rocscience

Slide2

Groundwater

Verification Manual

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1. Shallow unconfined flow with rainfall

1.1. Problem Description

The problem considered in this section involves the infiltration of water downward through soil. It is characterized by a boundary of flow domain also known as a free surface. Such a problem domain is said to be unconfined.

Water may infiltrate downward through the soil due to rainfall or artificial infiltration. Rainfall can be presented as a uniform discharge *P* (m/s), defined as the amount of water per unit area that enters the aquifer per unit time. Figure 1.1 shows the problem of flow between two long and straight parallel rivers, separated by a section of land. The free surface of the land is subjected to rainfall.

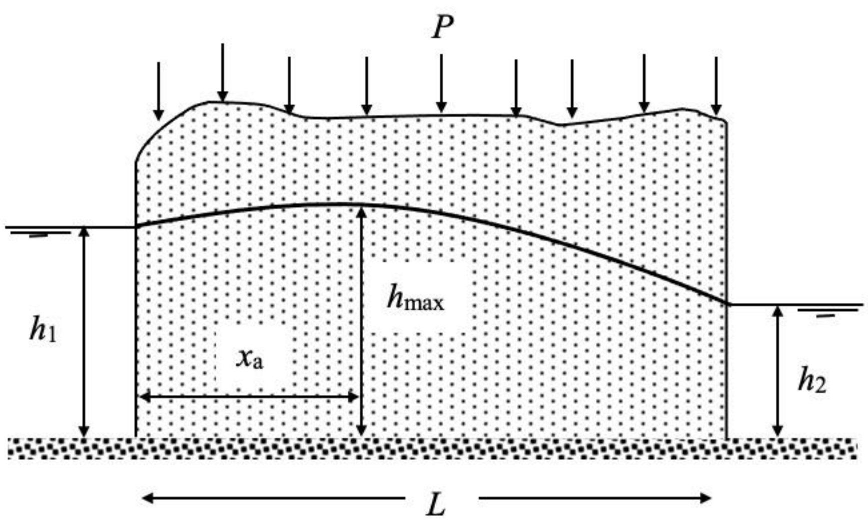


Figure 1.1: Model geometry

The equation for flow can be expressed as:

(1.1) $$\frac{\partial^2 \phi}{\partial x^2} + \frac{\partial^2 \phi}{\partial y^2} = \nabla^2 \phi = -P$$

For one-dimensional flow, such as that encountered in the present example, solution of equation (1.1) after application of the appropriate boundary conditions yields the horizontal distance, x_a, at which the maximum elevation of the free surface in Figure 1.1 is located, as [1]:

(1.2) $$x_a = \frac{L}{2}\left(1 - \frac{k}{P}\frac{h_1^2 - h_2^2}{L^2}\right)$$

The corresponding maximum height for the free surface, h_{max}, can be calculated as:

(1.3) $$h_{max} = \sqrt{h_1^2 - \frac{x_a}{L}(h_1^2 - h_2^2) + \frac{P}{k}(L - x)x}$$

1.2. Slide Model and Results

The **Slide** model for the problem is shown in Figure 1.2.

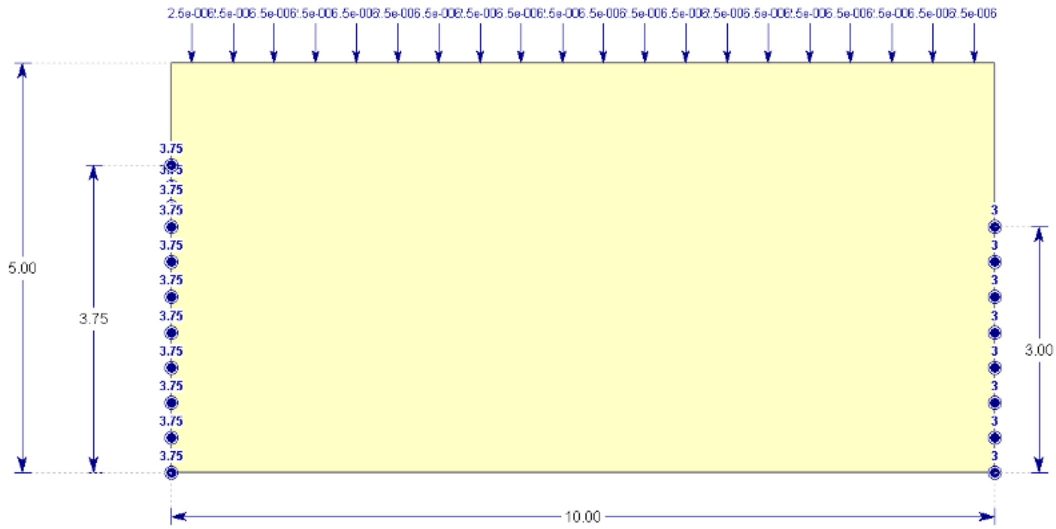


Figure 1.2: Slide Model

The **Slide** model uses the following input parameters:

- h_1 = 3.75m, h_2 = 3.0m (flow heads at the river boundaries),
- L =10.0m (separation between the rivers),
- P = 2.5e-6 m/s (rate of discharge), and
- k = 1.0e-5 (hydraulic conductivity)

The problem is modelled using three-noded triangular finite elements. The total number of elements used was 225 elements.

Figure 1.3 shows contours of pressure head with the coordinates (x_a, h_{max}) of point at which the maximum height of the free surface occurs.

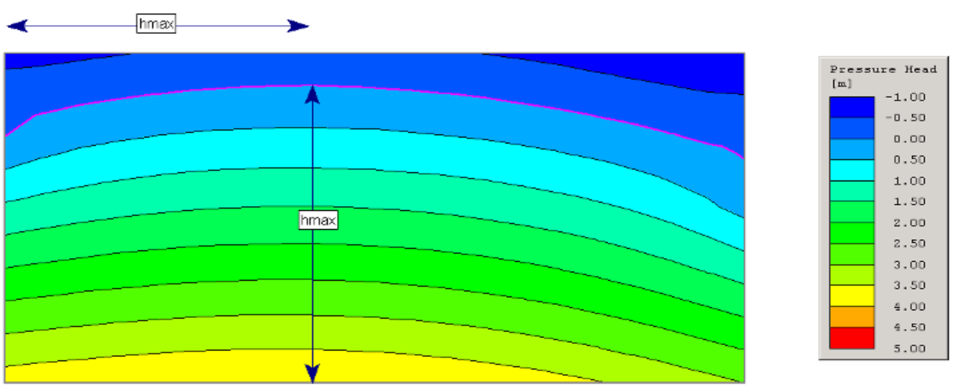


Figure 1.3: Pressure head contours

The table below compares the results from **Slide** with those calculated from equations 1.2 and 1.3

Table 1.1

Parameter	Slide	Equations (1.2 – 1.3)
x_a	4.06	3.98
h_{max}	4.49	4.25

The **Slide** results are in close agreement with the analytical solution. If necessary, a finer mesh discretization could be used to improve the results of Slide.

1.3. References

1. Haar, M. E. (1990) *Groundwater and Seepage*, 2nd Edition, Dover

Note: See file *Groundwater#01_1.sli* (regular mesh), *Groundwater#01_2.sli* (uniform mesh)

2. Flow around cylinder

2.1. Problem Description

This example examines the problem of uniform fluid flow around a cylinder of unit radius as depicted in Figure 2.1.

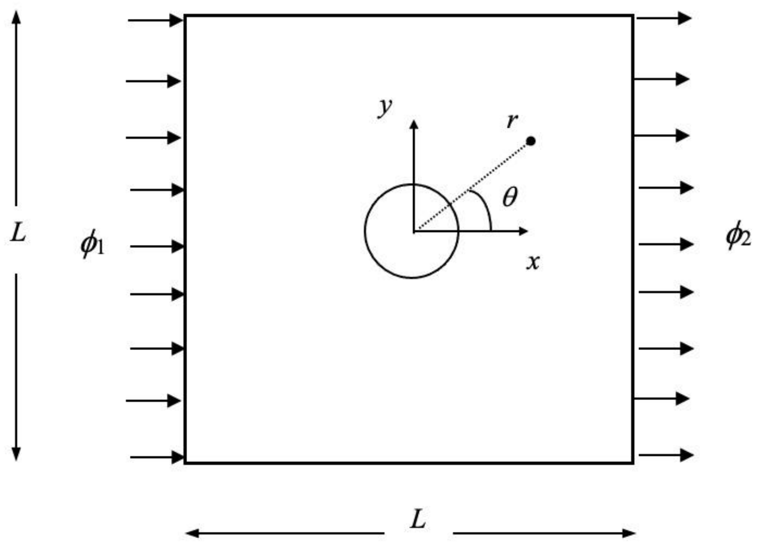


Figure 2.1: Model geometry

The closed form solution for this problem is given in Ref. [1]. This analytical solution gives the total head values at any point in the problem domain as:

(2.1) $$\phi = U\left(r + \frac{a^2}{r}\right)\cos\theta + 0.5$$

where *U* is the uniform undisturbed velocity = $\frac{\phi_1 - \phi_2}{L}$, $r = \sqrt{x^2 + y^2}$ and *a* is the radius of cylinder, and θ is the anti-clockwise angle measured from the *x* axis to the field point.

2.2. Slide Model and Results

The **Slide** model for the geometry is shown in Figure 2.2.

