Settlement of Dredged and Contaminated Material Placement Areas. I: Theory and Use of Primary Consolidation, Secondary Compression, and Desiccation of Dredged Fill

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Abstract: A 1D nonlinear numerical model, Primary Consolidation, Secondary Compression, and Desiccation of Dredged Fill (PSDDF), is presented to predict the settlement of fine-grained dredged material and/or underlying compressible foundation materials that may be over-, under-, or normally consolidated. The three most important natural processes affecting the long-term settlement and thus service life of dredged material placement areas are primary consolidation, secondary compression, and desiccation. Nonlinear finite-strain consolidation theory is used to predict the settlement due to self-weight and surcharge-induced consolidation. The C_α/C_c concept is used to predict the settlement from secondary compression, and an empirical desiccation model is used to describe the settlement from removal of water from confined dredged material by surface drying. This paper describes the modifications and improvements of PSDDF that present new functions and enhanced numerical efficiency. A companion paper describes the input parameters of PSDDF.

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Introduction

The U.S. Army Corps of Engineers and other agencies or companies are continually addressing the problem of providing storage for fine-grained material dredged from navigable waterways throughout this country. Increasing environmental concerns together with a general decrease in the number of available placement areas have created the need for maximum utilization of existing and planned dredged material containment areas. Such containment areas are usually filled by hydraulic means (e.g., pumping a slurry of dredged material). To achieve maximum long-term benefits from dredged material placement areas, the design and operation of the areas must accurately account for the increase in storage capacity resulting from sedimentation, primary consolidation, secondary compression, and desiccation of the dredged material following placement. Primary consolidation, secondary compression, and desiccation are accounted for in the microcomputer program Primary Consolidation, Secondary Compression, and Desiccation of Dredged Fill (PSDDF). The sedimentation process is completed shortly after material deposition and therefore is not included in PSDDF because it has little if any effect on the long-term storage capacity of a placement area.

Many fine-grained dredged materials undergo strains greater than 50% during self-weight consolidation. Greater strains are possible if the placement area is managed so that the surface water is removed and desiccation can occur. Predicting the magnitude of strain is just a minor part of the problem; the major problem is predicting the time rate of settlement for dredged material subjected to the effects of (1) self-weight consolidation, (2) secondary compression, (3) crust formation caused by desiccation, and (4) additional consolidation due to the surcharge created by the crust. In addition, the time rate of settlement for dredged material placed underwater for storage or land creation purposes is also of interest. Settlement of submerged dredged material is subjected to the effects of (1) self-weight consolidation, (2) surcharge induced consolidation usually by placement of a less compressible cohesionless soil cover (sand layer), and (3) secondary compression. This model can also be used to predict the time rate of settlement of contaminated bottom sediments previously placed or after remedial capping.

PSDDF has been modified and enhanced from the original versions such as PCDDF (Primary Consolidation and Desiccation Dredged Fill) by Cargill (1985) and PCDDF89 (Stark 1991). Major improvements made in PSDDF from the prior versions are (1) consideration of secondary compression, (2) ability to predict settlement of over- or underconsolidated compressible foundation materials, (3) ability to provide a profile of consolidation-induced advection and solids density at any time step for the evaluation of contaminant advection using the Capping Analysis Program (CAP) (Ruiz et al. 2003), (4) adjustment of the initial void ratio to the void ratio at zero effective stress in the void ratio-effective stress relationships, (5) consideration of less compressible cohe-

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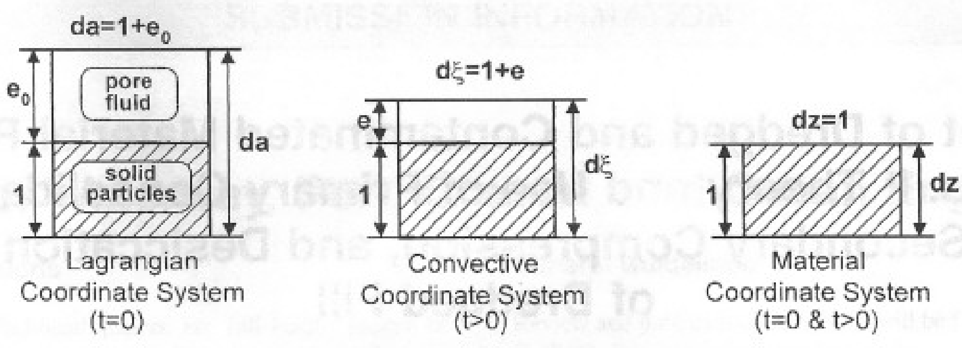

Fig. 1. Comparison of three coordinate systems with differential soil elements used in PSDDF

sionless materials in the model, and (6) improvement of numerical execution schemes.

The major inputs required by PSDDF are the void ratio-effective stress and void ratio-permeability relationships obtained from laboratory consolidation tests on the dredged fill and compressible foundation materials. The recommended laboratory testing procedures to obtain these relationships are described by Cargill (1985, 1986) and Poindexter (1988). In addition, the specific gravity of solids, initial void ratio, C_α/C_c ratio where C_α is the secondary compression index and C_c is the compression index, C_r/C_c ratio where C_r is the recompression index, and the desiccation characteristics of the dredged fill material are required. The values of C_α, C_c, and C_r can be obtained from a 1D oedometer test performed in accordance with ASTM (1999) Standard D 2435-96 for effective stresses ranging from 1 to 100 kPa. Self-weight consolidation tests can be used to obtain the consolidation parameters for effective stresses ranging from 0.003 to kPa. Climatological data, anticipated dredging schedules and quantities, water table elevation, and drainage characteristics of the containment site are also required.

