A Simple Model For Estimating End Of Construction Pore Pressures: Part 2 - Embankment Pore Pressures[1]

David J. Kerkes, Ph.D., P.E.
Consulting Geotechnical (Civil) Engineer
1311 Mustang Trail • Kingwood, Texas 77339 U.S.A.
dkerkes@geotechconsultant.com • http://www.geotechconsultant.com

INTRODUCTION

When a saturated compressible soil of low permeability is loaded, the fluid present in the soil's pore space inhibits the volumetric strain necessary to transfer the load to the soil skeleton, and this phenomenon manifests itself in the development of excess pore pressures in the pore fluid of the soil. This phenomenon has long been recognized in the construction of large earth structures, such as earth dams; however, it is not limited to earth structures and also occurs beneath any structure that applies a load to an underlying soil foundation. Furthermore, the soil need not be fully saturated for pore pressures to develop. The pore water pressures generated in this way dissipate over time; however, they can have an adverse effect on the stability of a structure both during and immediately subsequent to construction. Consequently, it is important for the designer to be able to estimate the magnitude of these pore pressures at any point during construction so the data can be incorporated into conventional slope stability analyses. In addition, the ability to estimate the rate at which pore pressures dissipate enables the designer to develop a plan for staged construction of the embankment if required by the specific site conditions.

This is the second of two application oriented papers available over the Internet at no cost, presenting the details of an analytical model for estimating both the magnitude and rate of dissipation of construction generated pore pressures. The purpose of the papers is to provide what is hoped will be a sufficient amount of detail for an engineer to apply the method to his/her specific problem. The method was specifically developed to permit the analyses to be performed on a personal computer using commercially available spreadsheet software, thereby eliminating the need for special application software. As a companion to each paper a spreadsheet has been prepared by the author, in Microsoft Excel (97-2000 & 5.0/95 compatible formats) illustrating the solution to an example problem. These spreadsheets are also available over the Internet at no cost.

The first paper written by the author, titled "A Simple Model For Estimating End Of Construction Pore Pressures: Part 1 - Foundation Pore Pressures," illustrates the basic method and is limited to the case of pore pressure development and dissipation in a fully saturated soft soil foundation beneath an earth structure. The second paper builds upon concepts presented in the first; therefore, the author recommends beginning with the first paper in order to develop an understanding of the various elements of the problem and to familiarize oneself with the basic method and algorithms associated with the proposed solution. This first paper will be referred to in subsequent sections simply as "*Part 1*." This second paper discusses how the spreadsheet model developed in *Part 1* can be extended to account for unsaturated soil, as well as the case of a moving drainage boundary, and illustrates how to estimate pore pressures generated within the core of a zoned earth dam both during and subsequent to construction. It is the author's hope that engineers will attempt to use the methods described in these papers in concert with field instrumentation to monitor actual performance, thereby either further validating the model or providing a mechanism through which the methods can be modified and improved. The author would appreciate any feedback from individuals regarding the methods presented in these papers and would be willing to answer questions via e-mail to assist those developing a solution to their specific problem.

EXAMPLE PROBLEM

A spreadsheet has been prepared as a companion to this paper that presents the solution to an example problem for the case of construction generated pore pressures in the core zone of an earth embankment. As previously noted, the spreadsheet solution is available at no cost over the Internet in Microsoft Excel format. The example problem is described below and illustrated in Figure 1.

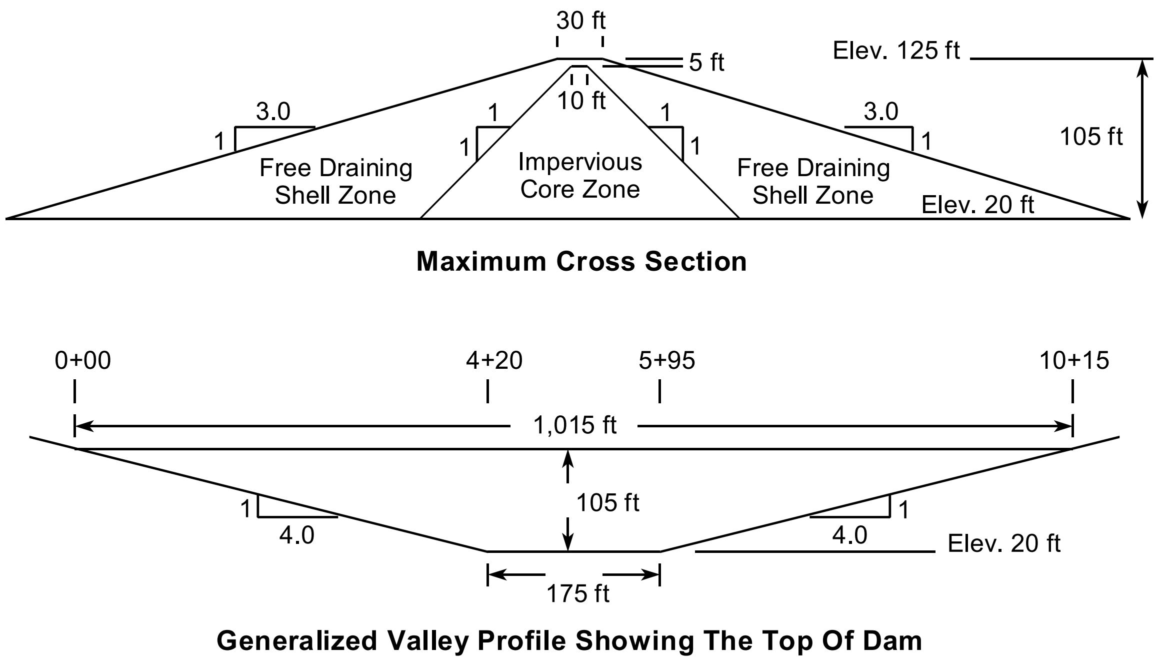

FIGURE 1: Valley profile and maximum embankment cross section for Problem 1.

A zoned earth dam is to be constructed over an impermeable rock foundation. The valley profile slopes downward uniformly at 4.0(H):1(V), from Sta. 0+00 at Elev. 125 feet to Sta. 4+20 at Elev. 20 feet, remains uniform at Elev. 20 feet between Sta. 4+20 and Sta. 5+95, and slopes upward at 4.0(H):1(V) from Sta. 5+95 to Sta. 10+15 at Elev. 125 feet. The embankment will have a maximum height of 105 feet from Sta. 4+20 to Sta. 5+95, upstream and downstream slopes of 3.0(H):1(V), and a crest width of 30 feet at Elev. 125 feet. The core zone will have a top width of 10 feet at Elev. 120 feet and uniform side slopes of 1.0(H):1.0(V). The core zone will be constructed of lean clay with a specific gravity of solids (G_s) of 2.70. The soil has a maximum dry unit weight ($\gamma_{d\text{-}max}$) of 108.1 pcf (pounds per cubic foot) at an optimum moisture content of 17.5 percent, and the core zone material will be compacted to at least 95 percent of the maximum dry unit weight. The analysis should be performed using the highest moisture content at which the specified minimum dry unit weight can be achieved, which corresponds to the highest initial degree of saturation. For this specific material, a moisture content of 20.0 percent is the highest moisture content at which 95 percent of the maximum dry unit weight can still be achieved, which yields an in-place total unit weight of 123.2 pcf. The results of one-dimensional consolidation tests on remolded samples of core zone material are presented in Table 1, the significance of which will be discussed in a subsequent section. While the coefficient of consolidation (c_v) varies during the consolidation process, a constant value of 0.80 square feet per day will be used for the purpose of this example. The embankment shell material will have an in-place moist unit weight of 130 pcf.

