work projects totaling about 5 to 10 mcy over the next six years. With a projected site capacity of about 40 mcy, the operational life of the Poplar Island site is estimated to be approximately 15 to 20 years, depending on the actual annual yardage placed (GBA & M&N, 1995 and 1996). In April, 1997, MPA entered into a Project Cooperation Agreement (PCA) with CENAB to construct the Poplar Island Restoration Project under the provisions of Section 204 of the Water Resources Development Act (WRDA) of 1992. Due to funding mechanism, the project was divided into two phases: (i) Phase I - 638 acres, and (ii) Phase II - 504 acres. Phase I of the island was constructed in 2000 and Phase II is currently under construction.

As part of the Poplar Island Site Development Plan, it was necessary to accurately predict the initial and final (stable) material elevations in the wetland cells. This was particularly important since the wetland cells are very sensitive to elevation and the successful establishment of vegetation at the site is critically dependent upon the ability to accurately predict the settlement nature of the fill. To address this, the U.S. Army Corps of Engineers model PSDDF was first executed for six cases and subsequently, comparative analyses were performed using the computer model CONDES, developed at the University of Colorado, Boulder. In the PSDDF analyses, the desiccation was modeled simultaneously with the consolidation, but the settlement results are presented separately for each process. In the CONDES analyses the settlements from each component cannot be separated in a single run. Thus, in order to facilitate the comparison, CONDES analyses were performed initially for consolidation only, and subsequently for consolidation and desiccation simultaneously.

Details of the models and results obtained are presented in subsequent sections.

THEORY AND FORMULATION FOR THE TWO MODELS

PSDDF Model

PSDDF is a microcomputer program developed by the U.S. Army Corps of Engineers (USACE) Waterways Experiment Station (WES) that accounts for the Primary Consolidation, Secondary Compression and Desiccation of Dredged Fill (Stark, 1994). PCDDF simulates the primary consolidation and desiccation processes in fine-grained soils using the finite strain theory of consolidation (Gibson et al., 1967) and an empirical desiccation model (Cargill, 1985). The model computes the total settlement of a dredged fill layer based on the consolidation characteristics of the soils above and/or below the layer, the consolidation characteristics of the dredged fill, local climatological data, and surface water management techniques within the containment facility. This settlement is then accumulated for each compressible layer within the area and a cumulative settlement for all dredged material and compressible foundation layers is calculated. Additional layers of dredged material can be added at any time during the simulation.

The model solves for the following governing equation of the consolidation process (Gibson et al, 1967):

$$\{(\gamma_s/\gamma_w)-1\}\ d/de\ \{k_e/(1+e)\}\ de/dz\ + d/dz\ \{ke/[\gamma_w(1+e)]\}\ (d\sigma_s'/de)\ (de/dz) + de/dt = 0$$

where, γ_s is the unit weight of solids, γ_w is the unit weight of water, e is the void ratio, k_e is the coefficient of permeability as a function of void ratio, z is the vertical material coordinate measured against gravity, σ_s' is the effective stress, and t is the time.

The model uses a finite difference scheme to solve the above equation using the procedure described in Cargill (1983, 1985). Once the void ratio-effective stress and void ratio-permeability relationships are defined, the model computes the void ratio in the consolidating layer at any specified time using an explicit finite difference scheme. The model computes the consolidation of the layer until the start of desiccation, after which it assumes normal consolidation until complete drying has occurred in the dredged material layer or a new material layer is placed. The desiccated layer is assumed to act as a surcharge on the underlying layers. Further details of the model are provided in Stark (1994).

The major input required by PSDDF is the void ratio versus effective stress and void ratio versus permeability relationships which are typically obtained from the results of laboratory consolidation tests on the dredged fill and foundation materials. The specific gravity of solids, initial void ratio and the desiccation characteristics of the dredged material are also required. In addition, climatological data, anticipated dredging schedules and quantities, water table elevation, and drainage characteristics of the containment site are required. The model allows for the bottom boundary of the dredged material and/or foundation to be defined as compressible or not compressible.

The PSDDF output includes the following: a brief description of the model input values, coordinates of the nodal points in the initial dredged material layer; initial void ratio, current void ratio, final void ratio, total stress, effective stress, total pore pressure, static pore pressure, and excess pore pressure at each of the coordinate points; current time for which the output is being given; degree of consolidation; primary consolidation, secondary compression, desiccation, and total settlement; pore pressure at the bottom boundary; and final surface elevation.

CONDES Model

CONDES (CONsolidation and DESiccation) is a PC based computer program, which offers numerical solutions for consolidation and desiccation of soft soils such as soft clay and silt, slurry mine waste, dredged spoils, sludge from water treatment plants etc. It provides crucial information for design and analysis, such as time-settlement, solid content profile, pore pressure distribution, crust and crack development of soft soil under surcharge, self weight consolidation, seepage induced consolation, and desiccation (evapotranspiration). CONDES is capable of simulating various boundary conditions of different loading, staged filling sequence (instantaneous, continuous, and step-wise), and various combinations of dewatering and evapotranspiration schemes.

CONDES desiccation analysis is based on a rational desiccation theory that has been verified in the lab and field experiments. The crust thickness, crack depth and volume, settlement, and solid content profile with time can be estimated using CONDES.

CONDES is developed using finite strain theory with soil constitutive models of compressibility and permeability in the form:

Compressibility $e = A \cdot (\sigma' + Z)^B$

Permeability $k = C \cdot e^D$

The parameters A, B, Z, C, and D are experimentally determined soil characteristic constants. Although these parameters can be interpreted from conventional consolidation test results, an advanced experimental procedure, Seepage Induced Consolidation Test (SICT) and Analysis (SICTA) can be used to directly obtain these parameters.

CONDES is an excellent design aid for soft soil consolidation and desiccation, natural habitat and wetland configuration, and dredged material disposal as well as mining spoil disposal design. Especially when a more stringent settlement criterion is required. Further details of CONDES and its application for dredged fill can be obtained from Gjerapic and Znidarcic (2000a).

MATERIAL AND CELL DESCRIPTIONS

Six cell-filling scenarios for Poplar Island (Figure 1) with varying initial heights, bottom elevations and material properties were used for the analysis. Table 1 presents the run designations and relevant analysis parameters.