Primary Consolidation Processes

The mathematical model of 1D primary consolidation used in PSDDF is based on the finite-strain theory of consolidation described by Cargill (1982). The governing equation of the finite-strain consolidation process developed by Gibson et al. (1967) is shown below:

$$\left(\frac{\gamma_s}{\gamma_w}-1\right)\frac{d}{de}\left[\frac{k(e)}{(1+e)}\right]\frac{\partial e}{\partial z}+\frac{\partial}{\partial z}\left[\frac{k(e)}{\gamma_w(1+e)}\frac{d\sigma'}{de}\frac{\partial e}{\partial z}\right]+\frac{\partial e}{\partial t}=0 \tag{1}$$

where γ_s=unit weight of soil solids; γ_w=unit weight of water; e=void ratio; $k(e)$=coefficient of soil permeability as a function of void ratio; z=vertical material coordinate measured against gravity; σ'=effective stress; and t=time.

This equation is well suited for the prediction of consolidation in thick deposits of very soft, fine-grained soils, such as dredged material, because it provides for the effects of (1) self-weight consolidation, (2) permeability varying with void ratio, (3) a stress-dependent void ratio-effective stress relationship, and (4) large vertical strains. Because a closed form solution of Eq. (1) is not possible, an explicit finite-difference scheme is used to reduce Eq. (1) to a tractable form. The numerical procedure is described by Cargill (1982), and the explicit finite-difference form of Eq. (1) is

$$e_{i,j+1}=e_{i,j}-\frac{\tau}{\gamma_w}\left(\left\{\gamma_c\beta(e_{i,j})+\left[\frac{\alpha(e_{i+1,j})-\alpha(e_{i-1,j})}{2\Delta z}\right]\right\}\times\left[\frac{e_{i+1,j}-e_{i-1,j}}{2\Delta z}\right]+\alpha(e_{i,j})\left[\frac{e_{i+1,j}-2e_{i,j}+e_{i-1,j}}{\Delta z^2}\right]\right) \tag{2}$$

where i=grid point in space; j=time step (j=present time and $j+1$=future time); τ=time step in finite-difference mesh; and γ_c=buoyant unit weight of solids ($\gamma_c=\gamma_{sat}-\gamma_w$ where γ_{sat}=unit weight of saturated soil solids). A function of the void ratio and permeability, $\beta(e)$, is defined by

$$\beta(e)=\frac{d}{de}\left[\frac{k(e)}{1+e}\right] \tag{3}$$

a function of void ratio, permeability, and compressibility, $\alpha(e)$, is defined by

$$\alpha(e)=\frac{k(e)}{1+e}\frac{d\sigma'}{de} \tag{4}$$

and the vertical space interval or sublayer thickness in material coordinates in the finite-difference mesh is Δz.

Three coordinate systems are used to analyze the consolidation processes: Lagrangian (a), convective (ξ), and material (z) coordinate systems. The Lagrangian coordinate system is used to represent the initial configuration of the soil layers measured at time t=0. For time t>0, that is, during the consolidation process, measurements are made using the convective coordinate system, which is a function of the Lagrangian coordinates and time. Both the Lagrangian and convective coordinates are a measurement of the soil system, including both solid soil particles and the pore fluid. The material coordinate system is a measure of only the volume of solid particles.

Because dredged materials are compressible and usually undergo large strains, it is necessary to choose a coordinate system that moves with the material. This is achieved by using the material coordinate system, which uses elements of constant size and is uniquely suited for use in the time-dependent consolidation problem because this coordinate system is independent of time and the amount of strain (Cargill 1982). Therefore, the material coordinate system is used in the PSDDF model for computational purposes.

It is necessary to develop a method of conversion from the material coordinate system to other coordinate systems so that the layer thickness may be expressed in conventional units at any time. Fig. 1 shows a differential element using the three coordinate systems (Cargill 1982). In Fig. 1, e_0=initial void ratio and

e=void ratio at some later time during consolidation. The coordinate systems can be related to each other using the following expressions (Cargill 1982):

$$\frac{dz}{da} = \frac{1}{1+e_0} \tag{5}$$

$$\frac{d\xi}{dz} = 1 + e \tag{6}$$

$$\frac{d\xi}{da} = \frac{1+e}{1+e_0} \tag{7}$$

These relationships are used to relate the material coordinate system to the other two coordinates so relevant values of settlement can be calculated by PSDDF.

Once the initial and boundary conditions are defined (described in the following section) and appropriate relationships between void ratio and effective stress ($e-\sigma'$) and void ratio and coefficient of permeability ($e-k$) are specified, the void ratio in the consolidating layer can be calculated by the explicit finite-difference technique for any future time. The void ratio distribution in the saturated dredged fill layer is used to calculate the corresponding stresses and pore-water pressures.

Initial and Boundary Conditions

The initial conditions of a saturated dredged fill layer can be written as

$$e(z,t) = e_{00} \text{ for } t = 0 \tag{8}$$

where e_{00}=void ratio at zero effective stress. This is an instantaneous condition when the dredged material reaches the end of the sedimentation process and the soil solids begin to form a continuous soil matrix. Effective stresses in the dredged layer are set to zero at this time. Although the entire layer does not end sedimentation and begin consolidation at the same time, this is a reasonable approximation because consolidation proceeds for a much longer time than the total time required for complete sedimentation (Cargill 1985).

The top boundary condition in a dredged fill layer not subjected to surface desiccation is represented as

$$e(L,t) = e_{00} \text{ for } t > 0 \tag{9}$$

where L=total layer thickness in the material coordinate system.