TABLE 1: One-Dimensional Compression Test Results On Core Zone Material

Vert. Eff. Stress (Tsf)	0.000	0.125	0.250	0.500	1.000	2.000	4.000	8.000	16.000
Void Ratio	0.6405	0.6255	0.6090	0.5780	0.5470	0.5160	0.4850	0.4540	0.4320

STRESS DISTRIBUTION MODEL

As discussed in *Part 1*, in order to estimate the pore pressures at any particular location it is first necessary to estimate the stress changes that occur at that location during construction. The method proposed herein uses a model based on the theory of elasticity for the prediction of the stress changes beneath a uniform strip load on a semi-infinite linearly elastic isotropic foundation. Using this method an embankment fill can then be modeled as a series of successive uniform strip loads stacked or layered to simulate the embankment cross section, as illustrated in Figure 2. Each uniform strip load can be sized to represent a construction lift, the horizontal dimension of which decreases as a function of the embankment height and side slopes.

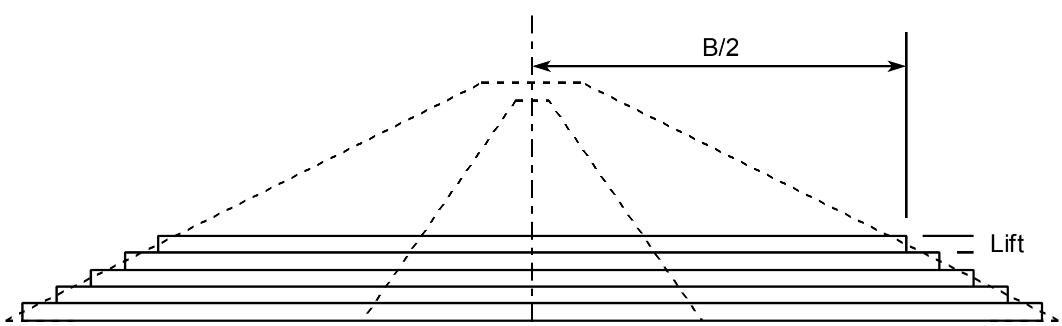

FIGURE 2: Approximation for simple elastic embankment model.

For the case of estimating construction generated pore pressures in the core of a zoned embankment the stress changes need only be computed for the portion of the embankment between the current lift and the top of the foundation. As noted in *Part 1*, the embankment model illustrated in Figure 2 violates the boundary conditions associated with the elastic strip load model, and this limitation will be addressed in a subsequent section.

PORE PRESSURE RESPONSE IN SOIL

As discussed in *Part 1*, pore pressures generated in a soil during undrained loading are a function of the total stress changes produced by the load, the soil porosity, degree of saturation, and the relative compressibilities of the soil skeleton and pore fluid. The relationship adopted for use in the model presented herein is that proposed by Atkinson and Bransby [1], which expresses the pore pressure change in terms of the total stress invariants p and q and the empirical coefficients 'a' and 'b'. These coefficients are fundamentally the same as the A and B parameters in the relationship developed by Skempton [8], which is perhaps the relationship most widely recognized among geotechnical engineers today. Atkinson and Bransby's expression is:

$$\Delta u = b(\Delta p + a\,\Delta q) \qquad (1)$$

Skempton demonstrated that the B parameter is a function of the soil's degree of saturation and is essentially equal to one for a fully saturated soil. The A parameter accounts for the actual behavior of the soil structure. The A parameter has been found to be a function of the compressibility of the soil under hydrostatic stresses and the tendency of the soil to expand or contract in response to shear stresses. As previously discussed in *Part 1*, taking Poisson's ratio equal to 0.5 for undrained loading of a saturated soil, the following expressions are obtained for the total stress invariants p and q for use in computing the pore pressure increment Δu in Equation 1. The w term in Equation 2 is computed simply as the product of the lift thickness and unit weight of the embankment fill material and is illustrated in *Part 1, Figure 2*.

$$p = w\alpha/\pi \qquad (2)$$

which is simply the mean normal total stress, and

$$q = (w/\pi)(\sin\alpha)(\sqrt{3}) \qquad (3)$$

If it is assumed that the pore pressure increment is simply a function of the degree of saturation and the change in the mean normal total stress, Equation 1 reduces to:

$$\Delta u = b\,\Delta p \qquad (4)$$

Equation 4 implies basically that the soil behaves as an elastic material, and for such a material a shear stress increment (addressed by the stress invariant q) would have no effect on pore pressures. While research has shown that shear stresses actually do affect pore pressures in soil, the author has found that this simpler form of the expression yielded a better estimate of pore pressures in a soft unsaturated soil compared to actual measured values than did methods that attempted to account for the 'a' parameter and the stress invariant q [5]. In *Part 1* the assumption was made that $\Delta u = \Delta p$, where the value of b = 1 corresponds to a fully saturated soil. Based on the author's experience, it is believed that this simplifying assumption should provide reasonable results when dealing with saturated soft soils; however, the approach needs to be modified somewhat for unsaturated soils and/or stiff soils, such as overconsolidated clays, where the stiffness of the soil matrix can also have a significant effect on the pore pressures generated. The following paragraphs discuss how the model can be extended to account for partial degrees of saturation, along with the actual stress-strain properties of the soil as determined from a conventional one-dimensional consolidation test.

The subject of pore air and pore water pressures in unsaturated soil is extremely complex and beyond the scope of this paper. It is sufficient here to recognize that an undrained unsaturated soil subjected to a stress increase undergoes a volume reduction as air voids compress, and air in the pore space will dissolve in the pore water as the stress level increases in accordance with Henry's Law[2] of solubility, until the void space is occupied entirely by water. The effect of the degree of saturation on pore pressure response is illustrated in Figure 3.

Assuming for this discussion that the pore pressure change is simply equal to the mean total stress change, a fully saturated soil element would be represented by line number 1 (where $\Delta u = \Delta p$). Curve number 2 represents a soil element that is initially unsaturated. As a result, the pore pressure change is less than the mean total stress change, but increases with increasing stress as the pore air is driven into solution until complete saturation is achieved, beyond which point the pore pressure change equals the mean total stress change. If the soil element does not drain as it is loaded, curve number 2 will remain parallel to curve number 1. The amount that curve number 2 is offset from curve number 1 is a function of the initial degree of saturation. Curve number 3 in Figure 3 reflects the situation where pore pressures are simultaneously generated and dissipated, as in the case of a zoned embankment where the core zone consolidates as the embankment is constructed.

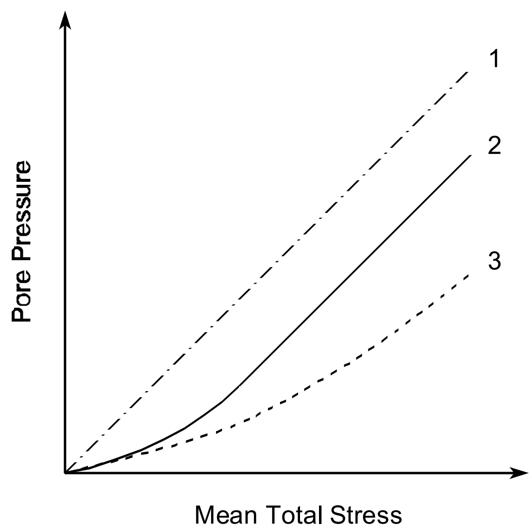

FIGURE 3: Pore pressure response in soil.

A plot of the pore pressures in the core of a zoned embankment during construction would be expected to resemble either curve number 2 or curve number 3 in Figure 3. The extent to which curve number 3 develops from curve number 2 is determined from the analysis of pore pressure dissipation during construction, and is a function of the consolidation properties of the core zone material, the core zone geometry, and the amount of time that elapses during construction. This aspect of the problem will be discussed subsequently. The analysis begins, however, with the development of curve number 2 for the soil to be used in the construction of the core zone, and the following paragraphs describe a simple method that can be used for the development of this curve.

In 1948, Hilf [4] proposed a simple, straightforward method for estimating construction generated pore pressures in earth dams using information obtained from a conventional one-dimensional confined compression test. In an unsaturated soil, loaded under conditions of no drainage (neither air nor water), a measurable reduction in the volume of the soil mass will occur as a result of the compression of the air voids. Hilf observed that, by combining Henry's Law and Boyle's Law[3] and

[2] Henry's Law relates to the solubility of a gas in a liquid, and states that at moderate pressures the solubility of a gas in a liquid is directly proportional to its partial pressure in the gas phase over the solution.