Table 1 - Analyses Parameters for Poplar Island Cells

Cell/ Run Designation	Nominal Fill Height (ft)	Fill Time (days)	Bottom Elevation (ft)	Initial Void Ratio	Material Type
1-6	6	35	-4.5	8.14	3
2N-24	24	51	-18.0	5.50	4
2S-15	15	31	-7.0	8.14	3
2S-10	10	31	-2.0	8.14	3
3-9	9	19	-7.5	8.14	3
3D-3	3	7	1.0	8.14	3

The material characteristics were determined previously from the seepage induced consolidation tests and the results are presented in E2CR (2000) and Gjerapic and Znidarcic (2000b). Only materials C3 and C4 from the report are considered in these analyses. Their properties are given by the following expressions:

Material C3

Compressibility	$e = 8.92\ (\sigma'+1.56)^{-0.205}$	σ' in psf
Hydraulic conductivity	$k = 8.16\ 10^{-6}\ e^{4.09}$	ft/day

Material C4

Compressibility	$e = 5.27\ (\sigma'+0.769)^{-0.163}$	σ' in psf
Hydraulic conductivity	$k = 2.88\ 10^{-5}\ e^{4.29}$	ft/day

Note that the values added to the effective stress in the compressibility relations are different from those reported with the test results. They are adjusted here in order to account for the higher initial void ratio expected in the field (when compared to the one tested in the lab). Also note that while these relationships were used in the CONDES runs, the PSDDF analyses used point data for the relationships. While this in general would not have a major impact on the results, it was noted that for the initial void ratio the permeability values used in PSDDF were lower than predicted by the stated relations. This leads to somewhat lower initial settlement rates.

RESULTS AND DISCUSSION

Consolidation Analyses

The results of the CONDES and PSDDF consolidation analyses are presented in Figures 2-9. For each cell the height versus time curve and the void ratio profiles at characteristics times are given. For the cell 2S, two filling heights and three bottom boundary conditions for each were considered.

Figure 2 presents the results for Cell 1 (6 ft lift). The final height after consolidation from CONDES is 3.7 ft (ie, 2.3 ft of nominal settlement), while the results from the PSDDF indicate a final consolidation settlement of about 2 ft. The time to reach the final height is about 24 months, again almost identical to the PSDDF results. The void ratio profile at 2500 days also indicates that the layer is fully consolidated at that time. The only difference between the two analysis methods is in the initial settlement rate that is caused by the stated difference in the hydraulic conductivity at the initial void ratio.

Figure 3 presents the results for Cell 2N (24 ft lift). This is a case where there is a noticeable difference between the results from PSDDF and CONDES. The final height from CONDES is 13.2 ft (10.8 ft of nominal settlement), while the PSDDF predicts a consolidation settlement of only 9.1 ft at that time. This discrepancy is most likely a consequence of a much slower consolidation rate predicted by PSDDF for this case. The CONDES analysis predicts almost complete consolidation in the 70 to 80 months time, while the PSDDF analysis predicts still a noticeable settlement rate at the end of 84 moths. While the difference in the hydraulic conductivity at the initial void ratio could account for the difference in the initial settlement rate, it cannot explain the much slower consolidation in PSDDF. The only possible explanation would be the creation of the desiccated crust in the PSDDF analysis, which would impede the subsequent consolidation. However, much more detailed comparison would be needed before a definite conclusion could be drawn.

Figures 4 to 6 present the results of the CONDES analyses for the Cell 2S with the nominal filling height of 10 to 15 ft. Three scenarios were considered. The first one is with an impervious bottom, which can be compared to the PSDDF run. Again, a reasonable agreement can be noticed even though the final height predicted by CONDES is about 0.5 ft higher than the one predicted by consolidation only in PSDDF (8.2 ft versus 7.7 ft). The apparent faster settlement rate in PSDDF is caused by the presence of the desiccation in the early settlement stage after filling. This aspect will be discussed in the section on desiccation.

An alternative case would be when an under-drain is constructed for the Cell 2S. The pressure head in the drain is assumed to be 7 ft, coinciding with the sea level (zero elevation), while the top boundary is assumed to be covered with water throughout the consolidation process. It is