Three interface boundary conditions are possible between consolidating layers: an impermeable boundary, a free-draining boundary, and a semipermeable boundary. The boundary condition at an impermeable lower interface is represented by

$$\frac{\partial e}{\partial z} = -\gamma_c \frac{de}{d\sigma'} \text{ for } t > 0 \tag{10}$$

The impermeable boundary condition is used where the dredged fill overlies a relatively impervious, incompressible foundation layer. The determination of the void ratio at an impermeable boundary requires the use of a fictitious mesh point outside the boundary, that is, $e_{0,j}$. In this case, the lower boundary denoted in Fig. 2 represents an impervious foundation layer, and the interface boundary is an impermeable boundary. The impervious foundation layer is not modeled numerically but only provides a fictitious mesh point or boundary condition for this layer. Using the void ratio distribution at any time step, t_j, the void ratio at the

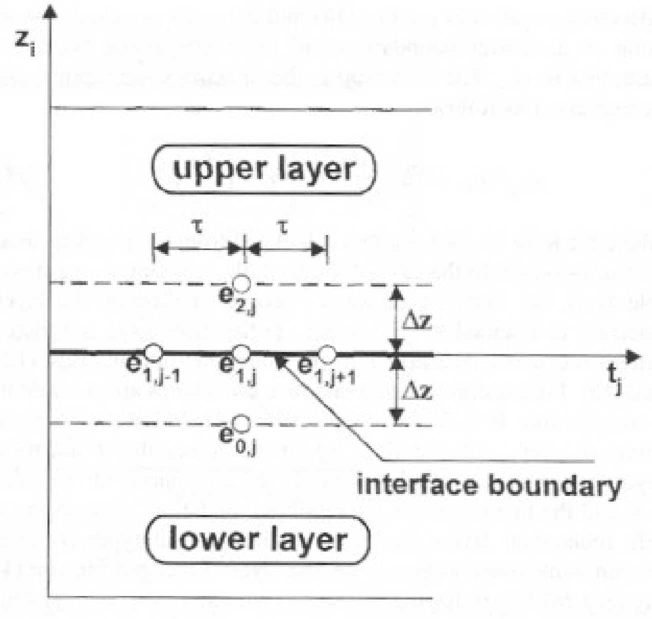

Fig. 2. Void ratio calculation at interface boundary

fictitious mesh point for the impervious foundation layer is calculated by expressing Eq. (10) in finite-difference terms as

$$e_{0,j} = e_{2,j} + 2\Delta z \gamma_c \left(\frac{de}{d\sigma'}\right)_{1,j} \tag{11}$$

where $(de/d\sigma')$ is determined for $e_{1,j}$ from the void ratio-effective stress relationship input data. With $e_{0,j}$ determined, $e_{1,j+1}$ is then calculated from Eq. (2) and the process repeats at each time step.

At a free-draining lower boundary, the excess pore-water pressure is set to zero and the total pore-water pressure is equal to the static or hydrostatic pore-water pressure. The effective stress at the boundary is found by subtracting the hydrostatic pore-water pressure from the total vertical stress. Therefore, the void ratio at the boundary is obtained by using the calculated effective stress and the void ratio-effective stress relationship input by the user.

At a semipermeable boundary between the upper and lower layers in Fig. 2, the quantity of fluid flowing out of one layer must equal the quantity of fluid flowing into the layer across their common boundary. This fact assures a fluid balance. The fluid balance relation can be expressed as follows:

$$\left[\frac{k}{(1+e)}\frac{\partial u}{\partial z}\right]_{\text{upper}} = \left[\frac{k}{(1+e)}\frac{\partial u}{\partial z}\right]_{\text{lower}} \tag{12}$$

where u=excess pore-water pressure. The total, static, and excess pore-water pressure must be equal in the two layers at their common interface boundary so that

$$(u)_{\text{upper}} = (u)_{\text{lower}} \tag{13}$$

From the principle of effective stress—that is, effective stress equals the total vertical stress minus the total pore-water pressure—and an equilibrium condition requirement, the semipermeable interface boundary, can be expressed as follows:

$$\frac{\partial e}{\partial z} = -\left(\gamma_c + \frac{\partial u}{\partial z}\right)\frac{de}{d\sigma'} \tag{14}$$

Calculation of the void ratio at the interface boundary between adjacent permeable layers is accomplished by writing a finite-difference expression for Eq. (14) and using an imaginary mesh point in the lower boundary, as in the impermeable boundary case, that is, $e_{0,j}$. The void ratio at the imaginary mesh point can be expressed as follows:

$$e_{0,j}=e_{2,j}+2\Delta z\left(\frac{de}{d\sigma'}\right)_{1,j}\left[\gamma_c+\left(\frac{\partial u}{\partial z}\right)_{1,j-1}\right] \tag{15}$$

where the term $(\partial u/\partial z)$ is either calculated from the previous time step or assumed. In the case of dredged fill overlying a compressible layer, the excess pore-water pressure gradient at the layer interface is assumed to be zero for the first time step, and thereafter it is calculated based on the previous condition and Eqs. (12) and (13). Four semipermeable interface boundaries are possible in a system that includes a compressible foundation overlain by dredged layers and underlain by an incompressible foundation layer. The interfaces are between (1) the incompressible foundation and the first layer of compressible foundation, (2) compressible foundation layers having different material types, (3) the compressible foundation and the first layer of dredged fill, and (4) dredged fill layers having different material types. In a system without a compressible foundation layer, the two possible semipermeable interface boundaries are between (1) the incompressible foundation and the first layer of dredged fill and (2) dredged fill layers having different material types.

Most semipermeable interface boundaries can be numerically simulated by calculating the excess pore-water pressure gradient $(\partial u/\partial z)$ from the previous time step. The only exception is the case of a semipermeable boundary between the incompressible foundation layer and the first layer of compressible foundation or the first layer of dredged fill. Because the gradient of excess pore-water pressure is not numerically calculated in the incompressible foundation layer, it is necessary to empirically compute this gradient along with the length of the vertical drainage path. The drainage path length is defined as the vertical distance through the incompressible foundation required for complete dissipation of the excess pore-water pressure existing at the interface boundary.

In the PSDDF model, an incompressible foundation that is free draining has a vertical drainage path length of 0.3 m (1 ft), while an impervious incompressible foundation has a vertical drainage path length of 30 m (100 ft). Thus a partially drained incompressible foundation has a vertical drainage path length between 0.3 and 30 m (1 and 100 ft). Only the vertical drainage path is considered because PSDDF is a 1D analysis. This excess pore-water pressure at the boundary and the drainage path length are used to determine the gradient of excess pore pressure at the incompressible side of the boundary by the following equation:

$$\frac{\partial u}{\partial z}=\frac{u}{\left(\frac{x}{1+e}\right)} \tag{16}$$

where u=excess pore-water pressure at the boundary; x=length of the vertical drainage path; and e=void ratio of the incompressible layer.