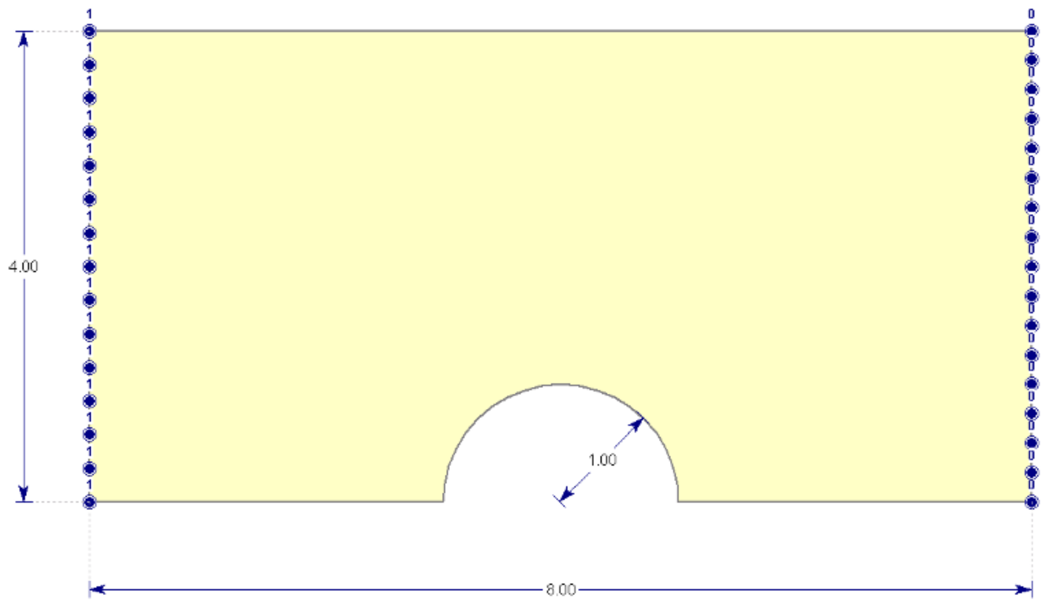


Figure 2.2: Slide model

It uses the following input parameters:

ϕ_1 = 1.0m, ϕ_2 = 0m (initial flow values at the left and right boundaries, respectively),

L = 8.0m (length of the domain),

This problem assumes fully saturated material with hydraulic conductivity of 1.0×10^{-5}.

Owing to the symmetry of the problem around the *x*-axis, only one half of the domain is discretized in the **Slide** model. The half domain is represented with 442 six-noded triangular elements.

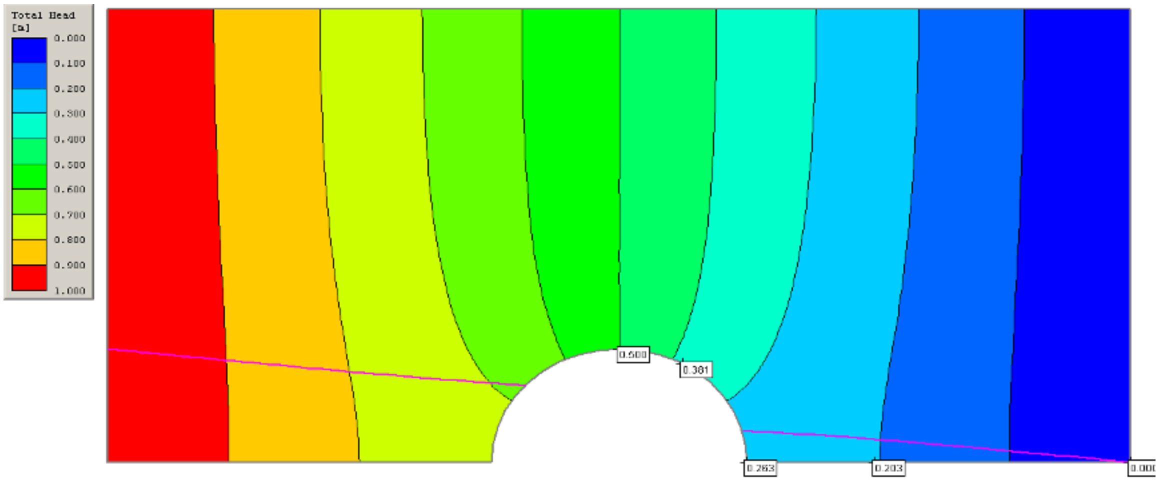


Figure 2.3: Total head contours

Figure 2.3 shows contours of total head with the values at a number of specified locations in the domain. These results from **Slide** are compared with those provided in Ref. [2]. The Slide results were within 4% of those provided in Ref [2], and also close to values calculated from equation (2.1).

The following table compares the results from **Slide** with those calculated from equation 2.1 and those presented in Ref [2]:

Table 2.1

Coordinate of Points in Problem Domain		**Flow Results from Slide**	**Flow Results from Equation (2.1)**	**Ref. [2]**
x	y			
4	1	0.5000	0.5000	0.5000
4.5	0.866	0.3810	0.3743	0.3780
5	0	0.2630	0.2500	0.2765
6	0	0.2030	0.1875	0.2132
8	0	0.0000	-0.0312	0.0000

2.3. References

1. Streeter, V.L. (1948) *Fluid Dynamics*, McGraw Hill
2. Desai, C. S., Kundu, T., (2001) *Introductory Finite Element Method,* Boca Raton, Fla. CRC Press

Note: See file *Groundwater#02.sli*

3. Confined flow under dam foundation

3.1. Problem Description

The problem considered is a simple example of confined flow. It was selected to help assess the performance of **Slide** on confined flow problems.

Figure 3.1 shows a dam that rests upon a homogeneous isotropic soil (Ref. [1]). In the example, the walls (entity 1) and base (entity 2) of the dam are assumed to be impervious. The water level is 5m, upstream of the dam, and 0m downstream. The coordinates for point A are (0,0).

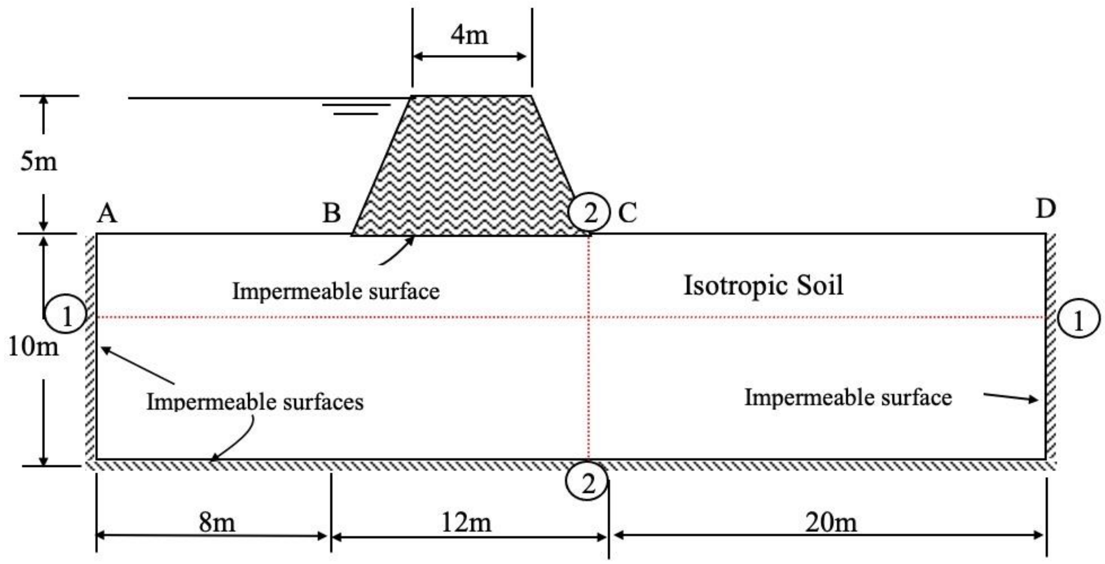


Figure 3.1: Model geometry

The flow is considered to be two-dimensional with negligible flow in the lateral direction. The flow equation for isotropic soil can be expressed as:

$$\frac{\partial^2 \phi}{\partial x^2} + \frac{\partial^2 \phi}{\partial y^2} = 0 \qquad (3.1)$$

Equation 3.1 can be solved either using a numerical procedure or a flow net. Flow net techniques are well documented in groundwater references.

The accuracy of numerical solutions for the problem is dependent on how the boundary conditions are applied. For the particular example in this document, two boundary conditions are applied:

- No flow occurs across the impermeable base, and
- The pressure heads at the ground surface upstream and downstream of the dam are solely due to water pressure

3.2. Slide Model and Results

The model created in **Slide** for this problem, with the mesh used, is shown in Figure 3.2.

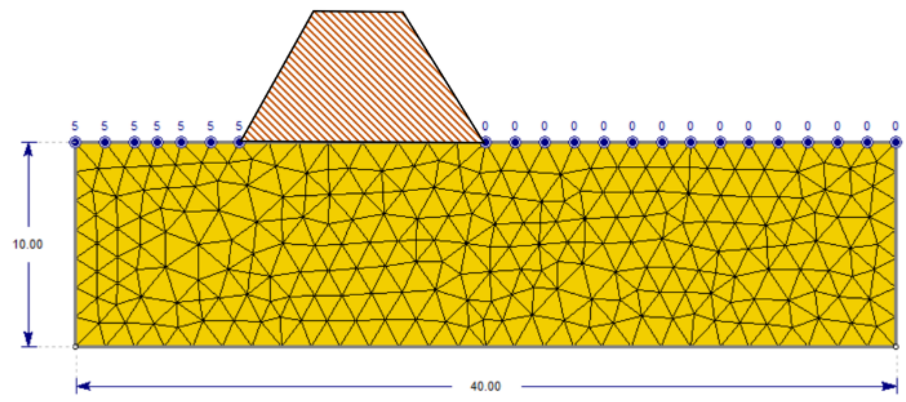


Figure 3.2: Slide model

The following boundary conditions were used for the model:

- The total head along the line segment, upstream of the dam, that lies between points A and B (see Figure 3.1), is equal to 5m
- The total head along the line segment, downstream of the dam, that lies between points C and D, is equal to 0m

The **Slide** model was discretized using 502 three-noded triangular finite elements. Figures 3.3 and 3.4 show contours of pressure head and total pressure head, respectively.