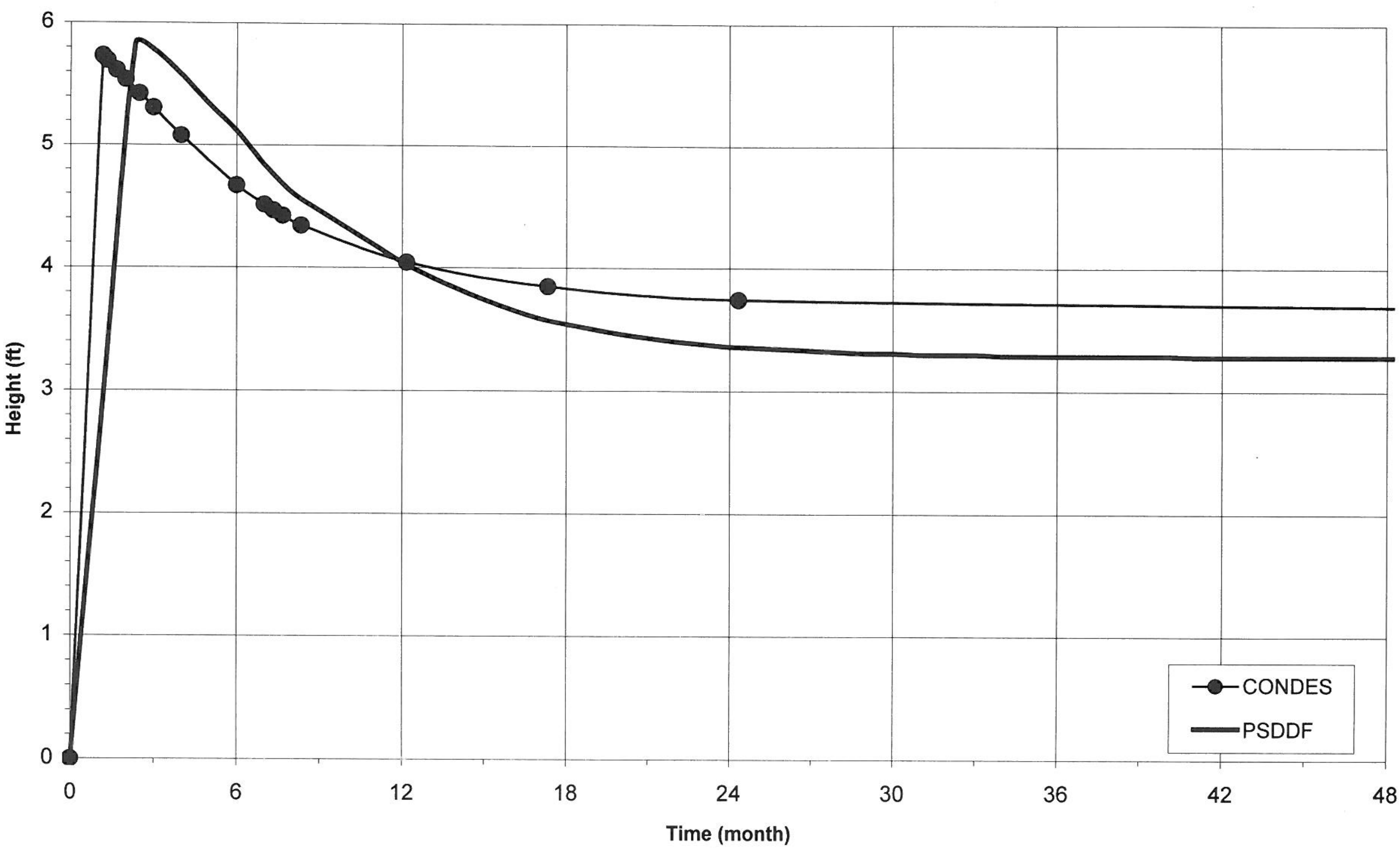

Figure 2. Cell 1-6 - Impervious Bottom, No Evaporation (Ht vs. Time)

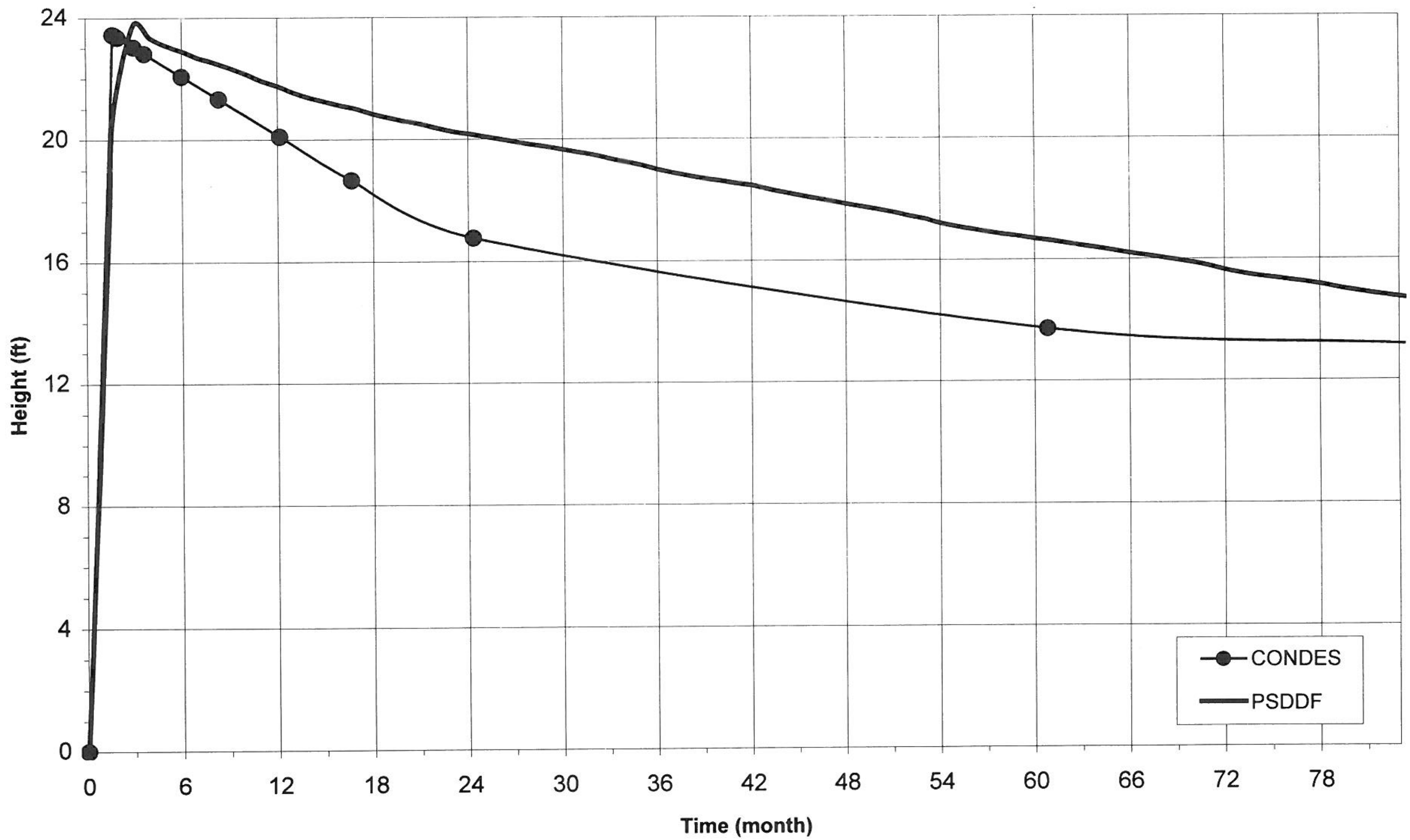

Figure 3. Cell 2N-24 - Impervious Bottom (Ht vs. Time)