Over- or Underconsolidated Compressible Foundation

PSDDF was recently modified to allow the compressible foundation material to be over- or underconsolidated. Prior versions of PSDDF only allowed the compressible foundation to be normally consolidated (Stark 1991). Compressible foundation layers overlain by dredged fill layers may consist of previously disposed of dredged material or naturally occurring material that is normally over- or underconsolidated. PSDDF is now able to simulate different stress histories for the compressible foundation layers using the overconsolidation ratio (OCR), which is the preconsolidation pressure divided by the effective vertical stress. This option was included in PSDDF because the compressible foundation layers may be over-, under-, or normally consolidated when new filling occurs.

Over- and underconsolidated compressible foundation materials exhibit different consolidation properties, that is, different void ratio-effective stress and void ratio-permeability relationships, than normally consolidated fine-grained soil or dredged fill. For example, if the compressible foundation material is overconsolidated and data of a normally consolidated material are used, PSDDF will overestimate the consolidation settlement of the compressible foundation material. Conversely, if the compressible foundation is underconsolidated and normally consolidated data are used, PSDDF will underestimate the consolidation settlement. Therefore, it is recommended that oedometer tests be conducted to measure the consolidation properties—for example, preconsolidation pressure—of the compressible foundation materials. Index property tests should also be conducted to characterize the foundation material and allow the oedometer test results to be compared with other data, as described in the companion paper (Stark et al. 2005).

In some instances, it is difficult to measure the consolidation properties of over- or underconsolidated compressible foundation materials. This can be caused by difficulties in sampling the material, especially if it is underwater and if there are gas pressures in the sediment. To overcome this problem, PSDDF now incorporates a procedure to predict the behavior of over- or underconsolidated compressible foundation layers using the average OCR of each layer and the normally consolidated material properties. The average OCR value is defined as follows:

$$\mathrm{OCR}_{\mathrm{average}}=\left(\frac{\sigma'_p}{\sigma'_0}\right)\text{ at middle point of layer being considered} \tag{17}$$

where σ'_p=preconsolidation effective stress, and σ'_0=effective overburden pressure.

In the case of an overconsolidated compressible foundation layer, the average OCR is greater than unity. Once the value of average OCR has been determined for a particular foundation

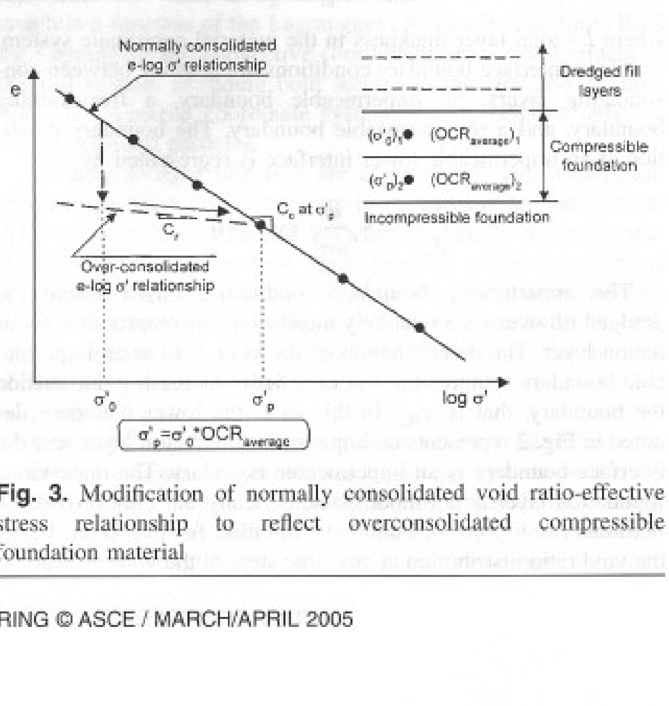

Fig. 3. Modification of normally consolidated void ratio-effective stress relationship to reflect overconsolidated compressible foundation material

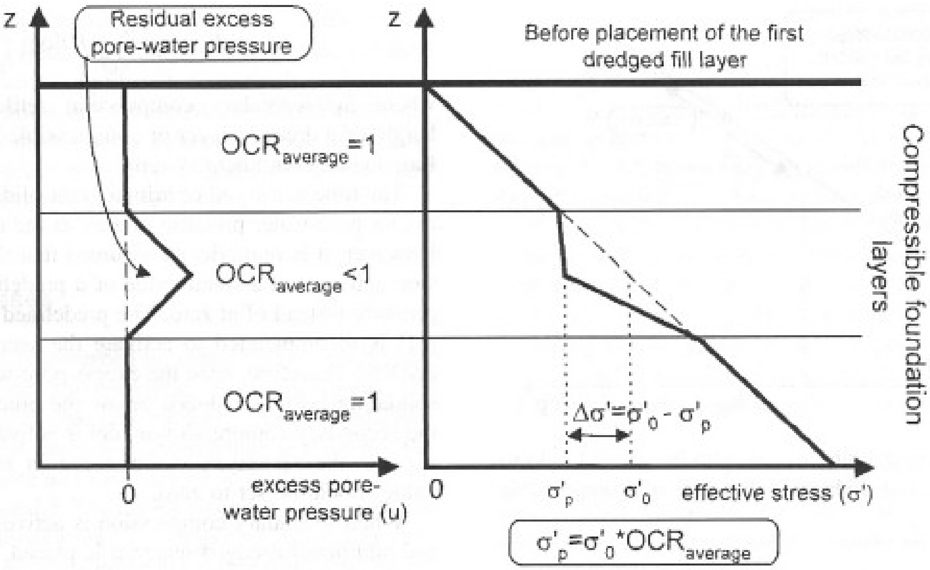

Fig. 4. Distribution of residual excess pore-water pressure and effective overburden pressure to reflect underconsolidated compressible foundation material

layer, the value of σ'_p for each sublayer used to model the entire compressible foundation layer can be calculated by multiplying σ'_0 at the midpoint of each sublayer by the average *OCR*. Using the compression index (C_c) at the value of σ'_p and the ratio of the recompression index to the compression index, that is, C_r/C_c, from oedometer tests, the void ratio-effective stress relationship for the overconsolidated compressible layer is shifted from that used to model a normally consolidated compressible layer to that for an overconsolidated layer, as shown in Fig. 3.