[3] Robert Boyle (1660) observed that the product of pressure and volume of a fixed mass of gas is very nearly constant at constant temperature (only true for ideal gases, but approximate for real gases).

knowing the initial volume of the air, the pressure in the air can be computed for ordinary temperatures from the following expression, if the vapor pressure of the water is neglected:

$$P = \frac{P_a \Delta}{V_a + hV_w - \Delta} \tag{5}$$

where:
- P total (pore) air pressure after consolidation minus atmospheric pressure; i.e., the piezometric pressure,
- P_a initial air pressure, which for compacted soils is very nearly atmospheric, 14.7 psi (pounds per square inch),
- V_a volume of free air in the voids, in percent of the initial volume of the soil,
- V_w volume of water in the voids, in percent of the initial volume of the soil,
- h Henry's solubility constant for air in water by volume (0.0198 at 68^o F), and
- Δ volume change in percent of the initial volume of soil, which equates to the volumetric strain (ε_v) expressed as a percentage.

If surface tension is neglected, the pore air pressure (P) is equal to the pore water pressure (u).

The relationship is *valid only for conditions of no drainage*; therefore, the total volume change (Δ) cannot exceed the initial volume of air (V_a), but can equal it at which point total saturation is achieved (P_s) and Equation 5 becomes:

$$P_s = \frac{P_a V_a}{hV_w} \tag{6}$$

Hilf proposed a method for developing a relationship between vertical total stress (σ_v) and pore pressure (u) using Equations 5 and 6 and the results of the conventional laboratory one-dimensional confined compression test, which yields a relationship between vertical effective stress (σ_v') and volumetric strain (ε_v or Δ) for conditions of total lateral restraint ($\varepsilon_h = 0$). While the relationship proposed by Hilf was developed on the basis of vertical effective stress and vertical total stress, the relationship can also be developed on the basis of mean normal effective stress (p') and mean normal total stress (p) by noting the following relationships for the case of confined compression:

$$\sigma_h' = \sigma_2' = \sigma_3' = K_o \sigma_v' \tag{7}$$

therefore,

$$p' = 1/3(\sigma_1' + \sigma_2' + \sigma_3') = 1/3(\sigma_1' + 2\sigma_h') = 1/3(\sigma_v' + 2K_o\sigma_v') \tag{8}$$

The lateral stress coefficient at rest (K_o) can be estimated using Jaky's formula ($K_o = 1 - \sin\phi'$).

The steps in Hilf's method are outlined below, and Sheet 2 of the example spreadsheet illustrates the procedure to develop the relationship between pore pressure (u) and mean normal total stress (p) using the data from a conventional one-dimensional compression test presented in Table 1.

1. Compute the phase relations (V_s , V_w , and V_a), expressed as percentages of the total volume, for the soil in its initial or compacted state.

2. Compute the percent volumetric strain for each value of void ratio at the end of each load increment, where: $\varepsilon_v = (\Delta e / 1+e_o)$
3. The initial volume of air (V_a) will correspond to the volumetric strain (Δ) at complete saturation (S = 1); therefore, insert this value into its proper location in the tabulated data.
4. Using the void ratio-effective stress relation obtained from the confined compression test, interpolate the vertical effective stress value (σ_v') for the volumetric strain (Δ) at complete saturation in Step 3.
5. Using Equation 5, compute the pore pressures (u = P, neglecting surface tension) for each of the values of Δ computed in Step 2, and for complete saturation in Step 3.
6. For each value of pore pressure computed in Step 5, add the respective value of mean normal effective stress to obtain the associated mean normal total stress.

As discussed in *Part 1*, the application of each successive lift during embankment construction will generate a pore pressure increment in the underlying soil. The following discussion explains how this pore pressure increment can be estimated for an unsaturated soil from the relationship between pore pressure and mean normal total stress developed using Hilf's method as illustrated in the example spreadsheet. Referring to curve number 2 in Figure 3, or the actual curve presented in the example spreadsheet, note that each pore pressure increment (Δu) can be computed simply as the product of the mean normal total stress increment (Δp) and the slope of the curve (m) depending upon the magnitude of the mean normal total stress existing at the time the stress increment is applied. As the magnitude of the mean normal total stress increases so does the degree of saturation in the soil until the soil becomes fully saturated, beyond which point the pore pressure increment simply equals the mean normal total stress increment. Thus, the coefficient 'b' in Equation 4 can be interpreted as the slope (m) of straight line segments used to define the curve describing the relationship between pore water pressure (u) and mean normal total stress (p), and the following simple relation can be used to define the pore pressure increment:

$$\Delta u = m \, \Delta p \qquad (9)$$

Note that since the pore pressure increment depends on the value of the mean normal total stress that exists prior to application of each subsequent total stress increment, the mean normal total stress at each point under consideration (nodal point) must be tracked in the analysis. This will be discussed further in subsequent paragraphs. Referring back to Sheet 2 in the example spreadsheet, the following conditions will be applied in the analysis using conditional statements:

$$0 < p < 497 \qquad \Delta u = 0.664\,(\Delta p) \qquad (10a)$$
$$497 < p < 1{,}170 \qquad \Delta u = 0.752\,(\Delta p) \qquad (10b)$$
$$1{,}170 < p < 3{,}384 \qquad \Delta u = 0.849\,(\Delta p) \qquad (10c)$$
$$3{,}384 < p < 12{,}520 \qquad \Delta u = 0.927\,(\Delta p) \qquad (10d)$$
$$12{,}520 < p < 21{,}531 \qquad \Delta u = 0.967\,(\Delta p) \qquad (10e)$$
$$21{,}531 < p \qquad \Delta u = 1.000\,(\Delta p) \qquad (10f)$$

As a final point, it must be kept in mind that the approach suggested herein carries with it all the assumptions inherent in the development of the pore pressure expression proposed by Hilf; namely, lateral confinement and no drainage. Since the actual field conditions violate these assumptions, the results of the analysis can only be considered an approximation.

When applying Hilf's method, the atmospheric pressure should be corrected for altitude, decreasing by approximately 0.47 psi per 1,000 feet, up to an altitude of 15,000 feet.

RATE OF LOAD APPLICATION

Since the rate at which the pore pressures will dissipate is dependent in part on the rate at which the embankment is constructed, the embankment construction rate, expressed in feet per day, needs to be defined for the analysis, as discussed in *Part 1*. To develop this estimate it is necessary to define a typical embankment cross section and a generalized profile along the embankment alignment, such as those defined in the example problem and illustrated in Figure 1. From this information a spreadsheet can be developed to compute the incremental volume corresponding to each vertical foot of embankment. Knowing the volume of material that the contractor expects to be able to place each day permits one to compute the amount of time that will be required to construct each incremental foot of the embankment. If the contractor's production rate is not known, the author's experience has been that a typical rate of 8,000 to 10,000 cubic yards per day can be assumed for a large earthwork construction project, such as an earth dam.