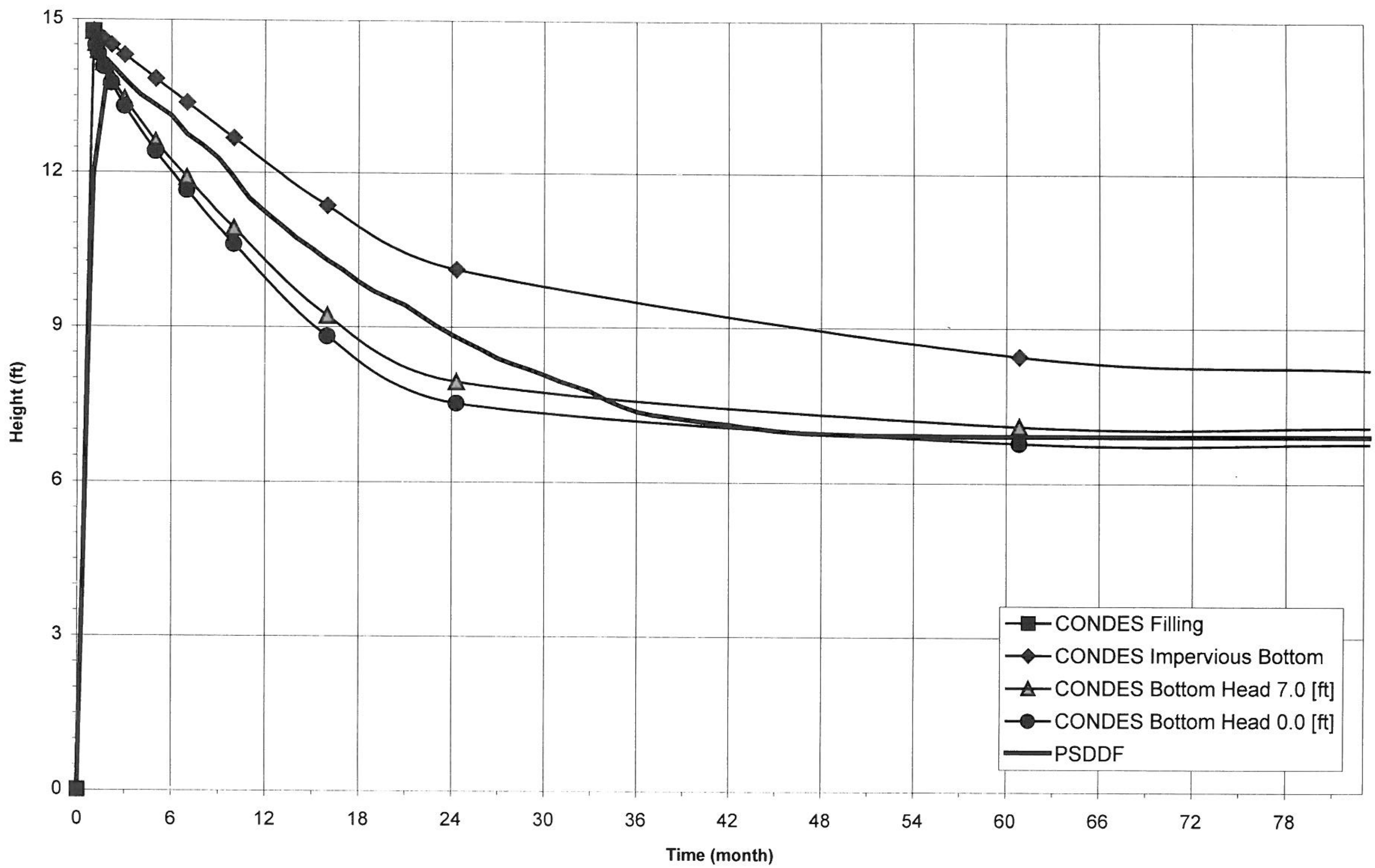

Figure 4. Cell 2S-15 - Various Bottom Boundary, No Evaporation (Ht vs. Time)