This modified void ratio-effective stress relationship is used as input for PSDDF as the void ratio-effective stress relationship for that layer. The initial effective stress profile is recalculated based on the modified void ratio-effective stress relationship, but that relationship has a flatter slope than the normally consolidated void ratio-effective stress relationship. A flatter void ratio-effective stress relationship can cause a reduced time step, which results in increased computational time.

In the case of an underconsolidated compressible foundation layer, residual excess pore-water pressures exist in the foundation layer, which results from previous loading or self-weight consolidation. In other words, primary consolidation of the compressible foundation has not been completed. In this situation, the residual excess pore-water pressure is equal to the reduction of initial overburden pressure, as shown in Fig. 4. The shape of the pore-water pressure distribution is assumed to be triangular in this method to satisfy stress continuity through each layer.

The average *OCR* is less than unity in the underconsolidated case, which means σ'_p ($<\sigma'_0$) becomes the new initial overburden pressure at the middle of the sublayer. As shown in Fig. 5, the initial overburden pressure is shifted by $\Delta\sigma'=\sigma'_0-\sigma'_p$, causing the initial void ratio to be increased along the normally consolidated void ratio-effective stress relationship. Therefore the differences between under- and normally consolidated conditions are the values of the initial effective stress, that is, σ'_p versus σ'_0, and the void ratio, that is, $(e_0)_{new}$ versus e_0.

Once the void ratio-effective stress relationship has been modified for an over- or underconsolidatd compressible foundation layer, modification of the corresponding void ratio-permeability relationship is obtained by shifting the initial permeability along the normally consolidated void ratio-permeability relationship according to the modified initial void ratio. As shown in Fig. 6, the initial permeability shifts downward for an overconsolidated compressible foundation layer, and the initial permeability shifts upward for an underconsolidated, compressible foundation layer. After the initial permeability is obtained, a new permeability is calculated at each time step in PSDDF using the corresponding void ratio along the normally consolidated void ratio-permeability relationship.

Subsurface Drainage or Capping Layers

Cohesionless materials can now be modeled in PSDDF to simulate a sand-capping layer for an underwater placement mound, sandy material for underwater filling or construction, and/or sandy dredged material placed in a confined area for drainage purposes. The cohesionless sandy materials will undergo a small amount of compression and provide the appropriate overburden stress and drainage to underlying cohesive soils, but will not undergo desiccation. Three types of cohesionless materials are now

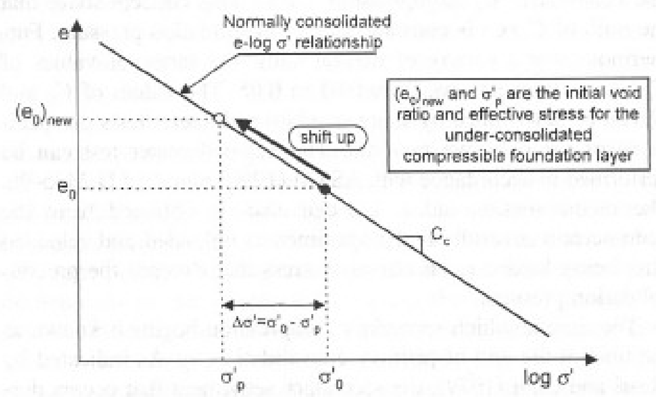

Fig. 5. Modification of initial effective stress and void ratio to reflect underconsolidated compressible foundation material

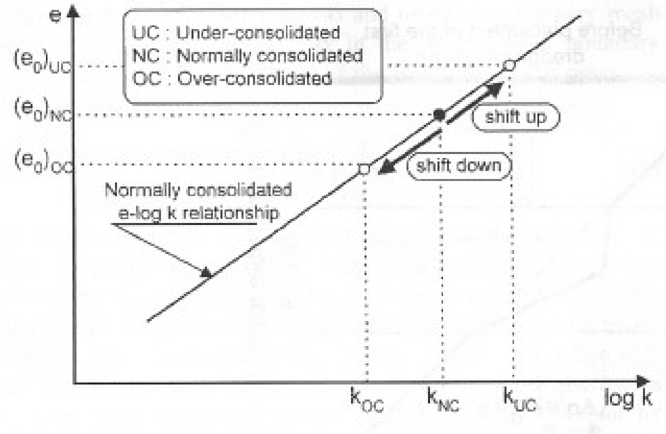

Fig. 6. Shifting initial permeability according to initial void ratio to reflect over- and underconsolidated behaviors of compressible foundation material

included in the PSDDF database: clean sand (SP, according to the Unified Soil Classification System), silty sandy (SM), and clayey sand (SM-SC) (Stark et al. 2005).

It is recommended that the void ratio relationships for one of these sandy materials be used for all sandy or cohesionless layers in a simulation to ensure stability of the computations. The shape, and thus slope, of the void ratio-effective stress relationships for a cohesionless soil is flatter than for a cohesive soil and can lead to a decreased calculation time step, and thus to increased computational time. The sandy soil relationships included in the database of PSDDF will not cause a large increase in computing time and will provide a reasonable representation of settlement caused by an increase in effective stress.