For the example problem, the first foot of fill material from Elev. 20 to Elev. 21 (placed between Sta. 4+16 and Sta. 5+99) will have an average length along the alignment of 179.0 feet (at Elev. 20.5) and an average cross sectional width (corresponding to dimension B in Figure 2) of 657.0 feet. This yields a volume of 4,356 cubic yards of material required to complete the first foot of the embankment. With each additional foot of fill placed, the average length along the alignment will increase by 8 feet (for the 4(H):1(V) slope downstation and the 4(H):1(V) slope upstation), and the average cross sectional width will decrease by 6 feet (for the 3.0(H):1(V) side slopes). The last foot of fill material, from Elev. 124 to Elev. 125, will only require 1,236 cubic yards of material. Though the required volume of fill decreases with each additional foot above Elev. 65, the actual working area decreases while the distance that the construction equipment must travel along the embankment alignment increases. Consequently, it typically requires about the same amount of time for the contractor to place and compact material near the top of the embankment as it does at the base. The total volume of fill required for this example is estimated to be 626,807 cubic yards, which is a relatively small quantity of earthwork. Therefore, if it is assumed that the contractor can place about 6,000 cubic yards per day, the "computed" time required to place and compact each foot of embankment fill varies from 0.73 days for the first foot of embankment fill to a maximum of 1.29 days near mid height of the embankment, and a minimum of 0.21 days for the last foot, with an average of 0.99 days per foot. The fill placement rate (given in feet per day) is the inverse or reciprocal of the average time required to complete each vertical foot of embankment fill. The calculations are illustrated on Sheet 3 of the example spreadsheet, which also yields the estimate of the total volume of fill required for the embankment. Based on these calculations, a fill placement rate of 1 foot per day was selected for use in the example problem. In reality, it is unlikely that the embankment would be constructed at this rate owing to normal delays encountered during construction; however, the use of this rate will tend to yield a conservative result since a slower rate of construction should allow more pore pressure dissipation.

DETAILS OF THE ANALYSIS

The first step in the analysis is to develop the finite difference grid for the cross section to be analyzed. The points defined by this grid are the locations, or nodal points, at which the values for pore pressure will be computed. As discussed in *Part 1*, for problems where an axis of symmetry exists, such as the centerline of an embankment, it is only necessary to perform the analysis for one side of the problem. In the analysis for the core zone of a dam, it is convenient to define the x-coordinate in terms of the distance from the embankment centerline (centerline offset x), and the y-coordinate in terms of the height (y) above the foundation, or elevation. Within the spreadsheet, a block of columns and rows, referred to as a range, is respectively assigned centerline offsets (x) and heights (y). The specific cells within each range correspond to the nodes of the finite difference grid. Figure 4 illustrates the finite difference grid selected for analysis of the example problem. A uniform grid spacing of 5 feet ($\Delta x = \Delta y = 5$ ft) has been selected for the example problem.

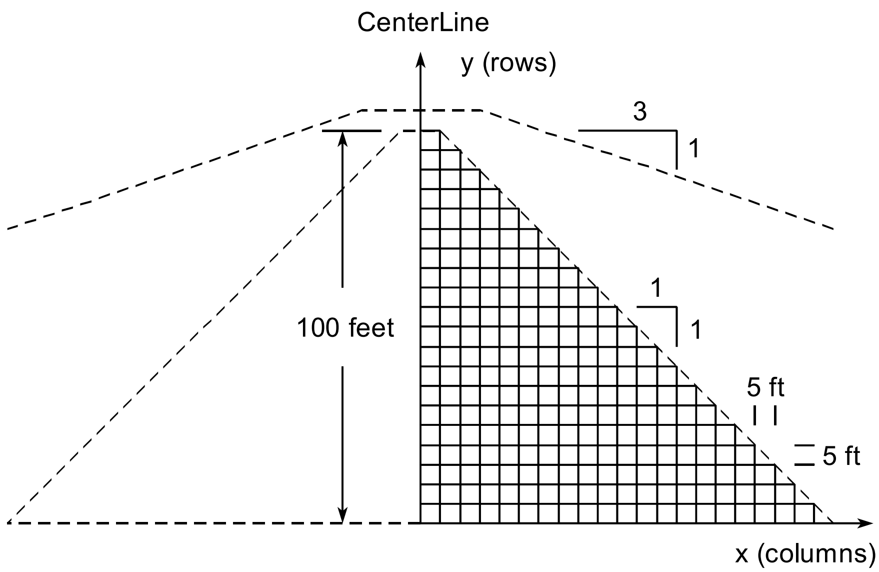

FIGURE 4: Finite difference grid for embankment core zone.

While only three such ranges were needed to perform the analysis for the example problem in *Part 1*, additional ranges are required when considering an unsaturated soil due to the need to track the mean normal total stress (p) at each nodal point in order to apply the appropriate pore pressure increment. A total of six ranges are used in the example spreadsheet. To assist the reader, the finite difference solution (*Part 1, Equation 10*) to the governing differential equation is provided below as Equation 11.

$$u(x_0, y_0, t+\Delta t) = u(x_0, y_0, t) + \frac{c_v}{(\Delta x)^2}\left[u(x_1, y_0, t) + u(x_3, y_0, t) - 2u(x_0, y_0, t)\right] + \frac{c_v}{(\Delta y)^2}\left[u(x_0, y_2, t) + u(x_0, y_4, t) - 2u(x_0, y_0, t)\right] + \left[\sigma(t+\Delta t) - \sigma(t)\right] \tag{11}$$

Summarized below are the “names” given to each of the six ranges in the example spreadsheet, the contents of each range, and the rows that the respective ranges occupy in the spreadsheet. Further details regarding these ranges are presented in subsequent sections. The advantage of naming ranges and cells in a spreadsheet is explained in *Part 1*. The reader may first wish to view the layout of the spreadsheet on Sheet 4 by selecting "View," then "Zoom," and choosing 25 percent, which will display the range names within their respective locations on the computer screen.

1- Range \u1 (Rows 23 through 43) values of the pore pressures at the nodal points prior to the time step Δt, (which correspond to the pore pressure terms on the right side of Equation 11),
2- Range \u2 (Rows 51 through 71) formulas in the form of Equation 11 that compute the pore pressure after the time step Δt (which correspond to the pore pressure term on the left side of the equation), using the values from Range \u1 and Range \du,
3- Range \dp (Rows 81 through 101) formulas in the form of Equation 2 that compute the mean normal total stress change (Δp) due to the placement of each lift of embankment material,
4- Range \du (Rows 109 through 129) conditional statements that compute the pore pressure increment (Δu) due to the placement of each lift of embankment material (which corresponds to the last expression [σ(t + Δt) - σ(t)] on the right side of Equation 11), using information from Equation 10a through 10f and Range \dp.
5- Range \p1 (Rows 137 through 157) values of the mean normal total stress (p) at each nodal point prior to the time step Δt and application of the mean normal total stress change (Δp), (this is simply the cumulative mean normal total stress change during the analysis),
6- Range \p2 (Rows 165 through 185) formulas (of the form p+Δp) that compute the mean normal total stress (p) at each nodal point after the time step Δt and application of the mean normal total stress change (Δp), using information from Range \p1 and Range \dp.

The next step in the analysis is to define the boundary conditions. For the two-dimensional problem considered herein, the conditions along four boundaries must be specified in the model. These boundaries can be either free draining or no flow boundaries. In the example problem, the foundation has been identified as an impervious boundary. Note, however, that a boundary need not be an impervious boundary to constitute a no flow boundary. An axis of symmetry, such as the centerline of the embankment, can also be a no flow boundary because the value of the pore pressure on the left side of the centerline would be equal to the value on the right side at any given elevation; consequently, the total head on both sides of the centerline will be equal throughout the analysis and as a result there will be no flow across the centerline. Therefore, the way to treat a no flow boundary is to modify the finite difference expression (Equation 11) for the nodal point(s) that lie on the boundary by simply using the value of the pore pressure at the nodal point just inside the boundary twice in the equation, since there is no nodal point beyond the boundary, but if there were it would have the same value as the interior point. This is illustrated in the spreadsheet for the example problem for both the lower impervious boundary and centerline boundary (axis of symmetry). Note also that the nodal point at the bottom of the core zone, along the centerline, lies on two no flow boundaries. To assist the reader in examining the variations in the finite difference expression, in Range \u2 containing the finite difference formulas for the pore pressure at time t+Δt, the cells that correspond to nodes that lie along the boundaries are shown shaded in different colors depending on the type of boundary along which the nodal point lies.