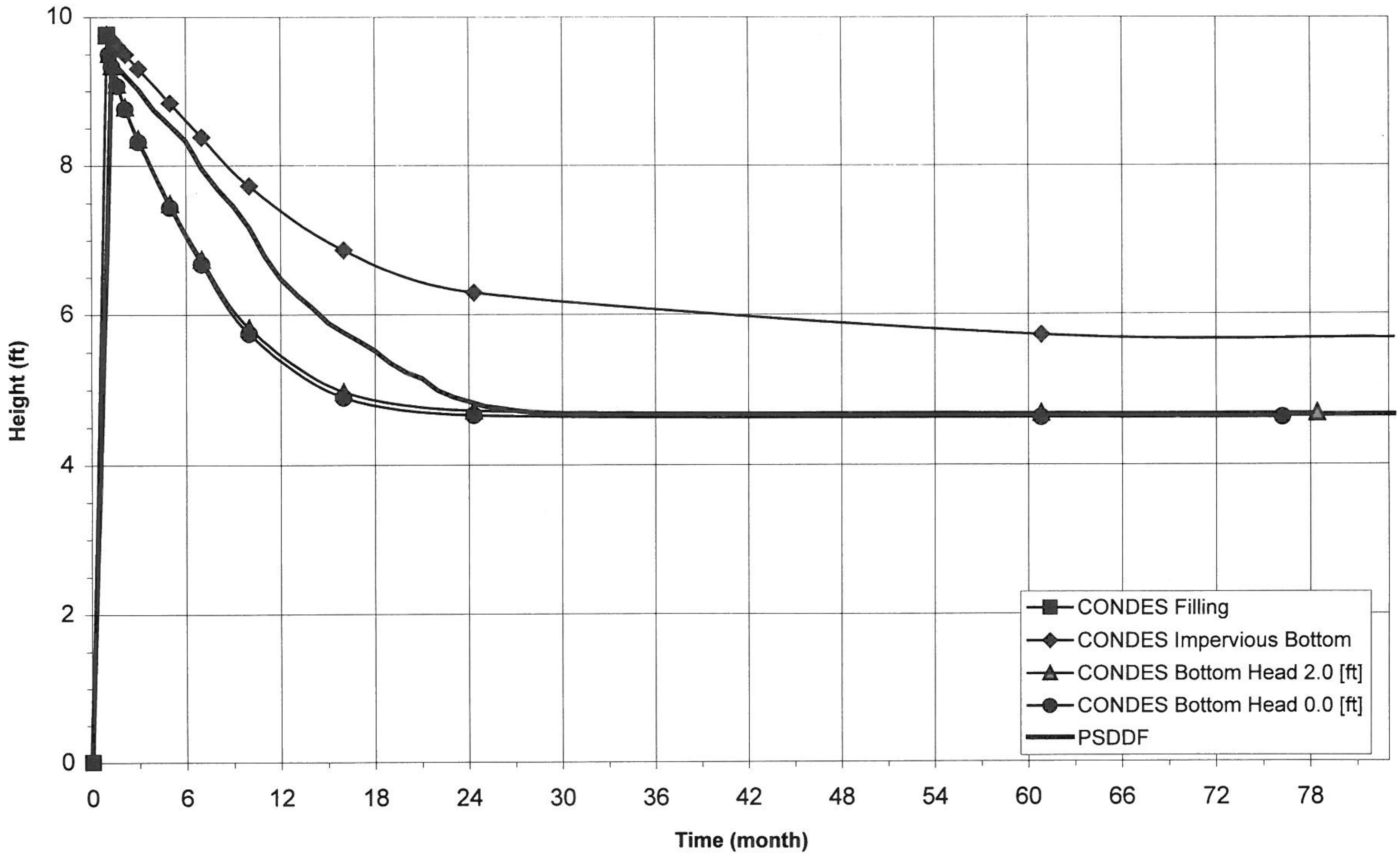

Figure 5. Cell 2S-10 - Various Bottom Boundary, No Evaporation (Ht vs. Time)

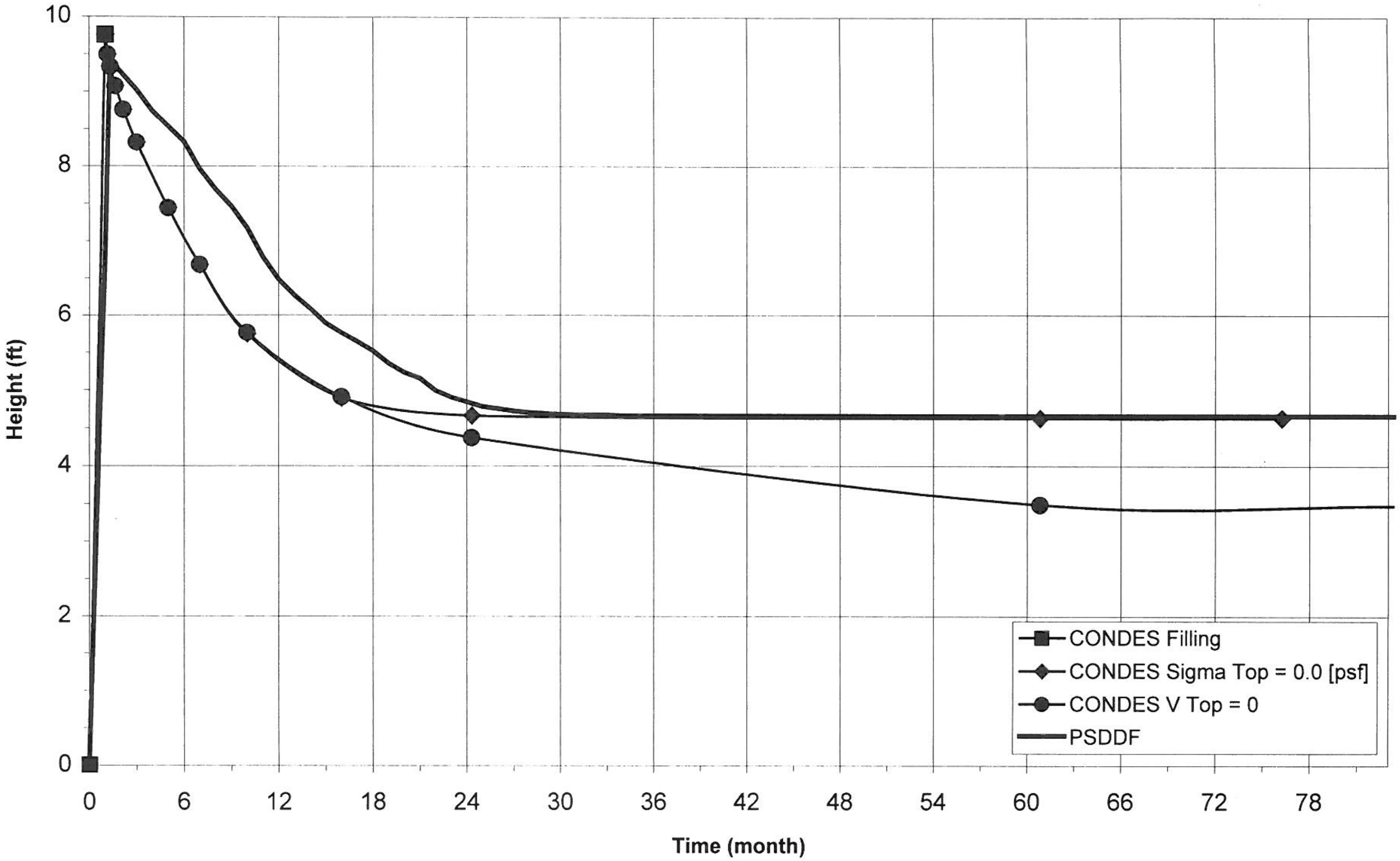

Figure 6. Cell 2S-10 - Various Bottom Boundary, Bottom Head 0 Ft (Ht vs. Time)

recognized that this my not properly represent the field situation as the layer will be deposited in a confined area without the direct contact with open water. The condition, without water at the top of the layer will be presented for the case of Cell 2S, but with the filling height of 10 ft. For the case with free water at the top of the layer, the bottom drainage caused an additional settlement of 1.2 ft (final height of 7.0ft) due to the action of the seepage force in the early stages of the consolidation. It is noted that the top portion of the layer is not affected by the bottom drainage and the maximum void ratio at the surface is the same as in the case of impervious bottom. These results show that the effect of bottom drainage, in addition to faster consolidation, causes the reduction of the void ratio in the lower half of the layer.