Secondary Compression Processes

The current version of PSDDF includes a new procedure to calculate secondary compression settlement of dredged fill layers and compressible foundation materials. The C_α/C_c concept developed by Mesri and Godlewski (1977) for the analysis of secondary settlement is incorporated into PSDDF. This concept is based on the observation that the magnitude and behavior of C_α (secondary compression index) with time is related to the magnitude and behavior of C_c (compression index). This concept states that the ratio of C_α/C_c is constant at any consolidation pressure. Furthermore, for a variety of natural soils, the range of values of C_α/C_c is rather narrow, from 0.01 to 0.05. The values of C_α and C_c can be determined by using standard oedometer tests on specimens of the cohesive material. The 1D oedometer test can be performed in accordance with ASTM (1999) Standard D 2435-96. The recompression index, C_r, can also be obtained from the oedometer test results if the specimen is unloaded and reloaded after being loaded to an effective stress that exceeds the preconsolidation pressure.

The time at which secondary compression begins is known as the time at the end of primary consolidation, t_p. As indicated by Mesri and Choi (1979), the secondary settlement that occurs during an increment of time from t_p to time t is estimated using the following equation, assuming that C_α remains constant during this interval:

$$S_s = \frac{C_\alpha}{(1+e_0)}(H)\log\left(\frac{t}{t_p}\right) \tag{18}$$

where S_S=secondary compression settlement, and H=initial height of a dredged layer or compressible foundation layer in the Lagrangian coordinate system.

The time at the end of primary consolidation corresponds to an excess pore-water pressure of zero at the midpoint of each layer. However, it is numerically assumed that the secondary compression activates at a small value of a predefined excess pore-water pressure instead of at zero. The predefined value of 0.22 kPa (4.5 psf) is recommended to activate the secondary compression in PSDDF. Therefore, once the excess pore-water pressure in a consolidating layer is reduced below the predefined value by users, the secondary compression model is activated by PSDDF. To deactivate the secondary compression in PSDDF, the predefined value should be set to zero.

When secondary compression is active in the old dredged fill and additional dredged material is placed, there is an increase in excess pore-water pressure, and when such an increase is detected, the secondary compression calculations are terminated and primary consolidation is activated. However, the secondary compression may have produced a decrease in void ratio at constant effective stress, σ'_i, as shown in Fig. 7. This decrease in void ratio is determined from the end of the primary consolidation void ratio-effective stress relationship. As a result, a new void ratio-effective stress relationship must be established that joins the void ratio at the end of secondary compression at σ'_i, that is, the time when additional dredged material was placed, with the end of the primary consolidation relationship. The effective stress at which the secondary compression void ratio connects with the end of primary consolidation relationship, σ'_c, is estimated from the following relationship presented by Mesri and Choi (1979) and shown in Fig. 7:

$$\frac{\sigma'_c}{\sigma'_i} = \left(\frac{t}{t_p}\right)^{[(C\alpha/Cc)/(1-Cr/Cc)]} \tag{19}$$

Establishing a new void ratio-effective stress relationship for each point in the soil profile usually results in a reduction in the

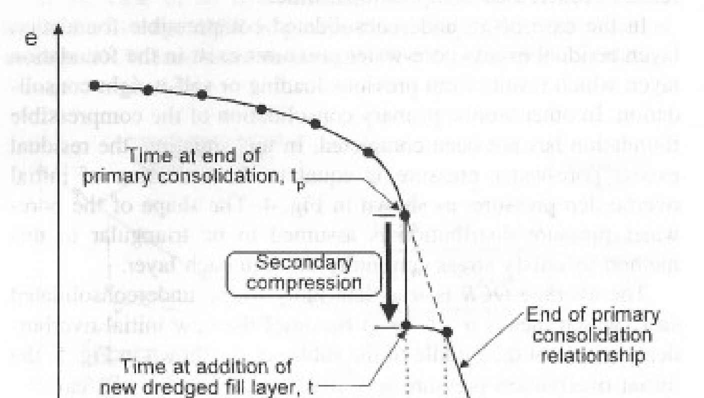

Fig. 7. Effect of secondary compression on end-of-primary void ratio-effective stress relationship (after Mesri and Choi 1979)

time step and an increase in computational time. With a large number of dredged fill layers and/or material types, the increase in computational time can be significant. As a result, it is recommended that the simulation be conducted initially without the secondary compression model being activated. After debugging the simulation, the secondary compression model can be activated to determine the impact on the total settlement.

Desiccation Processes

The desiccation process is governed by many factors whose predictability is often difficult. The empirical desiccation process described herein may seem inconsistent with the rather sophisticated consolidation model, but primary consolidation contributes the largest decrease in volume and thus should be analyzed by using the most sophisticated model. The development of a sophisticated theoretical model for desiccation is a subject of future research. Desiccation of a dredged material induces removal of water by changing the state of the water near the surface from liquid to vapor. This change of state results primarily from evaporation and transpiration. Plant transpiration is considered insignificant because of the recurrent deposition of dredged fill and the establishment of primarily shallow rooted vegetation.

The desiccation process appears to occur in two major stages. During the first stage, sufficient free water is available at the surface of the material so that evaporation takes place at the full potential rate, and in the second, drying proceeds at some fraction of the potential rate and decreases as the depth of dried crust increases. The procedure incorporated in PSDDF for quantifying settlement due to desiccation was developed by Cargill (1985). The water lost from a dredged material layer during first-stage drying can be expressed as follows:

$$\Delta W' = CS - \lfloor (C'_E)EP \rfloor + (1 - C_D)RF \qquad (20)$$

where $\Delta W'$=water lost during first-stage drying; CS=water supplied from lower consolidating soil; C'_E=maximum evaporation efficiency for soil type; EP=Class A pan evaporation; C_D=drainage efficiency of containment area; and RF=rainfall. Even though some minor cracks may appear in the surface during this stage, the material will remain saturated and vertical settlement is expected to correspond to the water loss.

$$\delta'_D = -\Delta W' \qquad (21)$$

where δ'_D=settlement due to first-stage drying.