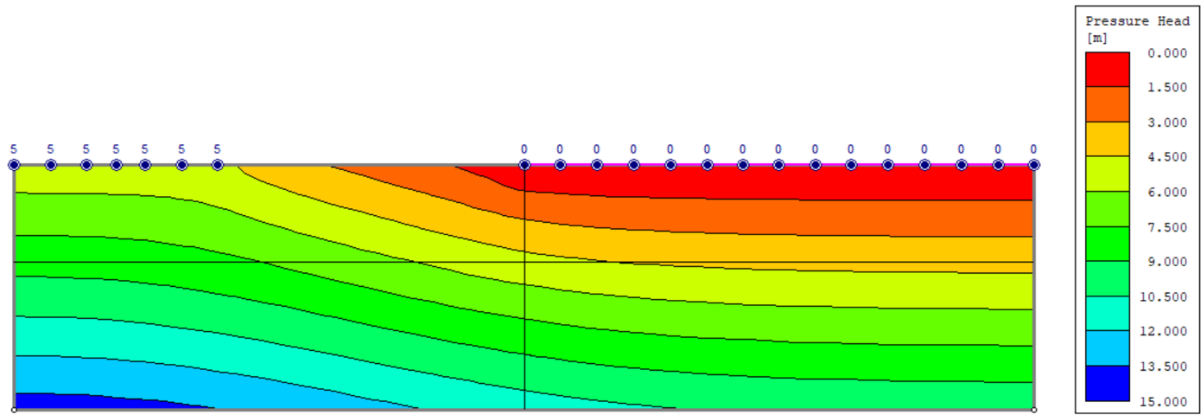


Figure 3.3: Pressure head contours

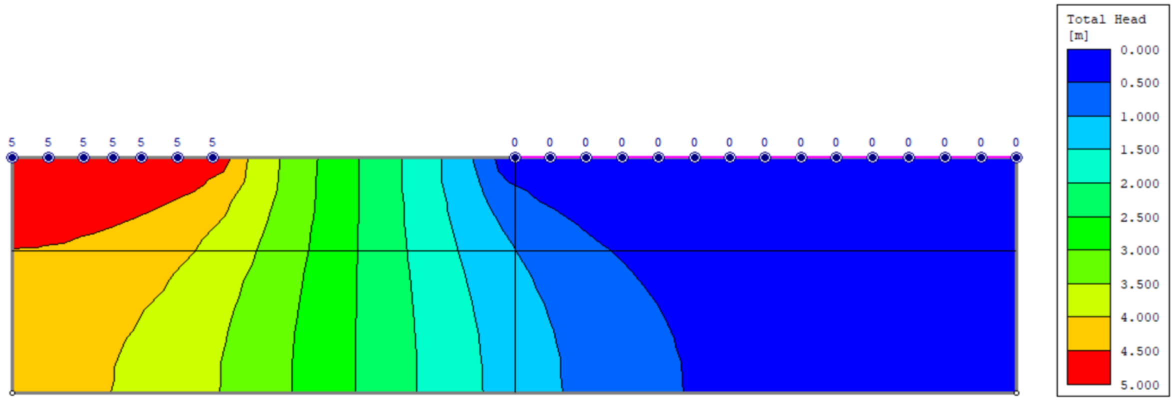


Figure 3.4: Total head pressure contours

Figures 3.5 and 3.6 compare total head pressure values from **Slide** with those obtained from Ref. [1]. These head pressures are calculated at points along line 1-1, which is located 4m below the dam base (see Figure 3.1), and along segment 2-2, a vertical cross section passing through the rightmost base of the dam.

The results from Slide agree closely with those provided in Ref. [1].

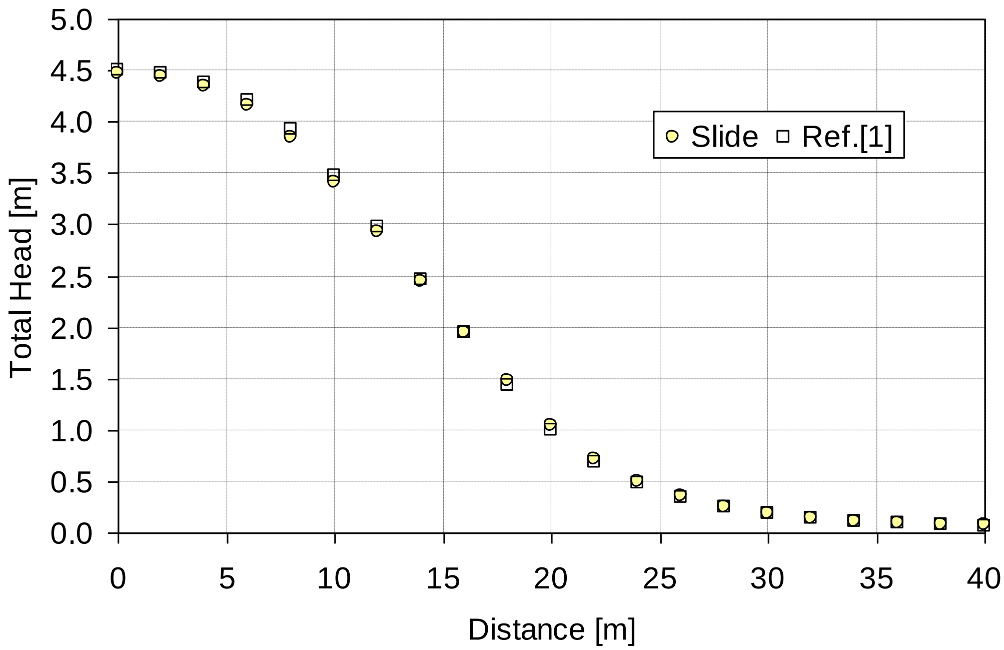


Figure 3.5: Total head variation along line 1-1

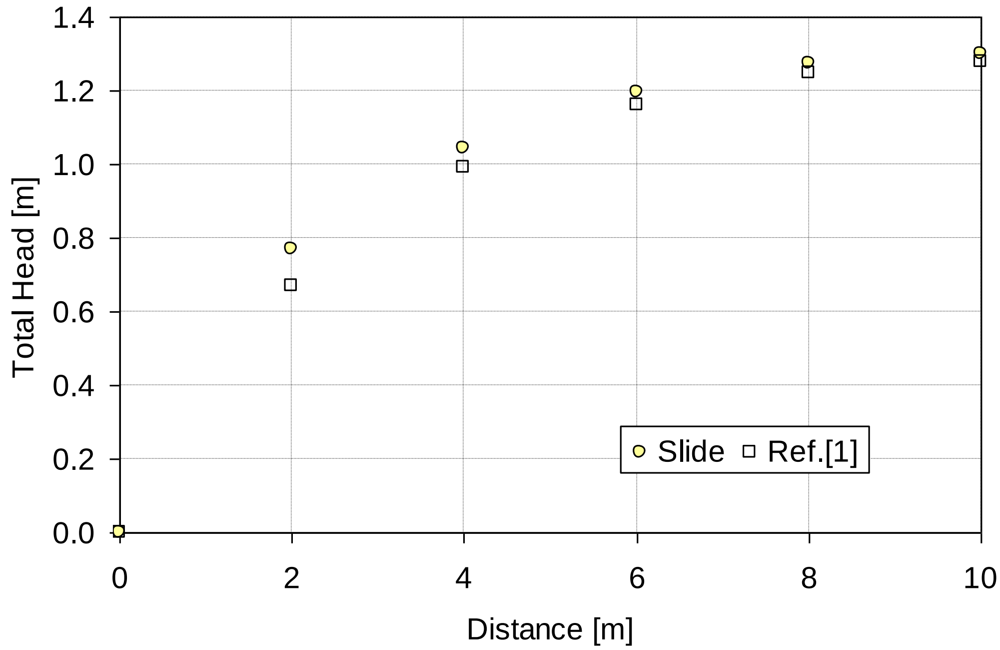


Figure 3.6: Total head variation along line 2-2

3.3. References

1. Rushton, K. R., Redshaw, S.C. (1979) *Seepage and Groundwater Flow*, John Wiley & Sons, U.K.

Note: See file *Groundwater#03.sli*

4. Steady unconfined flow through earth dam

4.1. Problem Description

This example considers the problem of seepage through an earth dam. The task of calculating the shape and length of the free surface (line of seepage) is quite complicated. Some analytical solutions, based on presenting flow nets as confocal parabolas, are available in Ref. [1] and [2].