A final analysis was performed with the under-drain, but for the case when the pressure in the drain was maintained at the atmospheric level (pressure head equal zero). The results are very similar to the previous ones with the pressure head of 7 ft, since the soil compressibility is low at the effective stress level in the range of 1000 psf, acting at the bottom of the layer. Again, the part of the layer near the surface is not affected by the bottom drainage as the surface is below the sea level. The relatively small difference between the two cases with the bottom drainage but different heads, can be easily explained with the presented results. The induced head difference across the layer and the associated seepage forces cause the additional consolidation. The bottom part of the layer has lower hydraulic conductivity and most of the head loss takes place near the bottom. The upper portion of the layer is not affected much by the seepage forces, and remains at low density. As the head increases, only the bottom portion is affected by the change. As this portion is already stiff, the additional settlement is minimal as the results indicate.

Results for Cell 3 are presented in Figure 7. Again, the final height of 5.3 ft (3.7 ft nominal settlement) from CONDES is in an excellent agreement with the consolidation settlement prediction of 3.8 ft from PSDDF. The consolidation time of 36 months from both analyses is also noted. The only noticeable deviation between the two analyses is the initial settlement rate caused by the already stated differences in the hydraulic conductivity at the initial void ratio.

Results for Cell 3D are presented in Figure 9. The final height of 2 ft from CONDES is exactly as predicted by PSDDF in which analysis the consolidation settlement of 1 ft is obtained. The consolidation times of about 14 months are also identical. The difference in the initial settlement rate is again noted.

Desiccation Analyses

In the PSDDF model, the desiccation effects are accounted for by a water balance approach. Two drying stages are identified. In both stages the pan evaporation potential, rainfall and the consolidation discharge are combined to find the net desiccation. In the first stage the evaporation potential and rainfall are multiplied by empirical factors and when added with the consolidation discharge, the desiccation settlement is calculated. In the second stage the evaporation potential is further modified by the factors accounting for the proximity of the water

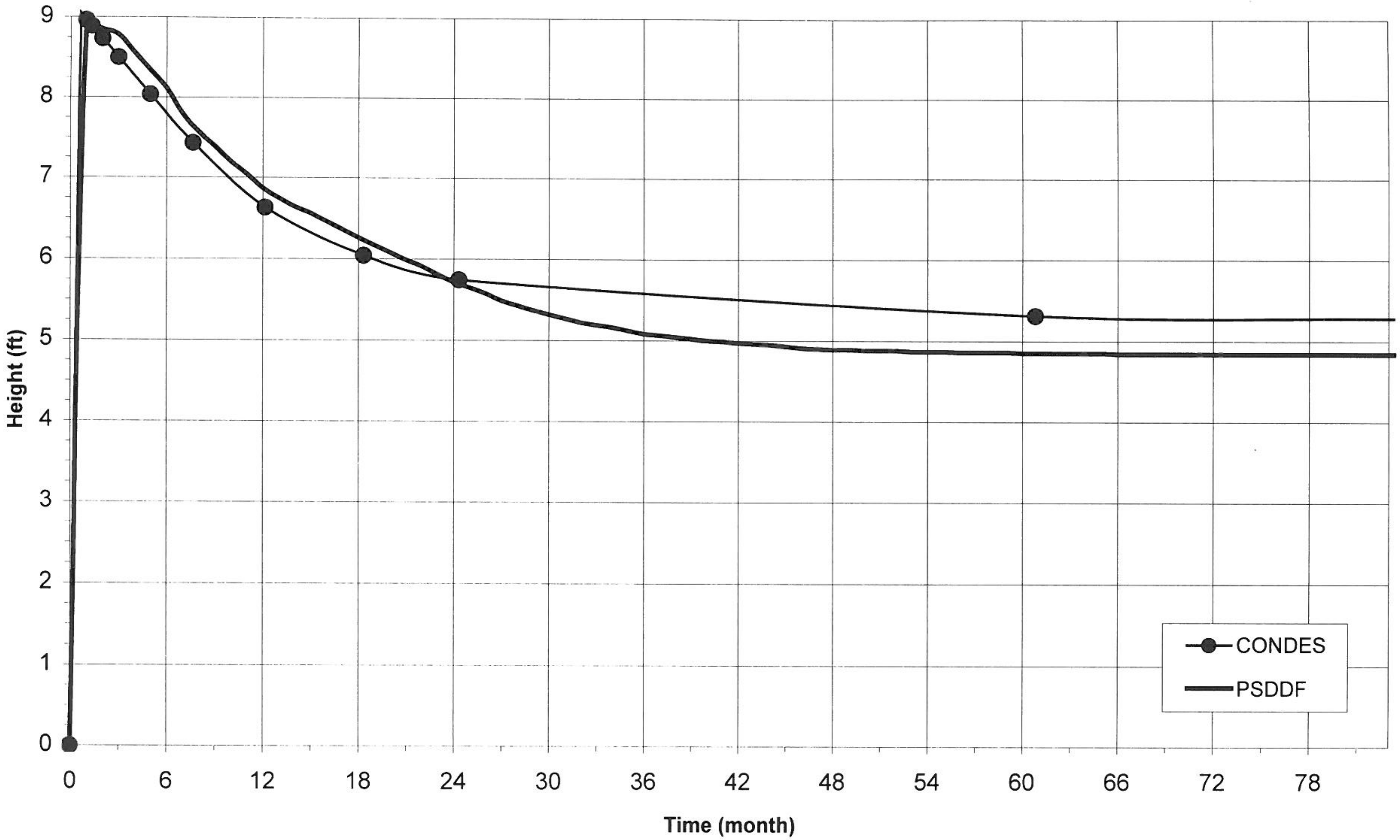

Figure 7. Cell 3-9 - Impervious Bottom (Ht vs. Time)