Since by definition no excess pore pressures develop at a drainage boundary, the value for the excess pore pressure at all points along such boundaries is set at zero. The downstream surface of the core zone (where a drainage zone is typically constructed) is expected to be a drainage boundary; therefore, a zero value is set at all nodal points along this boundary. The fourth and final boundary is the top of the fill itself, which is a **MOVING DRAINAGE BOUNDARY** because its location is changing throughout the construction process. At first glance this might appear to be a major challenge; however, the problem has a very simple solution. Note that during construction the nodal points above the current lift simply do not yet exist; that is, they have not yet been "built." Consequently, they can only have a zero value associated with them, and it is not until the elevation of the fill rises above the respective nodal points that actual pore pressures can begin to develop at these locations. Therefore, the problem is handled by a simple conditional statement expressing the fact that, "IF" the elevation of the nodal point is greater than or equal to the elevation of the fill, "THEN" the nodal point has a zero value, which essentially makes it a drainage boundary, "ELSE" the pore pressure at the nodal point is defined by the finite difference equation (Equation 11). The reader can examine this conditional statement in any of the cells within Range \u2 with the exception of the downstream drainage boundary. The reader will also find that when examining the contents of a cell containing a formula in an Excel spreadsheet, a "double click" of the computer mouse will (in addition to displaying the formula on the screen) illuminate all cells linked to the formula in that cell, which may help the reader in understanding the subscript notation in Equation 11 as it correlates to the actual spreadsheet. Owing to the size of this particular spreadsheet, it may also be helpful to "Zoom" out to 50 percent and then scroll up or down to locate the respective cells within the spreadsheet that are linked to the cell containing the formula.

With the physical limits of the problem defined in terms of the four respective boundaries, the next step in the solution is to define an "initial condition" at each of the nodal points. As previously discussed, the initial condition at each nodal point cannot be anything other than zero until the elevation of the embankment exceeds the elevation of the respective nodal points.

While the physical problem is three-dimensional in space and time, the spreadsheet is only two-dimensional. This limitation is overcome by using a macro to create a loop that functions in the same manner as a DO LOOP in a conventional computer program, where each execution of the loop corresponds to an increment in time (Δt). Figure 5 shows the basic spreadsheet flow diagram for the solution and may assist the reader in following the discussion. In the following discussion the terms in brackets [ ] refer to the name of the range in the spreadsheet for the example problem. The reader may note that when the cursor is placed over a cell that has been named, the name appears in the "Name Box" at the top left corner of the spreadsheet, below the tool bar.

To create the loop, one range [\u1] of cells in the spreadsheet is assigned the pore pressure values before the time step, for each of the nodal points. The cells within this range contain numerical values of the pore pressure at the respective nodal points at time t, identified in Cell G15 [TIME], prior to placement of the lift, which correspond to the terms on the right side of Equation 11, of the form $u(x_i, y_j, t)$. Another range [\u2] of cells within the spreadsheet contains the finite difference formula that computes the pore pressure at time $t+\Delta t$, for each corresponding node in the first range [\u1]. These formulas are developed from the finite difference equation, using relative cell addresses, and the cells within this range correspond to the pore pressure terms on the left side of the finite difference equation, of the form $u(x_i, y_j, t+\Delta t)$. The next range [\dp] contains formulas, based on Equation 2 and the embankment geometry, that compute the mean normal total stress change (Δp) arising from the placement of each successive lift of material as the embankment is constructed. These values of mean normal total stress increment (Δp) are used to calculate the pore pressure increase (Δu) at each nodal point in the fourth range [\du]. In the case of an unsaturated soil the pore pressure increment (Δu) depends not only on the mean normal total stress change (Δp), but also on the value of the mean normal total stress (p) prior to placement of the lift, which is tracked in the fifth range [\p1]. The pore pressure increase (Δu) at each nodal point is computed in the fourth range [\du] using a series of conditional "IF" statements that apply Equations 10a through 10f, developed using Hilf's method as illustrated in Sheet 2 of the example spreadsheet. The sixth and final range [\p2] calculates the cumulative mean normal total stress at each nodal point subsequent to the placement of each successive lift, which are transferred into the fifth range [\dp] prior to the next iteration.

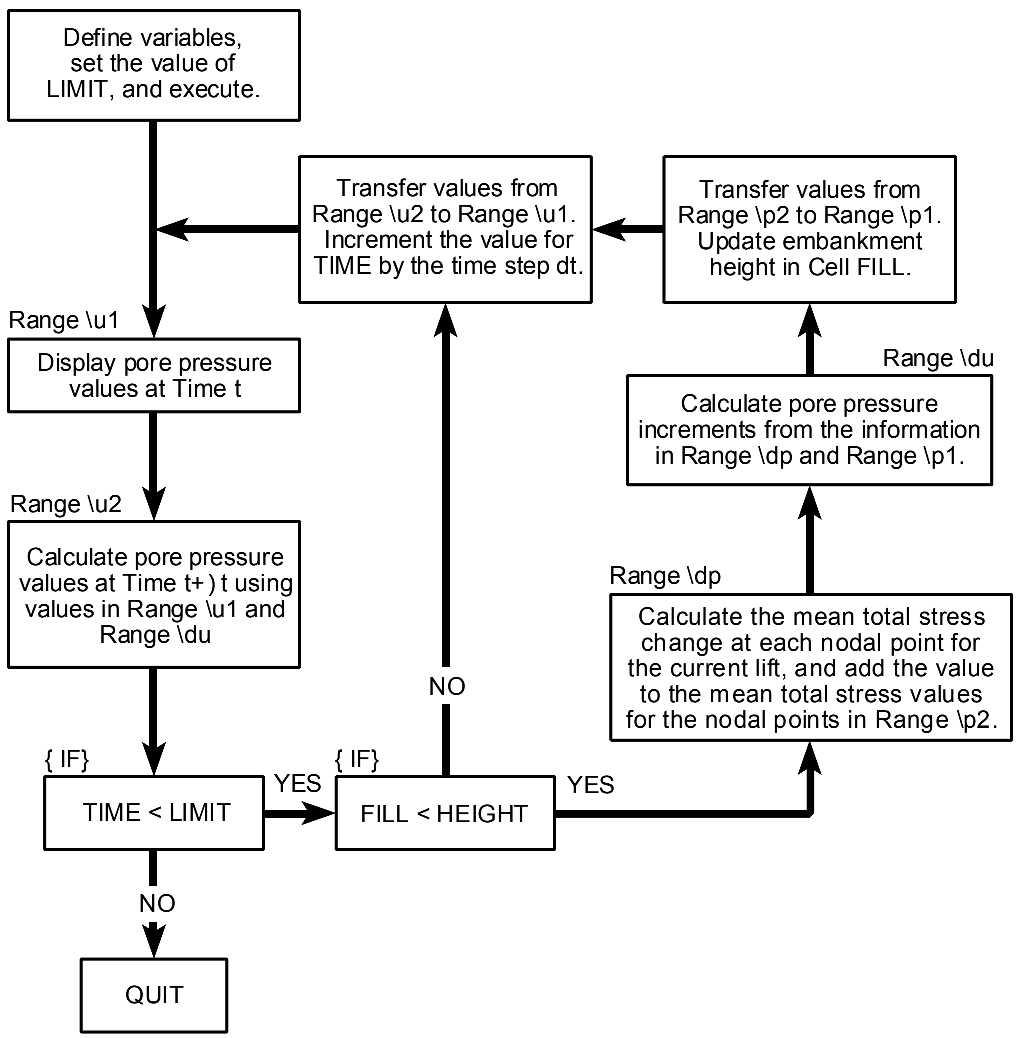

FIGURE 5: Spreadsheet Flow Diagram

The spreadsheet sequences through the analysis as follows:

a) the lift thickness [LIFT] is computed as the product of the time increment [dt] and fill placement rate [RATE] specified by the designer;
b) the vertical stress increase [W], corresponding to the strip load (w) is computed as the product of the embankment fill weight [WEIGHT] and lift thickness [LIFT] (refer to *Part 1, Figure 2*).
c) the computer automatically recalculates all formulas in the spreadsheet with this new cell entry; therefore,
    1) the mean normal total stress increment (Δp) is computed in Range \dp,
    2) the pore pressure increment (Δu) is calculated at each nodal point below the current fill elevation, using the mean normal total stress increment (Δp) in Range \dp and the mean normal total stress at each nodal point in Range \p1, and added to the pore pressure values in Range \u2, and
    3) the mean normal total stress at each nodal point is recomputed in Range \p2 for the end of the time step;
d) the pore pressures are computed in Range \u2 for each nodal point, using the pore pressures prior to the time step from Range \u1 and the pore pressure increments (Δu) from Range \du;
e) the pore pressures in Range \u2 are transferred as values into Range \u1;
f) the mean normal total stresses in Range \p2 are transferred as values into Range \p1;
g) the height of the fill [FILL] is incremented by the lift thickness [LIFT] and the half width (refer to Figure 2) of the next lift [B/2] is computed based on the lift thickness and side slope [SLOPE];
h) the current time [TIME] is incremented by the time step Δt [dt], and the sequence is reinitiated until the specified time limit [LIMIT] is reached.