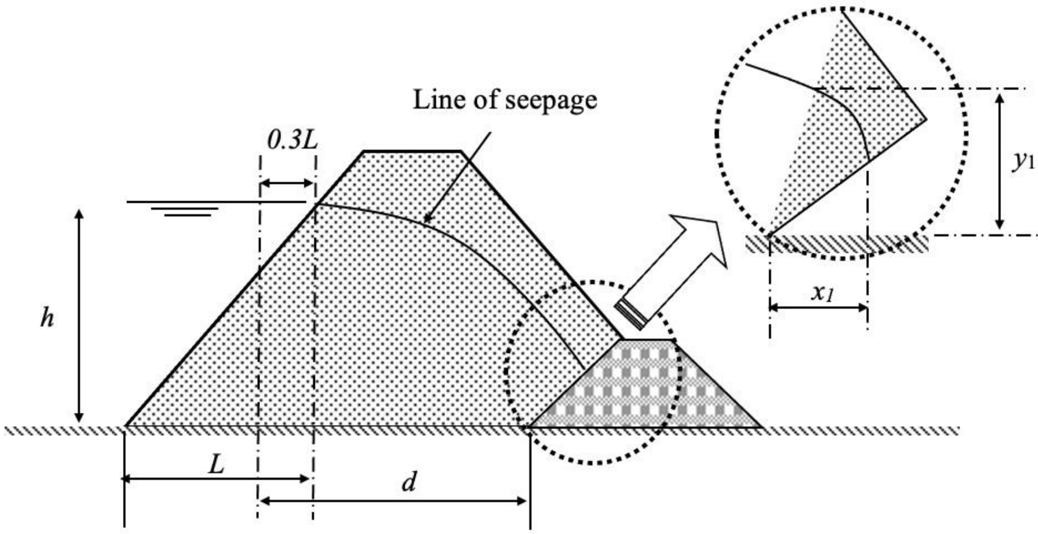


Figure 4.1: Model geometry

Figure 4.1 shows a dam that has a trapezoidal toe drain. By defining the free surface as Kozney's basic parabola (Ref. [1]), we can evaluate y_1, the vertical height of the underdrain, as:

(4.1) $$y_1 = \sqrt{d^2 + L^2} - d$$

Then the minimum horizontal length of the underdrain, x_1, equals

(4.2) $$x_1 = \frac{y_1}{2}$$

4.2. Slide Model and Results

The **Slide** model geometry and boundary conditions used in this example are shown in Figure 4.2.

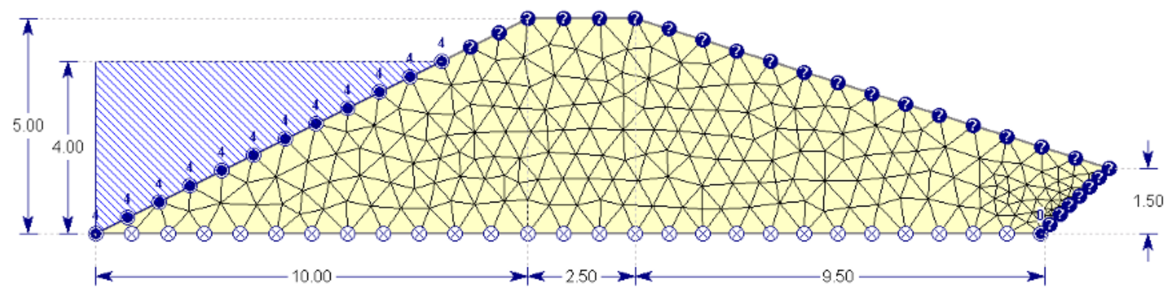


Figure 4.2: Slide model

The total head on the upstream face of the dam was taken to be 4m, and the toe drain was located at the downstream toe of the dam, i.e. total head at location (22,0) was taken to be 0. The boundary condition at the toe was assumed undefined, meaning that it initially either had flow, *Q*, or pressure head, *P*, equal to 0. A total number of three-noded triangular finite elements were used to model the problem.

Figures 4.3 and 4.4 show contours of pressure head and total head, respectively.

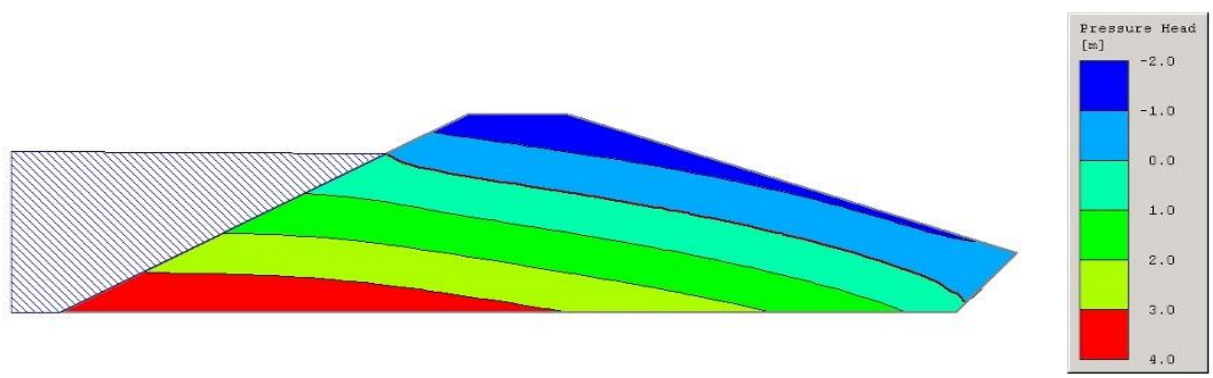


Figure 4.3: Pressure head contours

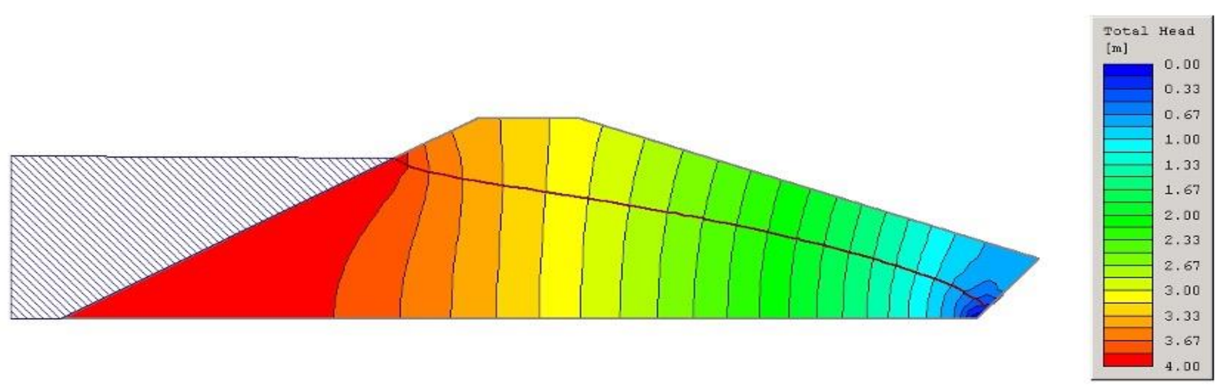


Figure 4.4: Total head contours

The minimum length and height of the underdrain were measured in **Slide** and the results are shown in Figure 4.5

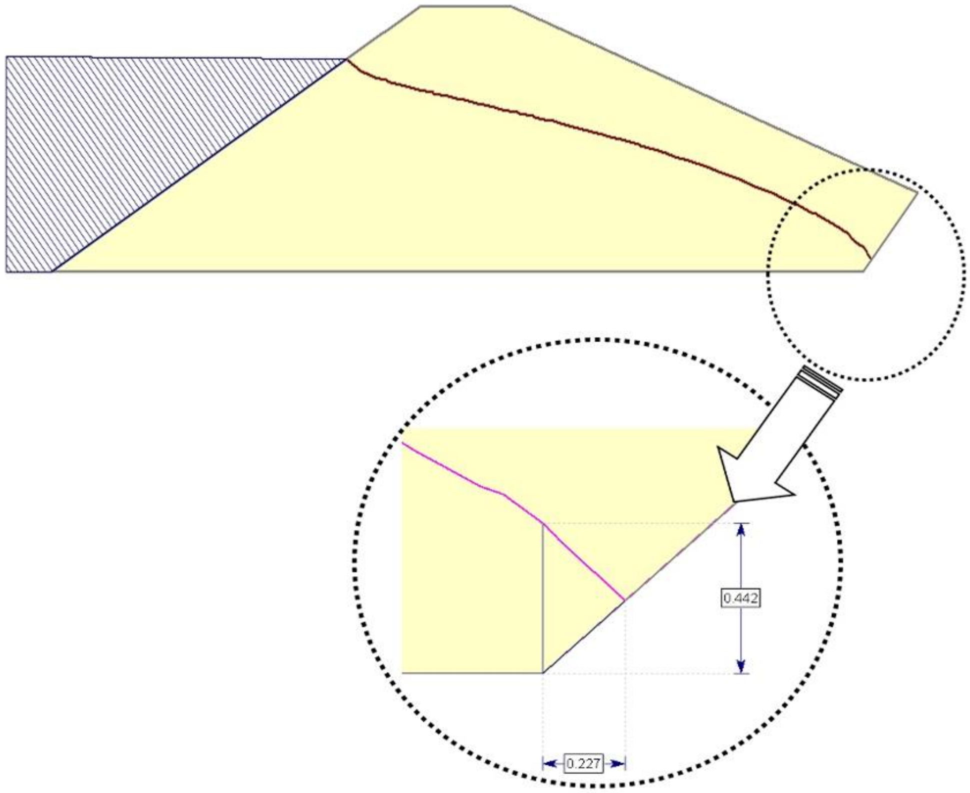


Figure 4.5: Length and minimum height of minimum underdrain

The following table compares the results from **Slide** with those calculated from equations 4.1 and 4.2

Table 4.1

Parameter	Slide	Equations (4.1 – 4.2)
x_1	0.227	0.240
y_1	0.442	0.480

As can be seen, the **Slide** results are in close agreement with the equations 4.1 and 4.2.

4.3. References

1. Haar, M. E. (1990) *Groundwater and Seepage*, 2nd edition, Dover
2. Raukivi, A.J., Callander, R.A. (1976) *Analysis of Groundwater Flow*, Edward Arnold

Note: See file *Groundwater#04.sli*

5. Unsaturated flow behind an embankment

5.1. Problem Description

The geometry of the problem considered in this section is taken from FLAC manual [1]. The example is modified slightly to handle two different materials. Two materials are considered with different coefficient of permeability. Figure 5.1 shows the geometry of the proposed model.