Water lost during second-stage drying can be written as

$$\Delta W'' = CS - C'_E\left[1 - \frac{h_{wt}}{h_{2nd}}\right]EP + (1 - C_D)RF \qquad (22)$$

where $\Delta W''$=water lost during second-stage drying; h_{wt}=depth of water table below surface; and h_{2nd}=maximum depth of second-stage drying. Two phenomena prevent an exact correspondence between water loss and settlement during second-stage drying. One is the appearance of an extensive network of cracks that may encompass up to 20% (Haliburton 1978) of the volume of the dried layer, and the other is the probable loss of saturation within the dried material itself. Combining these two occurrences into one factor enables the vertical settlement to be written as follows:

$$\delta''_D = -\Delta W'' - \left[\frac{PS}{100}\right]h_{wt} \qquad (23)$$

where δ''_D=settlement due to second-stage drying; and PS=gross percent saturation of dried crust, which includes cracks. Determining the second-stage drying settlement is an iterative procedure because there are two equations and three unknowns.

At the end of each monthly period during times when the desiccation process is active, the effect of the previous month's evaporation is applied to the dredged material. For computational simplicity, changes in void ratio are applied only at nodal points beginning at the surface of the dredged material. Also, to avoid the trial-and-error method of solving Eq. (23), PSDDF calculates the desiccation settlement (δ_D) as follows:

$$\delta_D = -\Delta W - \delta'''_D \qquad (24)$$

where δ'''_D represents any carryover desiccation settlement. Carryover desiccation usually includes desiccation caused by the loss of saturation the previous month (a value that also takes into account the crack network during second-stage drying). It may also include a negative desiccation quantity from the previous month (if water is lost because consolidation exceeds potential evaporation desiccation) and/or a quantity from any necessary adjustment in the void ratio at the top of the consolidating layer.

With the desiccation settlement from Eq. (24), PSDDF next determines the average void ratio reduction within a dredged material sublayer (that material between adjacent nodes) by

$$\Delta e = \frac{\delta'''_D}{\Delta z} \qquad (25)$$

Starting with the uppermost adjustable node, void ratios are adjusted toward or to the void ratio at the desiccation limit, e_{DL}, or the void ratio at the shrinkage limit, e_{SL} (depending on whether first- or second-stage drying is effective) until the average required reduction has been achieved. As the dredged material is desiccated below the value of e_{SL}, the free-water table drops below the surface of the material. In PSDDF the water table is set at the first calculation nodal point having a void ratio less than e_{SL} but not deeper than the maximum depth of second-stage drying. The desiccation subroutine in PSDDF also calculates a new ultimate void ratio distribution for material in the consolidating layer based on the surcharge created by dried material above the new water table. The uppermost void ratio in the consolidating layer is then set to its ultimate value, which becomes the top boundary condition for the next series of consolidation calculations.

There are some drawbacks to this simplistic treatment of the desiccation process in fine-grained dredged material. No attempt has been made to model the complex mechanisms of how a soil gets to the final desiccated volumetric condition or how and to what magnitude stresses and pore-water pressures develop in the desiccated portion. A rigorous analysis of desiccation process is not warranted because of the limited information available on the factors that actually control the process and the large magnitude of consolidation settlement. The mathematical model and solution technique proposed avoids the necessity of knowing the complex mechanisms or the multitude of factors that control them. The overall effect of desiccation is correctly represented because desiccation leads to a void ratio reduction in the dried material. The presence of a dried surface does change the boundary condition in the consolidating material, and the effect of an extensively cracked crust is to increase the speed and magnitude of consolidation in the underlying material. The accuracy of this method

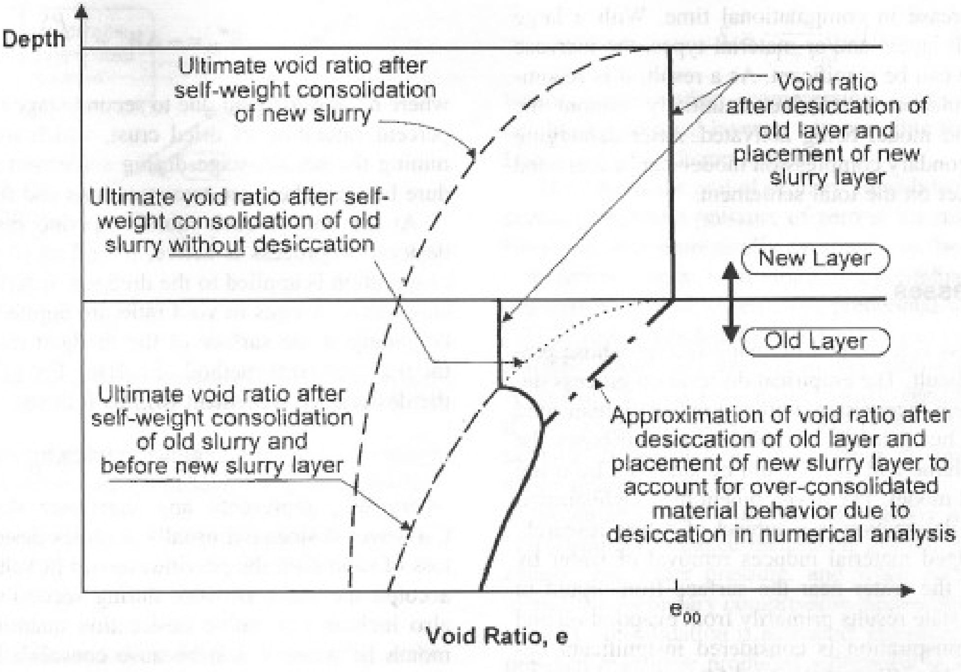

Fig. 8. Void ratio distribution immediately after placement of new dredged layer on partially overconsolidated layer (after Cargill 1985)

depends on properly defining the proposed quantities e_{SL} and e_{DL} and how well these quantities can be used to represent the true boundary condition, that is, cracked crust, of the consolidating layer.