When execution is terminated, the pore water pressures displayed in Range \u1 are those corresponding to the current time [TIME] and embankment fill height [FILL] displayed in those respective spreadsheet cells. However, the values displayed in Ranges \u2, \dp, \du, and \p2 will actually apply to the next iteration, since the cells linked to the formulas in these ranges changed during the current iteration and the spreadsheet is set to automatically recalculate all of the formulas in these ranges.

In defining the size of the finite difference grid, the distances between nodal points (Δx and Δy) is quite important. Note that the values of Δx and Δy must be chosen so as to satisfy the stability and convergence criterion defined in *Part 1, Equation 11*. The author finds it preferable to specify Δx and Δy and compute a value for the time step (Δt) that ensures convergence and stability of the finite difference equation. While Δx and Δy do not have to be equal, setting them equal simplifies Equation 11 by allowing terms to be combined as was done in the spreadsheet for the example problem. A value of Δx = Δy = 5 feet was used in the example in order to minimize the size of the spreadsheet file while still illustrating the important points of the analysis. Thus, from *Part 1, Equation 11* (stability and convergence criteria), using $c_v = 0.80$ square feet per day, the maximum time step (Δt) must not exceed 7.8 days for the example problem in order to ensure convergence and stability of the finite difference solution. By selecting a value for the time step (Δt) so as to limit the strip load (w) to a value corresponding to no more than the thickness of a construction lift, and decrementing the width of the lift (dimension B in *Part 1, Figure 2*) after each time step prior to the application of the next load increment, it is possible to simulate construction of the actual embankment geometry, as illustrated in Figures 1 and 2, for any construction sequence that the designer may wish to examine. Setting $\Delta t = 1$ day corresponds nicely with the fill placement rate of 1 foot per day selected for the example problem and will more than satisfy the stability and convergence criteria. Thus, the analytical solution assumes that 12 inches of fill (a reasonable assumption for an uncompacted lift of fill material) will be placed at the end of each time step (Δt) of 1 day. Unlike the example problem in *Part 1*, however, the earth structure in the example for Part 2 involves materials with different total unit weights. The core zone material will have an in-place total unit weight of 123.2 pcf while the embankment shell material will have an in-place total unit weight of 130 pcf. Using the larger value for the analysis will result in higher estimates for the computed pore pressure increments and attendant end of construction pore pressures, which should be conservative and prudent in light of the assumptions inherent in the proposed method. Therefore, the vertical stress increase (w) for the uniform strip load (lift) in Equation 2 becomes 130 psf (pounds per square foot) for the example problem.

Part 1, Figure 6 illustrates the geometric relation between the location of a nodal point (coordinates x and y) and the position of a lift of material applied to the embankment (distance z). Each nodal point is fixed in space by its x and y coordinates, which remain constant during the analysis. The distance z is always measured from the bottom of the lift to the nodal point, which varies with the position of each respective lift in the embankment. The variable z is tracked in Column B of Range \dp, and it is naturally referenced to the top of the foundation where the first lift is placed. Thus, for each successive lift the variable z is incremented by the lift thickness [LIFT]. A conditional statement is used in the formulas for the variable z in the spreadsheet, since no value can be assigned to the variable until the elevation of the embankment rises above the elevation of the nodal points in Range \dp.

The reader should pay particular attention to the manner in which the variables B/2 and LIFT are computed in the spreadsheet. The variable B/2 is located in Cell B15, which contains a simple algorithm that computes the parameter based on the geometry of the embankment and the position of each respective lift, which is tracked by the variable FILL in the example spreadsheet. The value of the variable LIFT is the product of the fill placement rate [RATE] (which was determined to be approximately 1 foot per day for the example problem) and the time step [dt] (which was chosen to be 1 day for this problem). A conditional statement has been placed in the cell containing the variable LIFT that compares the value of the variable FILL to the completed embankment height [HEIGHT] and sets the value of LIFT to zero once the embankment height has been reached in the model. Thus, for any value of time beyond completion of the embankment, the analysis simply tracks the dissipation of pore pressures.

The macro commands that execute the analysis are shown in red to the right of the variables at the top of Sheet 4 in the example spreadsheet; however, the actual macro resides in a Visual Basic Module. Pressing the keys [Alt] and [F8] will open the dialogue box for the module. The macro in the example spreadsheet was given the name "Execute," and clicking on the EDIT button opens the macro. The reader may note that the use of range names makes the logic used in the macro commands much easier to understand. In addition, Excel allows the reader to insert comment statements (which appear in green) anywhere within the macro itself. At the heart of the spreadsheet model is the macro command, "Range("\u1").Value = Range("\u2").Value," which updates the simultaneous process of pore pressure generation and dissipation by copying the results of the equations in Range \u2 as values into Range \u1 for each iteration corresponding to a time step (Δt).

The designer will generally be interested in the magnitude of the pore pressures in the core zone that are likely to exist at the end of construction in order to evaluate the stability of the dam under these conditions. It is relevant to note that the most critical post construction condition typically will occur under conditions of a partial pool in the reservoir; however, this is beyond the scope of this paper. Nevertheless, the method proposed herein allows the designer to estimate the pore pressures at any point during and subsequent to construction. For example, if it is determined from slope stability analyses that unacceptably high pore pressures are expected to exist when the embankment is topped out, the model allows for the analysis of a staged construction to determine if it might be possible to control end of construction pore pressures in this manner. The use of a conditional statement in the cell containing the value of the variable LIFT allows the designer to model stage construction to permit some dissipation of pore pressures before the embankment is built to its final height. The following illustrates this approach for the example problem. Let us assume that the embankment will first be constructed to approximately half of its height, or 50 feet (which would require 50 days in this model). Construction will then be halted for 180 days, or approximately 6 months, after which fill will be placed until the embankment is completed (which will require 55 more days). Placing a conditional statement of the following form in the cell for the variable LIFT will simulate this sequence of events.

```
=IF(FILL<HEIGHT,IF(TIME<=50,RATE*dt,IF(TIME>230,RATE*dt,0)),0)
```

Observe the values in Range \u1, LIFT, and FILL as the spreadsheet executes this analysis. FILL varies from 0 to 50 feet in the first 50 days, remains constant at 50 feet from 50 to 230 days, increases to 105 feet from 230 to 285 days, and remains constant for any time greater than 285 days.

Depending on the speed of the reader’s computer, the analysis may execute too quickly for the reader to visually follow what the spreadsheet is doing. Therefore, the reader may wish to sequence through the analysis by resetting the value of LIMIT during the analysis. First set the value of LIMIT to 50 and execute the analysis, then reset the value of LIMIT to 230 and execute again, and finally reset the value of LIMIT to 285 and execute a final time.

Note that the solution employs a forward marching technique, which is to say that the analysis can be stopped at any point by simply specifying the value of LIMIT; however, the value must always be increasing in time. For example, in the case of the stage construction previously described the designer can set the value of LIMIT at 50 days and run the analysis. The value can be reset to 230 days, the macro rerun, and the spreadsheet will continue the analysis up to the specified value of LIMIT. Similarly the value of LIMIT can be reset to 285 days, 365 days, etc.; however, the LIMIT cannot be set to one value (say 285 days), the macro executed, and the LIMIT reset to a lesser value (such as 50 days). Just as in the real world, time only moves in the forward direction. However, resetting the spreadsheet for the example problem is a very simple process. Simply delete all the values in Range \u1 and Range \p1 (which has the same effect as setting them to zero), reset the values in cells FILL and TIME to zero, and reset the desired value of LIMIT.