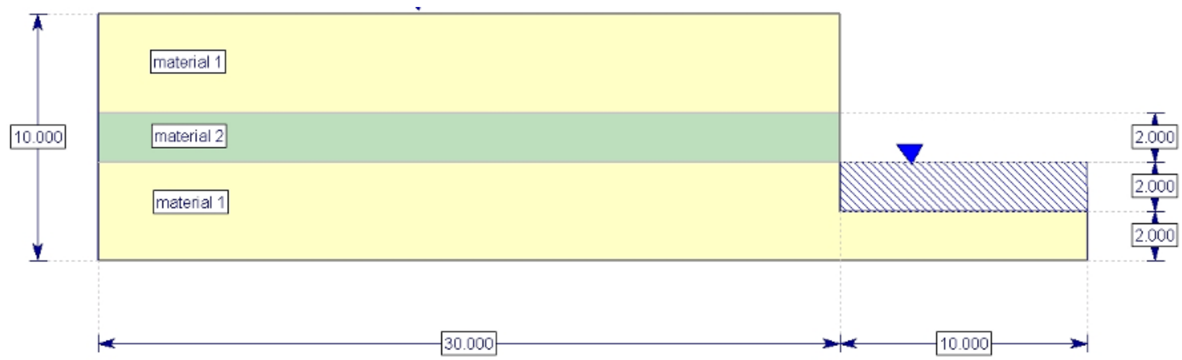


Figure 5.1: Model geometry

5.2. Slide Model and Results

The saturated hydraulic conductivity of material 1 and material 2 is $1x10^{-10}$ m/sec and $1x10^{-13}$ respectively. **Slide** model geometry is presented in Figure 5.1. The problem is discretized using 6-noded triangular finite elements. The total number of elements used was 746 elements. The boundary conditions are applied as total head of 10m at the left side and 4m at the right side of the geometry. Zero flow is assumed at the top and at the bottom of the geometry.

Figures 5.2-5.3 show contours of pressure head from **Slide** and FLAC respectively.

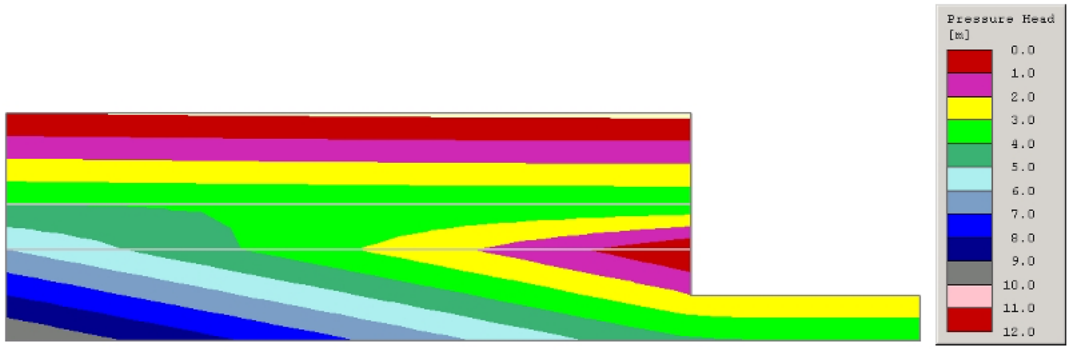


Figure 5.2: Pressure head contours from **Slide**

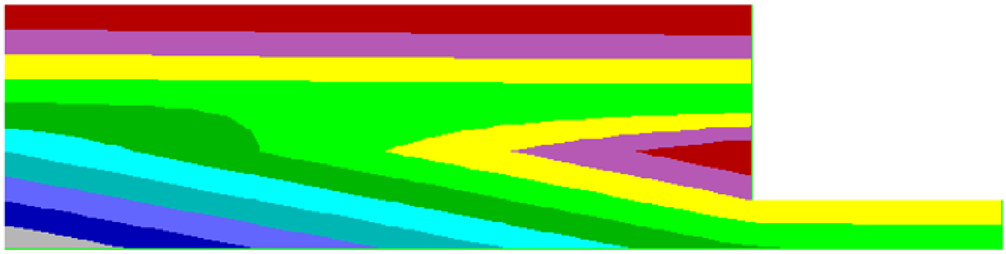


Figure 5.3: Pressure head contours from FLAC

Figures 5.4 and 5.5 show the flow lines obtained from **Slide** and FLAC.

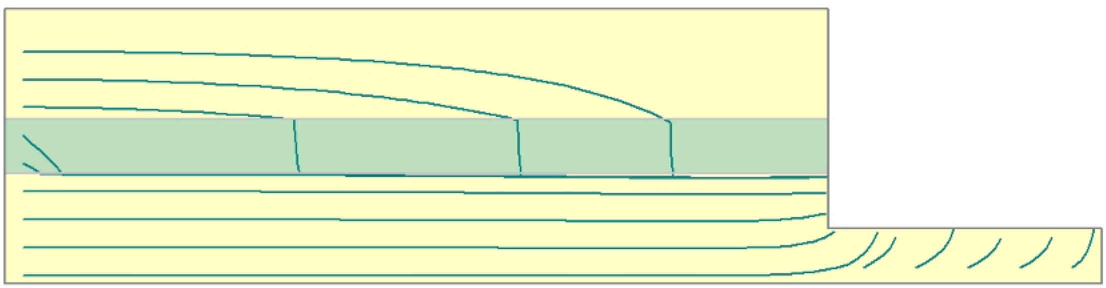


Figure 5.4: Flow lines from **Slide**

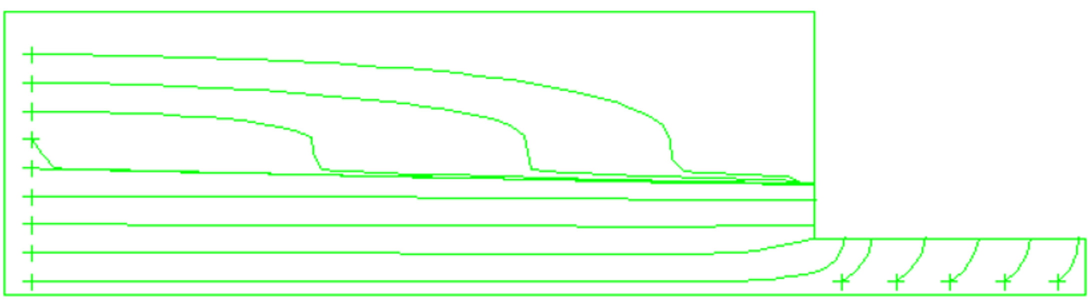


Figure 5.5: Flow lines from FLAC

The results from **Slide** and FLAC compared very well with the predicted performance.

5.3. References

1. FLAC manual, Itasca Consulting Group Inc., 1995

Note: See file *Groundwater#05.sli*

6. Steady-state seepage analysis through saturated-unsaturated soils

6.1. Problem Description

This example considers the problem of seepage through an earth dam. The geometry of the problem considered in this section, which is shown in Figure 6.1, is taken from *Soil Mechanics for Unsaturated Soils* by Fredlund & Rahardjo [1].

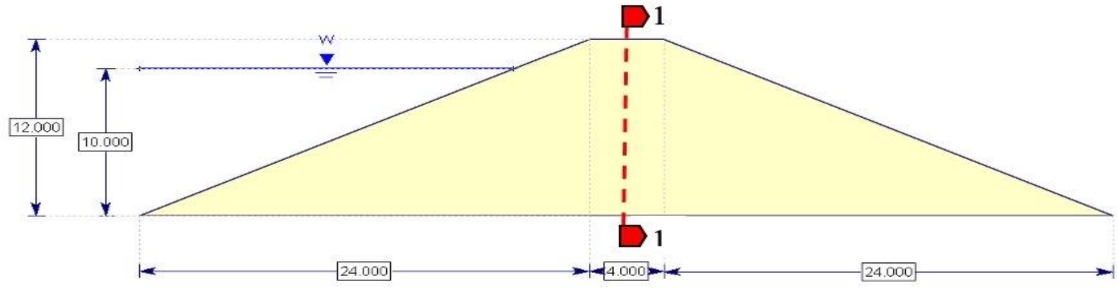


Figure 6.1: Model geometry

6.2. Slide Model and Results

The problem is discretized using 3-noded triangular finite elements. The total number of elements used was 336 elements. The mesh used for this example was created using mapped mesh option to replicate similar mesh of Ref. [1]. Five different cases are presented in this example as follows:

1. <u>Isotropic earth dam with a horizontal drain</u>

The first case considers an isotropic earth dam with 12m horizontal drain. The permeability function used in the analysis is shown in Figure 6.2.

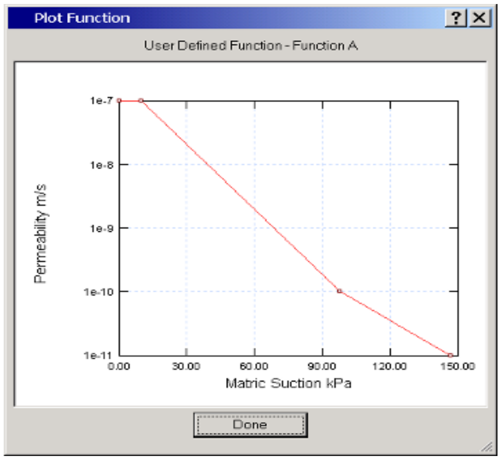


Figure 6.2: Permeability function for the isotropic earth dam

Figure 6.3 presents the flow vectors and the location of the phreatic line from **Slide** ground water model.