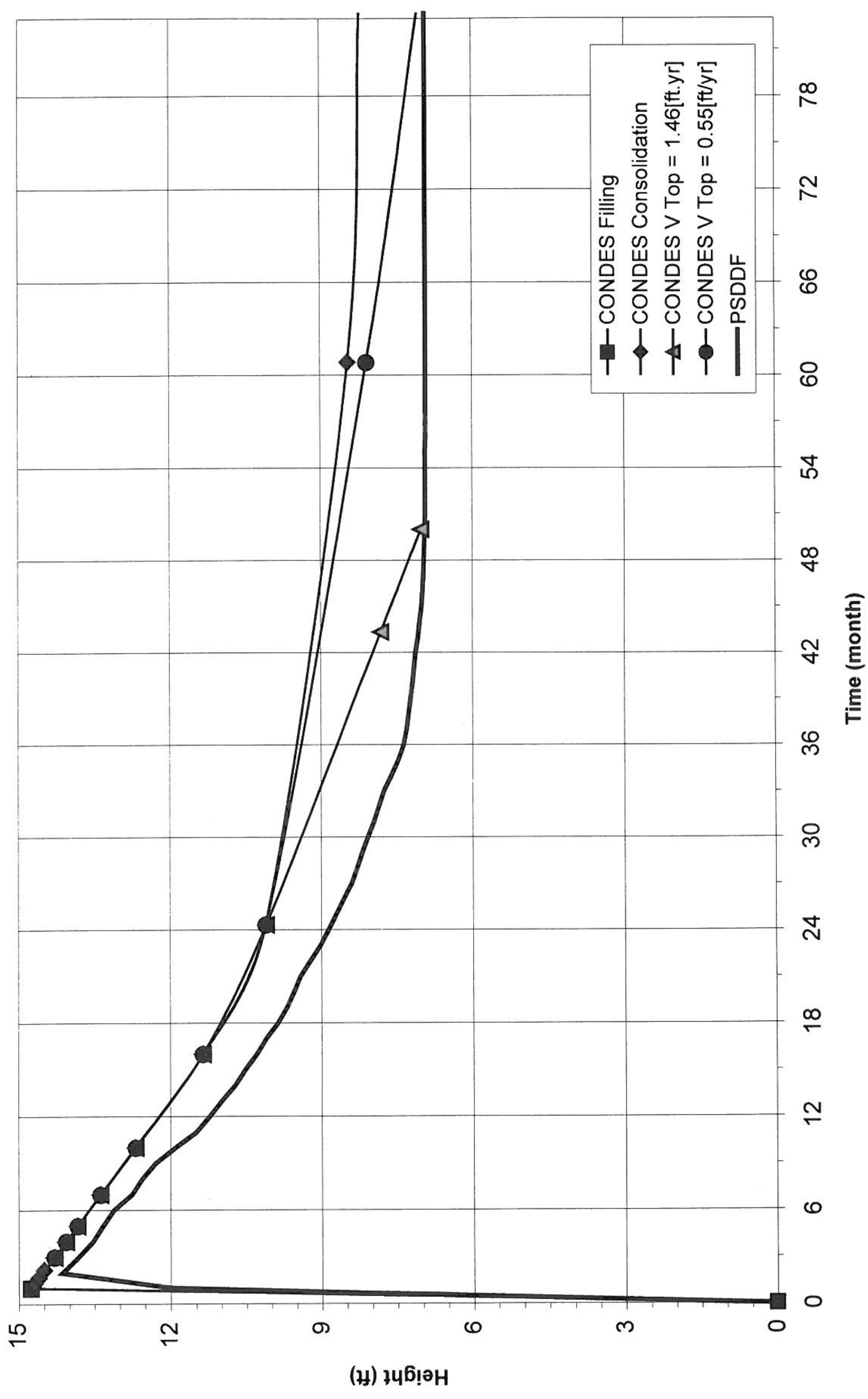

Figure 8. Cell 2S-15 - Impervious Bottom (Ht vs. Time)