The main purpose of performing the analysis will normally be to obtain information for use in a slope stability analysis of the embankment upon completion of construction. Consequently, the information needs to be presented in a form suitable for use in a computerized slope stability analysis. Toward that end, the last page of the spreadsheet, Sheet 5, computes values of the pore pressure ratio (r_u) at each nodal point for the designer's use in a slope stability analysis. The pore pressure ratio is computed in the spreadsheet in the same manner as it is computed in most slope stability computer programs, which is simply the ratio of the pore pressure (u) to the total vertical stress (σ_v) at the point under consideration [2, 9]. That is:

$$r_u = \frac{u}{\sigma_v} \tag{12}$$

Note that in this calculation, no consideration is given to the distribution of stress that occurs in the actual embankment as was done in the analysis on Sheet 4. Rather, the calculation is performed in the same manner as it will be executed in the slope stability analysis, where the total vertical stress is computed simply from the height (h) of a vertical column of soil directly above the point under consideration (i.e., $\sigma_v = \gamma h$). The value of the pore pressure ratio should always be less than one when dealing with construction generated pore pressures. A value greater than one, however, could conceivably be "computed" depending on the assumptions used by the designer and the limitations inherent in the model. For example, the pore pressures computed in Sheet 4 are based on a unit weight of 130.0 pcf for the embankment material, due to a limitation inherent in this particular analytical model; however, the pore pressure ratios computed in Sheet 5 use the actual unit weights of 130.0 pcf for the shell zone material and 123.2 pcf for the core zone material because these are the values that will be used in the computerized slope stability analysis. Consequently, it is important that the designer not only understand how to run any particular computer model, but the designer must also understand the limitations associated with the model if he/she is to properly interpret the results. Some of the limitations of the analytical model proposed herein are presented in a subsequent section.

OBSERVATIONS ON THE EXAMPLE PROBLEM

If the reader analyzes the example problem for stage construction as suggested above using the companion spreadsheet to this paper, he/she will notice that the 6 month delay had no real significant effect on the pore pressures; however, a 6 month delay is extremely significant in terms of the actual construction schedule for a dam. The most significant effect in terms of pore pressures is seen at the bottom of the core zone along the centerline of the embankment, which is not a critical location in the slope stability of the embankment. At this location the value of the pore pressure changes from a value of 7,942 psf to a value of 7,416 psf, which is a reduction of less than 7 percent. Consequently, the reader may have cause to wonder about either the validity of the model or the value of stage construction, or both. Consider, however, that the primary criterion in selecting a material for the core zone of a dam is its low permeability; therefore, it should not come as a surprise that pore pressures may dissipate very slowly both during and subsequent to construction. The results of the model simply reflect the characteristics of the material that is being modeled. In this example, it is reasonable to conclude that staged construction is not a practical option, and the designer needs to consider another approach if end of construction pore pressures are at issue. In the case of foundation pore pressures, as addressed in *Part 1*, the designer might wish to install vertical drains to facilitate the process of pore pressure dissipation; however, installing drains in the core zone of a dam essentially defeats the purpose of the core zone. Consequently, the designer might wish to reconsider the geometry of the core and/or shell zones. In either case, the model is functioning as the design tool that it is intended to be.

Alternatively, the designer may wish to consider the potential effect of lowering the degree of saturation on the pore pressures generated during construction, which is achieved by limiting the moisture content during compaction. Referring back to Sheet 2 of the example spreadsheet, the reader will note that changing the water content (w) from 20.0 percent to the optimum moisture content of 17.5 percent reduces the degree of saturation from 84.3 percent to 73.8 percent. The reader would have to make some further minor revisions to the spreadsheet in order to develop a new relationship between pore pressure and mean normal total stress using Hilf's method as described herein. If one were to do so, then revise the formulas in Range \du and execute the analysis it would be seen to have a dramatic effect on the end of construction pore pressures. For example, the estimated pore pressure at the bottom of the core zone along the centerline of the embankment decreases from a value of 7,942 psf to a value of 6,105 psf, which is a reduction of about 23 percent. The reader may be aware that, for this reason two schools of thought have evolved in the area of dam design and construction. One school of thought advocates limiting the compaction moisture content in order to limit end of construction pore pressures. The other school of thought contends that a wet core zone is likely to be more flexible and therefore less prone to cracking despite the potentially high end of construction pore pressures, and the embankment geometry is developed around this limitation. Irrespective of the opinion that the reader may have on this issue, the author hopes he/she will agree that the ability to numerically model the problem is advantageous to the designer.

If the reader analyzes the example problem for stage construction, he/she may have observed that while pore pressures decreased in the lower regions of the core zone, they continued to increase for a time at other locations despite the fact that additional loads were not being added to the embankment. Once again, the reader may have cause to wonder about the validity of the model. Consider, however, what must actually be happening in the core zone. Pore pressures in the lower regions of the core zone dissipate as fluid flows out of the soil pore space, and fluid flows from regions of higher total head to regions of lower total head. Since the elevation head at any point in the core zone remains constant (except for the actual effects of settlement during consolidation, which are being neglected in this model), then only the pressure head (which is directly related to pore pressure) is subject to change. Accepting this fact, however, the reader may still argue that while water must certainly flow into an element of soil within the core zone from adjacent elements possessing higher total heads, water is simultaneously flowing out of that element of soil as it too drains. Consequently, must the pore pressure necessarily increase as water passes though an element of soil? This all depends on the relative magnitude of the pore pressure at adjacent nodal points in the finite difference analysis. It may be helpful for the reader to refer to *Part 1, Figure 4*, which illustrates the reference system for nodal points in the finite difference grid. More convincing, however, is the fact that evidence of this phenomenon occurring in a dam was obtained from measurements made during construction of a well instrumented, large zoned earth and rockfill dam where pore pressures were actually observed to continue to increase during a particular period during construction when the contractor was unable to continue fill placement for several months [5]. The reader who wishes to examine a portion of this data can download the file "phd-toc.pdf" from the author's web site, which illustrates the construction pore pressures at piezometer P-6 for the referenced dam. The line illustrated in that same figure, identified as "One-Dimensional Model" reflects the actual construction sequence from which it can be seen that embankment construction was delayed for a period of time around March 1986; however, the field data clearly shows that pore pressures monitored by this piezometer continued to increase. Similar behavior was observed at several other piezometers.

LIMITATIONS

While the author believes that the approach presented in this paper is a useful method for analyzing the problem of construction generated pore pressures, it is important for the reader to be aware of the limitations inherent in this approach. It is hoped that this will not only assist the designer in correctly applying the method, but also in interpreting any field data collected during project construction. Regarding the various limitations associated with the method, the reader is referred to this same section in *Part 1* as the comments presented in that section still apply and will not be reiterated here.

As noted in the section on pore pressure response in soil, it is not unreasonable to assume a value for Poisson's ratio of 0.5 for a fully saturated soil, since the soil is relatively incompressible until it drains. On the basis of this assumption the expressions given by Equations 2 and 3 can be developed for the total stress invariants p and q. In the case of an unsaturated soil, however, the soil skeleton would be compressible to some degree and the value for Poisson's ratio of 0.5 would not be strictly correct. Following the development for the stress invariants presented in *Part 1, Appendix A*, the reader can develop expressions for the stress invariants for any value for Poisson's ratio considered appropriate.