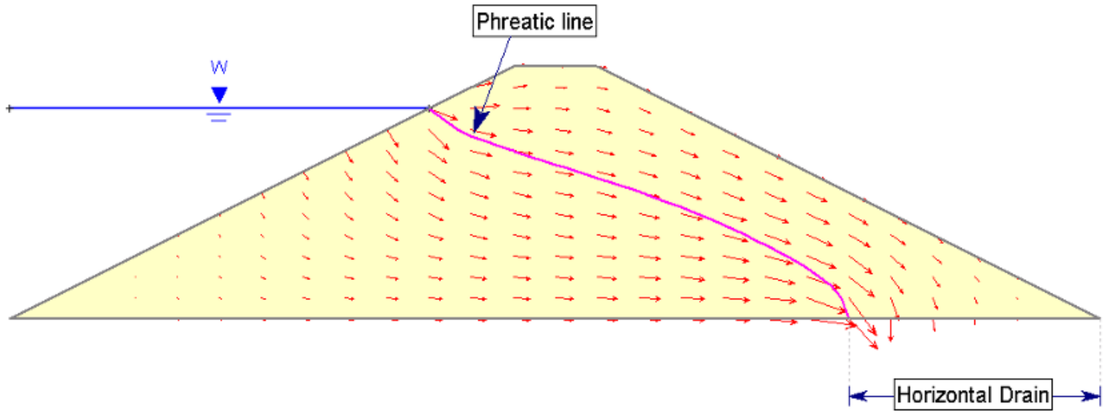


Figure 6.3: Flow vectors

The contours of pressure and total head calculated using finite element method are presented in figures 6.3-6.4 respectively.

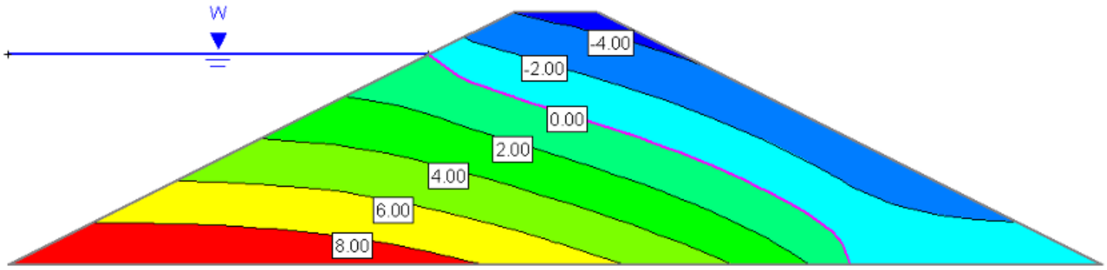


Figure 6.4: Pressure head contours

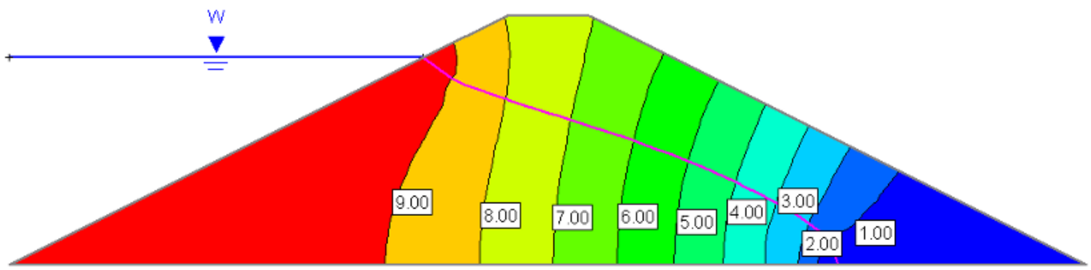


Figure 6.5: Total head contours

Figure 6.6 shows a comparison between slide results and results from Ref. [1] for pressure head distribution along line 1-1.

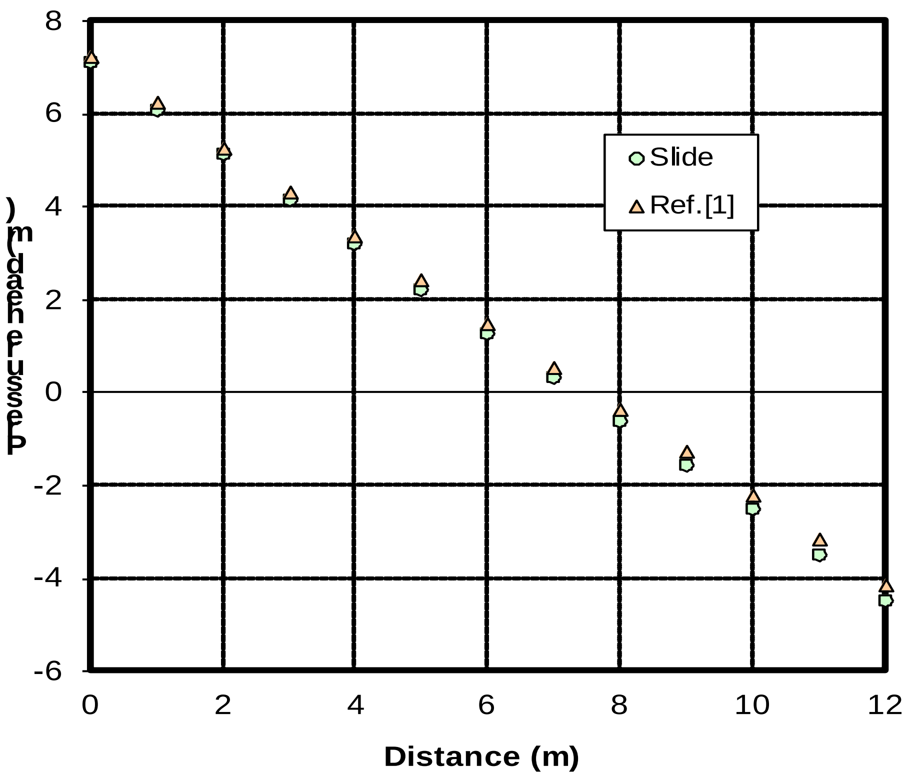


Figure 6.6: Pressure head distribution along line 1-1

Note: See file *Groundwater#06_1.sli*

2. <u>Anisotropic earth dam with a horizontal drain</u>

The dam is modeled with anisotropic soil with water coefficient permeability in the horizontal direction is assumed to be nine times larger than in the vertical direction. Figures 6.7 – 6.8 show the contours for pressure head and total head throughout the dam.

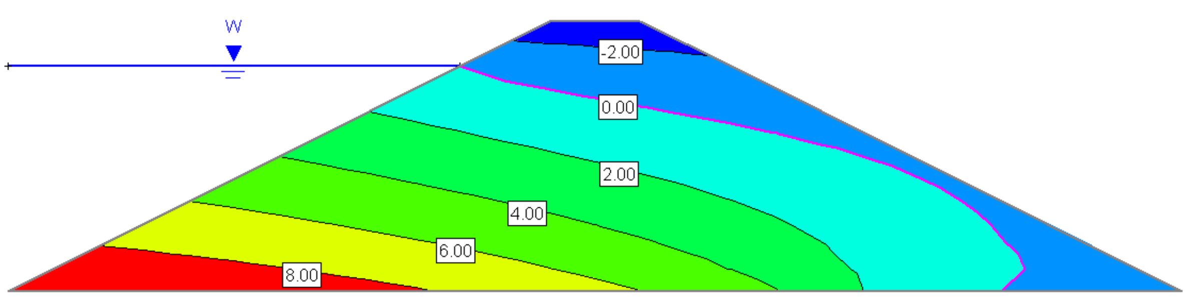


Figure 6.7: Pressure head contours

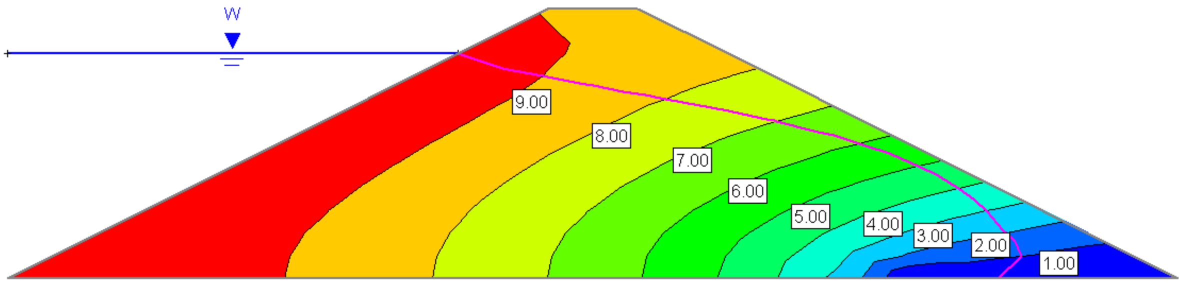


Figure 6.8: Total head contours

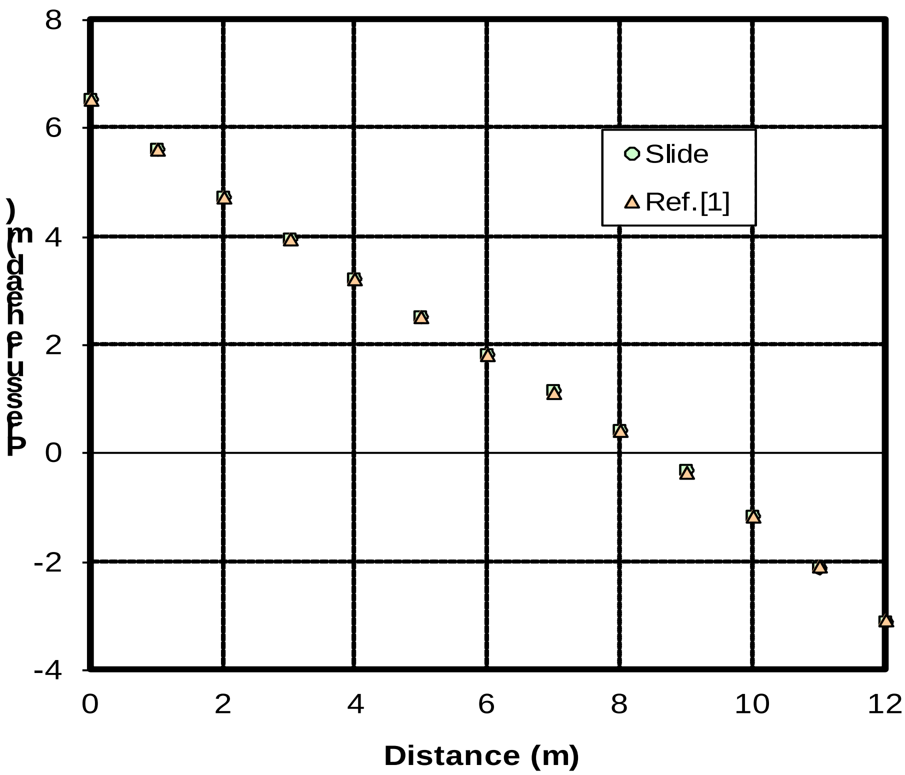


Figure 6.9: Pressure head distribution along line 1-1

Figure 6.9 shows a comparison between slide results and results from Ref. [1] for pressure head distribution along line 1-1.