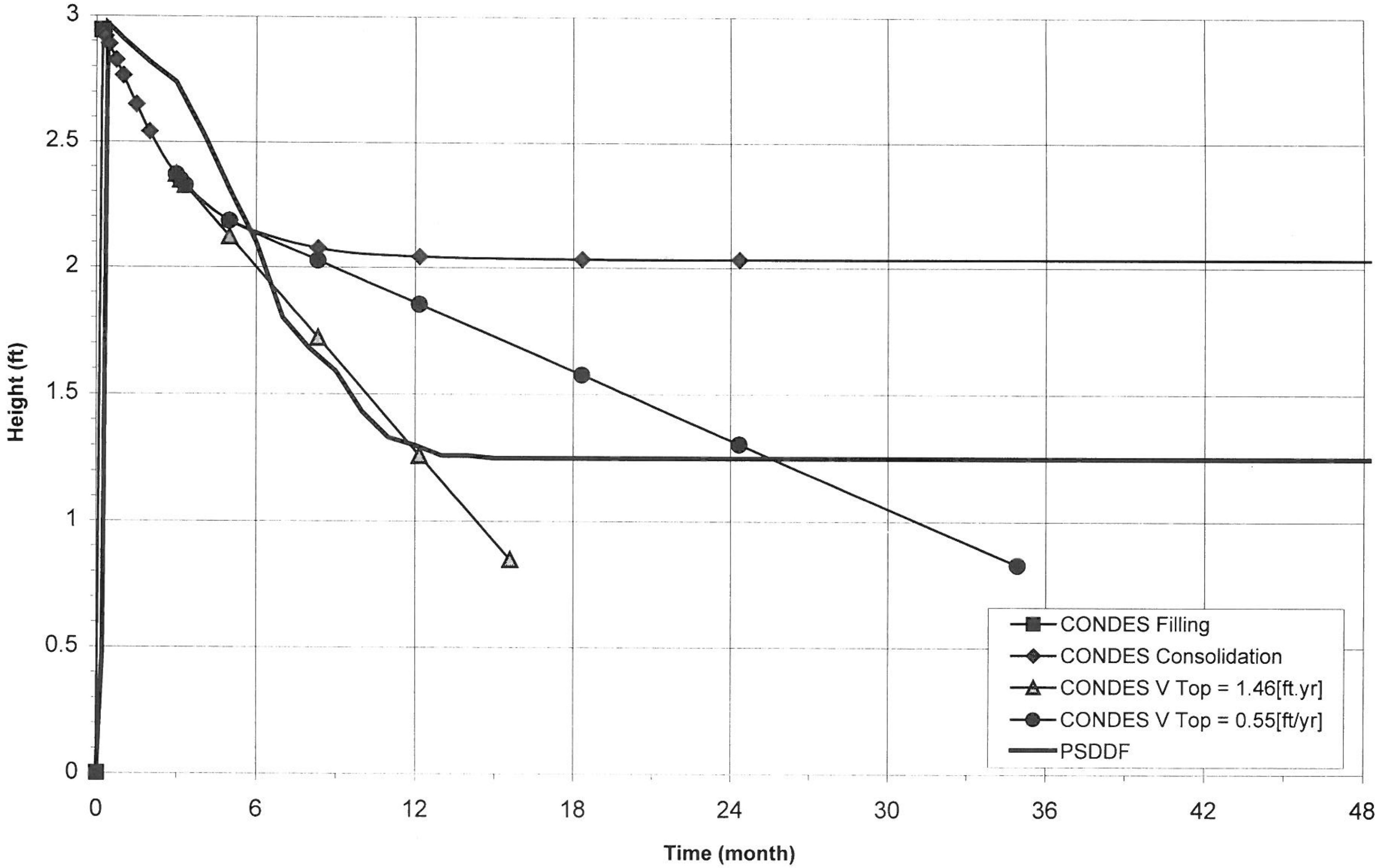

Figure 9. Cell 3D-3 - Impervious Bottom, Min e = 1.5 (Ht vs. Time)

table to the surface of the desiccating layer, by the thickness of the drying crust and by its saturation. The resulting volume change due to the desiccation is then distributed within the drying crust as the change in the void ratio. The desiccation analysis is thus purely empirical in the PSDDF model.

In the CONDES model, the desiccation is an integral part of the solution of the governing equation. It continues without any discontinuity from the consolidation analysis, the only difference being the surface boundary condition. The climatic and drainage factors are considered when selecting the appropriate desiccation rate as the boundary condition. Thus the analysis is rational and the effects of consolidation discharge and the position of the groundwater table are determined as a part of the analysis and do not need to be specified a priory. Thus, the desiccation analysis in CONDES is completely rational and it represents a logical continuation of the consolidation process.

It should be noted that the determination of a realistic effective desiccation rate is essential for a successful desiccation analysis. This is true in both the PSDDF and CONDES models. While in PSDDF the effective desiccation rate is a result of several calculation steps with a number of empirical parameters in each step, in CONDES, the effective desiccation rate is a boundary condition and no additional parameters are needed in the calculation.

For the purpose of this analysis and in order to simulate as closely as possible the same field situation with the two models, the effective desiccation rate was calculated from the desiccation settlements obtained in PSDDF. For each cell the total desiccation settlement was divided by the time over which the desiccation took place. This calculation gave the desiccation rates between 0.0015 ft/day (0.55 ft/year) and 0.004 ft/day (1.46 ft/year) and the CONDES desiccation analysis was performed with these two rates. It is noted that these rates are much smaller than the yearly pan evaporation rate of 4.35 ft/year input in the PSDDF, and certainly smaller than the maximum evaporation rate of 8.16 ft/year reported for the month of July. Also note that the effective evaporation rates are even smaller than the minimum evaporation rate reported for the month of November (2.16 ft/year). However, the selected effective desiccation rates are consistent with our experience in our current field test in Florida, where the effective desiccation rates are only a fraction of the potential evaporation rate at the site.

When the selected effective desiccation rates are applied to a consolidating layer, no desiccation can take place until the consolidation settlement rate drops bellow the imposed desiccation rate. The review of the consolidation settlement curves reveals that for the Cells 1, 2N, and 3 the layer surface dips bellow the sea level while the settlement rate is still higher than the imposed effective desiccation rates. Once the soil surface is bellow the sea level any desiccation stops. Thus, for these cases no desiccation is expected. While the PSDDF analyses show some desiccation effects the associated settlements are small and less than 0.5 ft in all cases. It is noted that if higher effective desiccation rates were selected for the analysis, the CONDES model would also show some, albeit small, desiccation effects. This indicates that for the stated cells minimal, if any, desiccation effects should be expected. The desiccation effects were more significant for Cells 2S and 3D.

CONCLUSIONS

The presented analysis results clearly indicate that both models, PSDDF and CONDES, are able to predict the consolidation and desiccation processes of soft dredged material. When the same conditions are imposed in both models the results are quite similar. This is not a surprise for the consolidation part of the process, as both models are based on the same theory. For the desiccation analysis the crucial parameter is the effective desiccation rate. As long as that rate is the same for both models, similar results should be obtained. The effective desiccation rate in PSDDF is obtained as a result of water balance calculation with a number of empirical factors affecting the outcome. In CONDES, the effective desiccation rate is the top boundary condition and must be specified as input data.

The porewater pressure profiles demonstrate that in the CONDES analyses the groundwater table location is obtained as a part of the output and is not an input data, as is in the PSDDF analysis. For CONDES analyses the site conditions affect only the boundary conditions for the layer, while the quantities within the layer (void ratio and porewater pressure) are obtained in the solution process.

The presented analyses for the Cell 2S with undardrains is one-dimensional and assumes that a blanket drain is constructed at the bottom of the disposal area prior to the filling process. We do understand that the underdrains will be constructed as discrete drains, with certain spacing between them. Strictly speaking the presented analyses would then apply only to the layer above the drain and within its radius of influence. As a one-dimensional model, CONDES cannot be used to analyze 2-D and 3-D cases. However, an approximate solution could be found by combining the CONDES analyses with a two-dimensional seepage model. In such an analysis, the one-dimensional consolidation results will be generated for two bottom boundary conditions, one with the drain, and the other impervious. The spatial hydraulic conductivity distribution within the layer obtained from the analyses will then be input into the two-dimensional seepage model in order to obtain the seepage pattern within the drains. A few iteration steps might be needed to produce a realistic prediction of the material behavior in between the drains.

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