As previously noted herein, the subject of pore pressure in unsaturated soil is extremely complex, and the author does not mean to suggest anything to the contrary. For more information on this subject the reader may wish to refer to the work of Fredlund [3] and numerous others. Unfortunately, as the designer develops a better understanding of the complexity of the problem he/she may also become overwhelmed by not only the number of variables involved, but with the seemingly insurmountable task of attempting to quantify these variables for an actual problem. While the method proposed by Hilf may certainly be criticized as not being "state-of-the-art" technology, it should not be dismissed simply for that reason as it does provide a means of approaching the problem using data from a standard one-dimensional consolidation test that is routinely performed by the majority of soil testing laboratories at a reasonable cost. Despite the fact that the method does not address all of the variables currently recognized by the profession, very good agreement was obtained between actual field data and the relationship between vertical total stress (σ_v) and pore pressure (u) developed using Hilf's method for the core zone of a well instrumented large earth and rockfill dam [5], which can be seen in the data for total stress cell TSC-1 and piezometer P-10 presented in the file "phd-toc.pdf." This data confirms the fact that very little pore pressure dissipation actually occurs in the core zone, which is consistent with the observations on the example problem discussed in the previous section. This data also supports the assumption of essentially no drainage, which is one of the conditions associated with Hilf's method.

The issue of estimating the stress change in soil that will occur in response to an applied load is perhaps the greatest limitation of all solutions to problems in consolidation and settlement. It has long been recognized that soils are not totally elastic, though their behavior may be relatively elastic within certain stress ranges. Consequently, we must accept that our methods can only be considered approximate when applied to earth structures and/or natural stratified deposits of soils with differing stress-strain properties. Even a finite element solution should be considered approximate, since the values for the Elastic Modulus and Poisson's Ratio are not only difficult to determine, but actually change during the consolidation process. Therefore, in the author's opinion, our ability (or inability) to predict the stress changes that will occur in the earth structure in response to a surface load is perhaps the greatest limitation in our current solutions to the problems of consolidation and settlement. As previously noted, the method proposed herein violates the boundary conditions associated with the solution for a strip load applied to a linearly elastic half space. Actual horizontal stresses in the embankment are expected to be less than assumed in the model since the soil is free to strain laterally in the upstream and downstream directions, though the core zone will be somewhat confined by the upstream and downstream shell zone. Despite this limitation, however, the method yielded results that compared quite well to the data collected for a well instrumented large earth and rockfill dam [5], a portion of which can be seen in the data for piezometer P-6 presented in the file "phd-toc.pdf" that can be downloaded from the author's web site.

Fortunately computer models lend themselves well to sensitivity analyses and the spreadsheet is no exception. Once the spreadsheet model has been built it is a simple matter to change one or more of the variables and rerun the analysis to establish potential maximum and minimum values for the pore pressures.

CONCLUDING REMARKS

A simple model has been proposed for estimating pore pressures generated in the core of a zoned embankment, constructed with unsaturated compacted clay. The model provides a method for considering the stress distribution within the embankment along with a two-dimensional finite difference algorithm that addresses the simultaneous generation and dissipation of pore pressures that occur during construction. The primary advantage of the model is that it involves relatively simple algorithms that lend themselves well to solution using commercially available spreadsheet software run on a personal computer. Despite its limitations, pore pressures estimated using this simple model have been found to agree reasonably well with actual values observed during the construction of a well instrumented large zoned earth and rockfill dam, with the error being on the conservative side; that is, the actual observed values were less than the predicted values.

As stated in the opening paragraphs of this paper, it is the author's hope that engineers will attempt to apply the methods described in these papers, with care and good judgment, to the design of projects that are sufficiently instrumented to ensure that pore pressures generated during construction do not exceed the values predicted by the model. While the method shows promise as a design tool, additional comparisons should naturally be made between the proposed model and actual field data collected from other projects. A larger body of comprehensive field data, consisting of total stress, pore pressure, and settlement measurements, will be required to develop a better understanding of actual soil behavior, which will ultimately lead to safer, more cost effective designs. Though the primary purpose of geotechnical instrumentation should be to verify the design assumptions and identify potential problems before the structure experiences significant distress, the data collected to meet these objectives also provides the best means of developing a better understanding of the actual behavior of earth structures and soil foundations. Since clients will naturally be reluctant to finance what they may perceive as research, the designer must communicate the importance of instrumentation to the client by pointing out that a more cost effective design can be achieved if the need for overly conservative assumptions can be eliminated; however, this requires verification of actual performance in the field. To maximize the effectiveness of the instruments, the project should also call for index property tests of the material in the immediate vicinity of the instruments, as well as shear strength and consolidation tests of the respective materials in the embankment and/or foundation, which will provide the information necessary for application to analytical models.

The author would sincerely appreciate any suggestions or comments on this paper and/or the spreadsheet for the example problem from those engineers who attempt to use this model. The author would also be particularly interested in the details of any projects where instrumentation data collected in the field has been, or is being compared to values predicted using the model. As a final note, the author strongly urges individuals not to attempt to modify the example spreadsheet to fit any problems on which they are working, but rather to develop their own spreadsheet solution. Creating a spreadsheet not only helps the designer to better understand the underlying technical principles, but also minimizes the potential for errors that frequently arise when attempting to use someone else's spreadsheet.

ACKNOWLEDGMENTS

The author wishes to express his sincere appreciation to Ms. Marie J. Starich. P.E., of Solum Engineering, Inc., Houston, Texas; and Mr. Jeffrey B. Fassett, P.E., of Golder Associates Inc., Houston, Texas, for their time in reviewing this paper and associated spreadsheet, as well as for their valuable comments and suggestions.

DISCLAIMER

The spreadsheet solution to the example problem is made available over the Internet as a companion document to the paper titled "A Simple Model For Estimating End Of Construction Pore Pressures: Part 2 - Embankment Pore Pressures," by David J. Kerkes. The spreadsheet provides a solution to the example problem discussed in the paper and is intended for that purpose and no other. This spreadsheet is not intended to serve as a template for use by individuals in the development of solutions to their own specific problems. While considerable effort has been spent to ensure that the spreadsheet is free of errors and computer viruses, the author does not warrant that the spreadsheet is error or virus free. Individuals should be particularly cautious when using copies of the spreadsheet not obtained directly from the author's web site. The decision to place any reliance on the method illustrated in this spreadsheet, as well as any conclusions drawn from the results, are ultimately the responsibility of the designer.

The date is provided on Sheets 1 through 5 of the spreadsheet and individuals may wish to periodically visit the author's web site for updated copies.

REFERENCES

1. Atkinson, J. H. & Bransby, P.L., The Mechanics of Soils: An Introduction to Critical State Soil Mechanics, McGraw-Hill Book Company (UK) Ltd., London, 1978

2. Bishop, A. W. & Bjerrum, L., "The Relevance of the Triaxial Test to the Solution of Stability Problems," Proceedings of the ASCE Conference on Shear Strength of Cohesive Soils, Boulder, Colorado, June, 1960

3. Fredlund, D. G., & Rahardjo, H., Soil Mechanics For Unsaturated Soils, John Wiley & Sons, Inc., New York, 1993

4. Hilf, J. W., "Estimating Construction Pore Pressures in Rolled Earth Dams", Proceedings of the 2nd International Conference of Soil Mechanics and Foundation Engineering, Rotterdam, 1948, Vol III

5. Kerkes, D. J., "Analysis and Prediction of Stresses and Pore Pressures Associated with Wet Core Construction for Embankment Dams," a thesis submitted to the faculty of the Graduate School of the University of Colorado in partial fulfillment of the requirements for the degree of Doctor of Philosophy, May 1990

6. Kerkes, D. J., "A Simple Model For Predicting Construction Pore Pressures In A Zoned Embankment," Dam Engineering, Vol. IV, Issue 2, 1993

7. Kerkes, D. J., "Spreadsheet Solution For Transient Fluid Flow In A Porous Media With A Moving Drainage Boundary," Proceedings of the First International Conference on Computational Modeling of Free and Moving Boundary Problems, Southampton, England, July 1991

8. Skempton, A. W., "The Pore-Pressure Coefficients A and B," Geotechnique, Vol. 4, No. 4, 1954

9. Winterkorn H. F., & Fang, H-Y, Foundation Engineering Handbook, Chap. 10: "Stability of Earth Slopes,", Van Nostrand Reinhold Co., New York, 1975