Deposition of Additional Dredged Materials

PSDDF allows the deposition of additional dredged material types at any time after placement of the first dredged material layer. One hundred different dredged fill material types can be deposited in a simulation. In the absence of any desiccation in prior deposits, there is a natural transition between the old and new materials because the void ratio at the top of the old layer and the bottom of the new layer correspond to the void ratio at zero effective stress. However, when the top of the old layer has been desiccated and extensively cracked, there is a discrepancy in the value of the actual void ratio at the boundary node. Because of the probable extensive cracking at this point, it is reasonable to approximate the actual void ratio as an average of the void ratio of the recently deposited material (the zero effective stress void ratio) and the desiccated void ratio. Void ratios in the remainder of the previously desiccated material are assumed to be maintained at their desiccated values.

While evidence exists to indicate that old dredged fill layer boundaries offer some enhancement to material drainage, it would be optimistic to assume that these boundaries are free draining for consolidation purposes. Therefore, future consolidation is based on the void ratio distribution shown in Fig. 8, commensurate with the effective stresses through the previously desiccated material. In the previously desiccated zone, the calculated void ratios are linearly varied between the boundary node at the zero effective stress void ratio and the node below the desiccated zone at a void ratio due to prior consolidation and not desiccation. When the calculated void ratios are again equal to the actual void ratios, consolidation of the entire layer proceeds according to the finite-strain consolidation theory in the normal manner (Cargill 1985).

Execution of PSDDF

The new version of PSDDF increases the total number of sublayers from 500 to 1,000 for dredged fill layers and from 50 to 200 for compressible foundation layers. Also, the total number of material types for both dredged fill and compressible foundation materials is increased from 50 to 100. These increases allow PSDDF to be more versatile and simulate detailed dredged material placement schedules for relatively long-term dredged fill operation areas.

To obtain an accurate numerical solution in PSDDF, a small sublayer thickness and time step should be used. However, the sublayer thickness and time step are a function of each other. For the solution of the finite-strain consolidation, Eq. (1), to converge, the sublayer thickness and time step should be small enough to ensure consistency and stability of the solution. A solution is consistent when the numerical discretization accurately represents the continuous problem described by the differential equation. Solution stability means that the small errors introduced initially are small enough that the cumulative magnitude of the error does not impact the solution process. The Lax equivalence theorem states that consistency and stability are both necessary and sufficient for convergence (Heath 1997). Convergence means that the numerical solution is in agreement with the exact solution of the differential equation.

To satisfy the consistency and stability criteria in PSDDF, Cargill (1982) recommends that the sublayer thickness in the material coordinate system and time step be calculated using the following equations:

$$\Delta z \leq \left| \frac{2\alpha(e)}{\gamma_c \beta(e) + \frac{\partial}{\partial z}[\alpha(e)]} \right|_{\min} \qquad (26)$$

$$\tau \leq \left| \frac{\Delta z^2 \gamma_w}{2\alpha(e)} \right|_{\min} \quad (27)$$

where Δz and τ are the sublayer thickness in the material coordinate system and the time step, respectively. It can be seen that τ is a function of Δz, and thus the two values are a function of each other.

If a value of Δz for a certain sublayer of dredged fill in PSDDF becomes larger than the criterion in Eq. (26), PSDDF provides a warning message to decrease the sublayer thickness by increasing the number of sublayers used to model that dredged fill layer.

The prior version of PSDDF (Stark 1991) required the user to choose the appropriate time step to satisfy the criterion in Eq. (27). In the present version, a suitable time step is calculated at an initial stage of the analysis. The time step is calculated using Eq. (27) and the minimum value of Δz for any dredged fill layer. The value of Δz is calculated for each dredged fill layer using Eq. (5) and expressed as follows:

$$\Delta z = \left[\frac{\text{thickness of fill layer}}{\text{number of sublayers}} \right] \frac{1}{(1+e_0)} \quad (28)$$

A value of Δz is calculated for each dredged fill layer that will be applied in the simulation, and the minimum value of Δz for all of the fill layers that will be applied throughout the simulation is selected for calculating the time step. A small value of Δz for any fill layer results in a small τ, which slows the execution of the entire analysis but is necessary to ensure stability.

Using a large number of sublayers results in PSDDF utilizing a small value of time step that increases the computational time. Therefore it is necessary to choose an appropriate value of the number of sublayers so PSDDF calculates a sufficiently large value of Δz, which increases the time step. The value of Δz cannot be too large; otherwise, the consistency and/or stability requirements may not be satisfied. A good starting point is to select the number of sublayers such that the mesh size/thickness of any sublayer in the Lagrangian coordinate system is greater than 0.15 m ($\approx$0.5 ft) when the dredged fill layer thickness is greater than 0.5 m ($\approx$1.5 ft). The mesh size/thickness of a sublayer in the Lagrangian coordinate system is calculated by dividing the dredged fill layer thickness by the number of sublayers for that dredged fill layer. To ensure compatibility with adjacent dredged fill layers, the number of sublayers should always be greater than 3.

Conclusions

A 1D effective stress-dependent numerical model, PSDDF (Primary Consolidation, Secondary Compression, and Desiccation of Dredged Fill) simulates the primary consolidation, secondary compression, and desiccation processes in fine-grained soils (e.g., dredged fill) using the 1D finite-strain theory of consolidation (Gibson et al. 1967), the secondary compression theory proposed by Mesri and Godlewski (1977), and an empirical desiccation model (Cargill 1985). PSDDF has been enhanced from the original numerical models, that is, PCDDF, by Cargill (1985) and PCDDF89 (Stark 1991) to include (1) the consideration of secondary compression using the C_α/C_c concept, (2) the ability to predict settlement of over- or underconsolidated compressible foundation materials using an average overconsolidation ratio, (3) use of less compressible cohesionless materials to simulate subsurface drainage or capping layers, (4) improvement of numerical execution schemes such as an increase in the total number of sublayers and material types of dredged fill and compressible foundation layers and time step control, and (5) an enhanced database of input parameters (Stark et al. 2005) to facilitate the use of PSDDF.

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