Note: See file *Groundwater#06_2.sli*

3. Isotropic earth dam with a core and horizontal drain

The third case considers an isotropic dam having core with lower coefficient of permeability. Figure 6.10 shows the permeability function used for the core material. The results show that the hydraulic head change take place in the zone around the core. The flow vectors show that the water flows upward into the unsaturated zone and go around the core zone as shown in Figure 6.11. Pressure head and total head contours are presented in Figures 6.12-6.13 respectively.

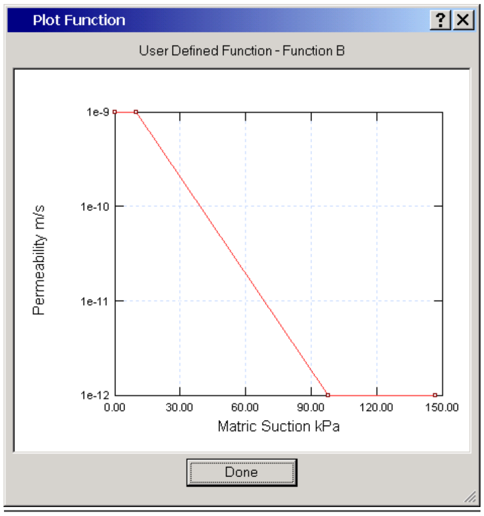


Figure 6.10: Permeability function for the core of the dam

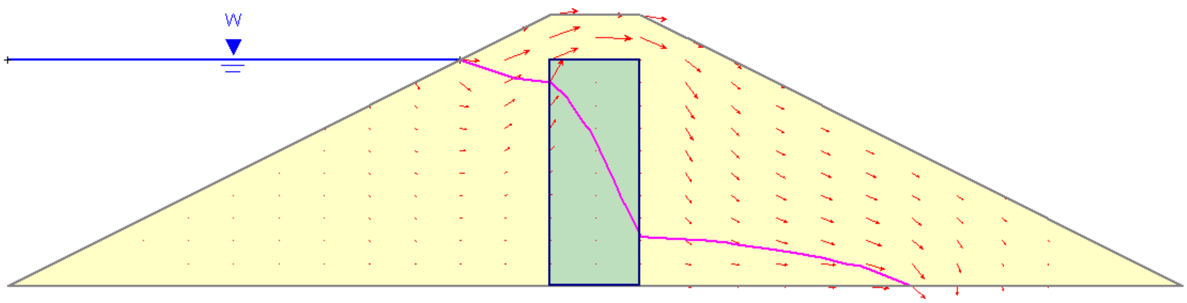


Figure 6.11: Flow vectors

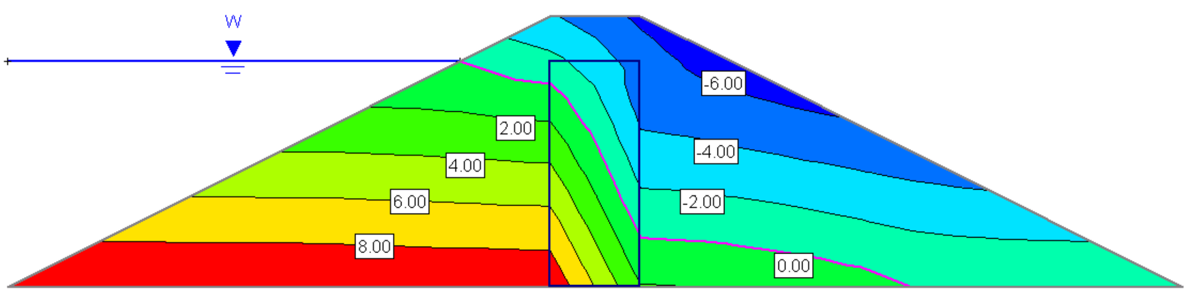


Figure 6.12: Pressure head contours

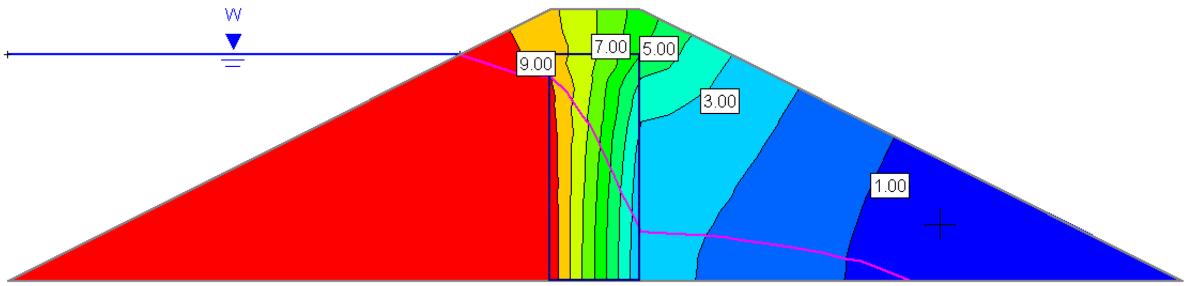


Figure 6.13: Total head contours

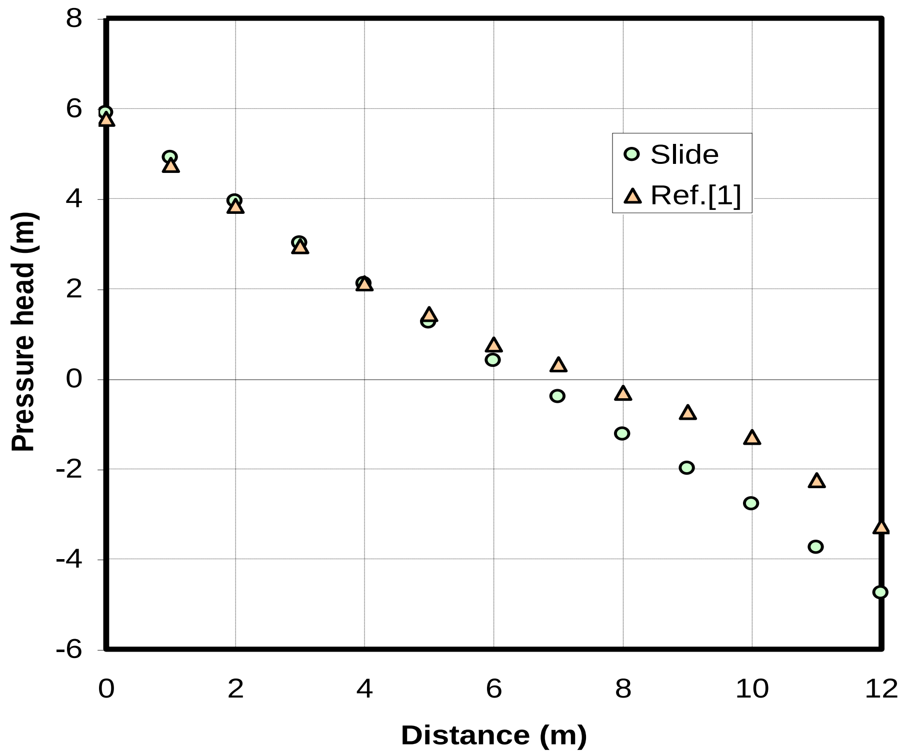


Figure 6.14: Pressure head distribution along line 1-1

Figure 6.14 shows a comparison between slide results and results from Ref. [1] for pressure head distribution along line 1-1.

Note: See file *Groundwater#6_4.sli*

4. Isotropic earth dam under steady-state infiltration

The fourth case considers the effect of infiltration on the dam shown in Figure 6.15. Infiltration is simulated by applying a flux boundary of 1x10^{-8}m/s along the boundary of the dam. Pressure head and total head contours are presented in Figures 6.16-6.17 respectively.

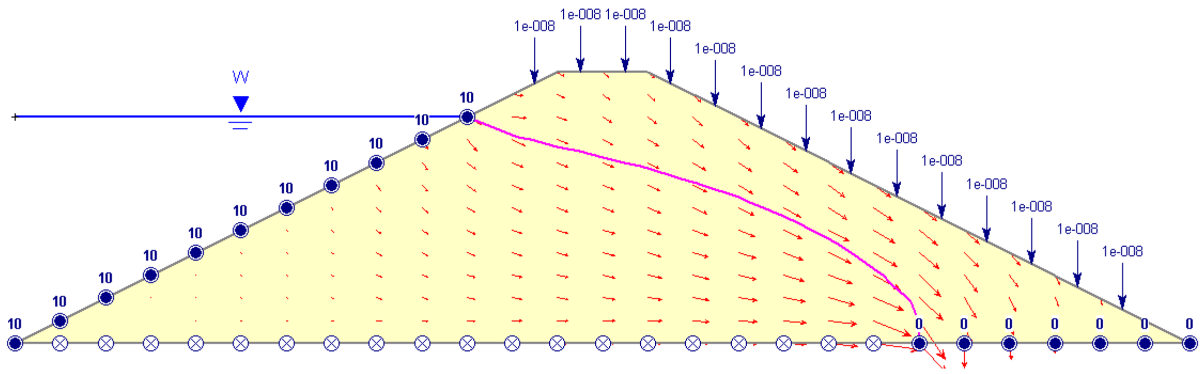


Figure 6.15: Seepage through dam under infiltration

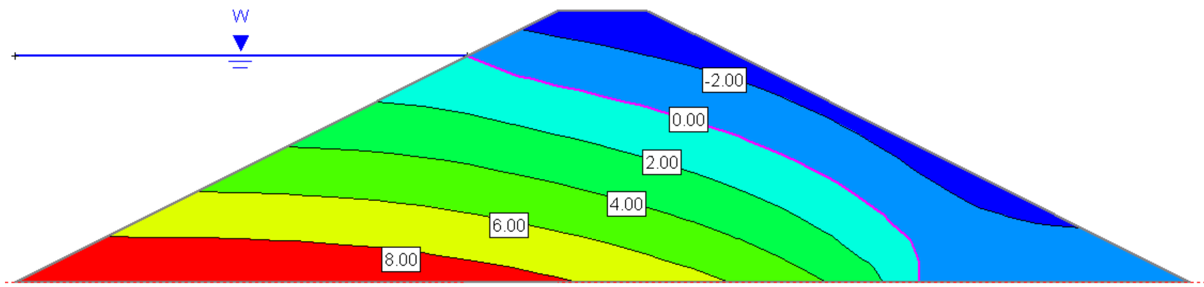


Figure 6.16: Pressure head contours

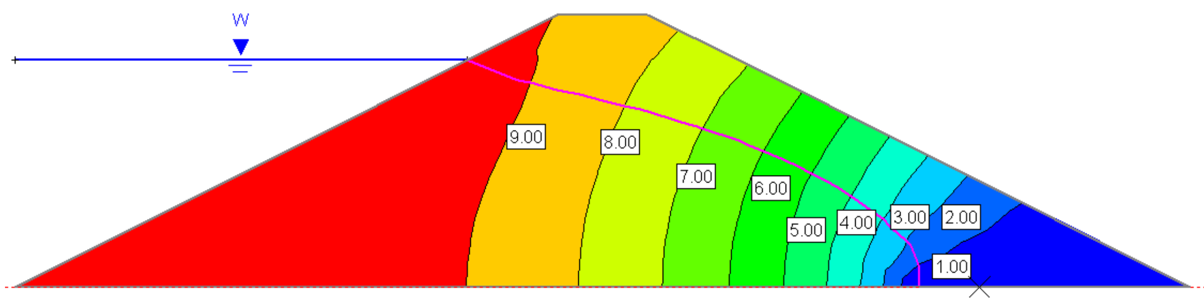


Figure 6.17: Total head contours

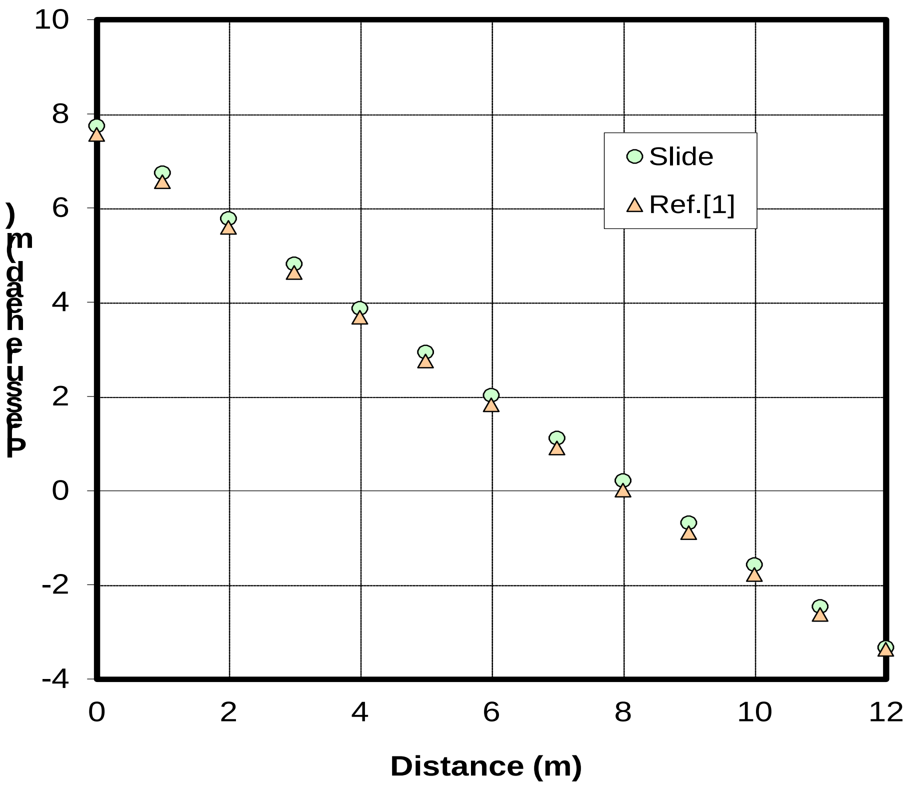


Figure 6.18: Pressure head distribution along line 1-1

Figure 6.18 shows a comparison between slide results and results from Ref. [1] for pressure head distribution along line 1-1.

Note: See file *Groundwater#6_5.sli*

5. <u>Isotropic earth dam with seepage face</u>

The fifth case demonstrates the use of unknown boundary condition which is usually used for the case of developing seepage faces. The boundary conditions and the phreatic surface are presented in Figure 6.19. Pressure head and total head contours are presented in Figures 6.20-6.21 respectively.

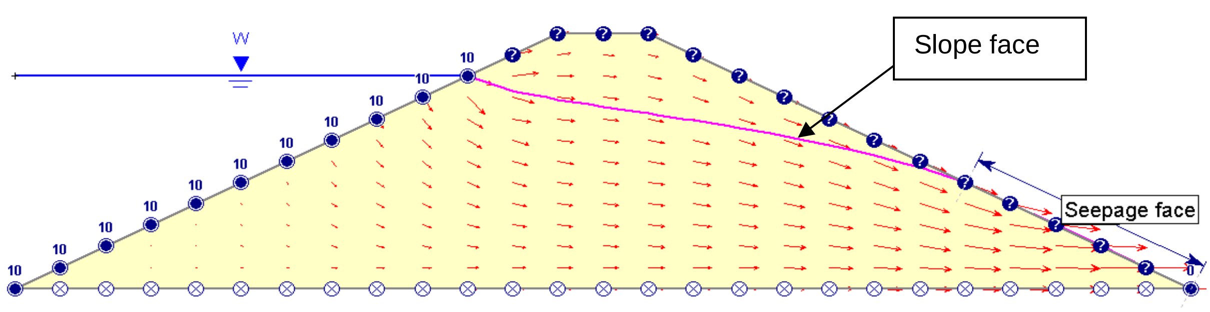


Figure 6.19: Seepage through dam under infiltration

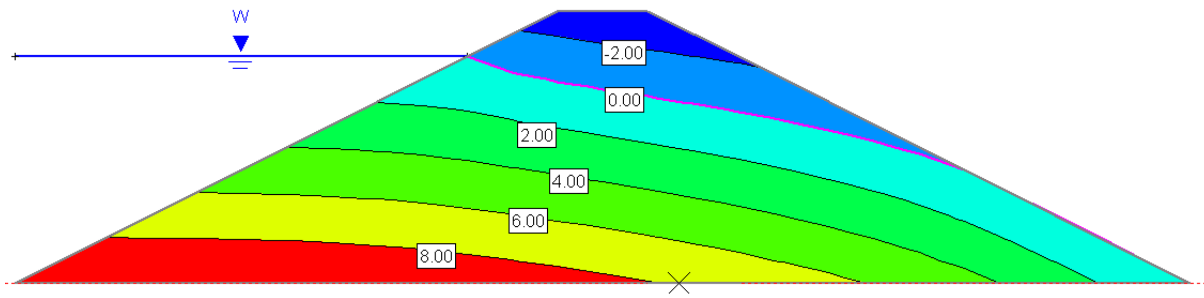


Figure 6.20: Pressure head contours

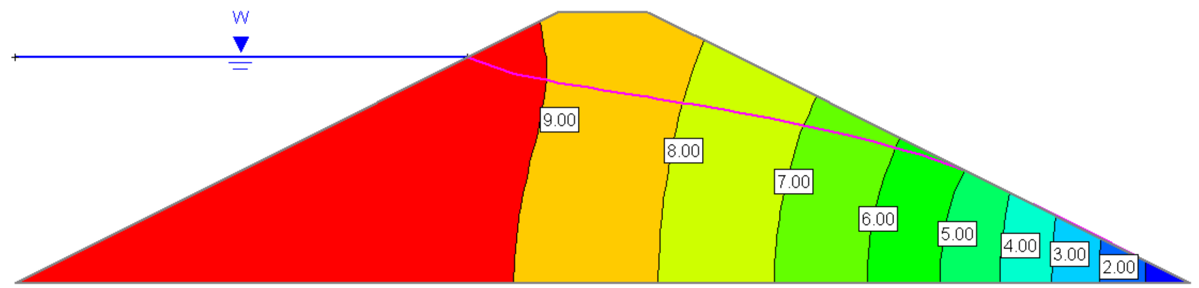


Figure 6.21: Total head contours

Figure 6.22 shows a comparison between slide results and results from Ref. [1] for pressure head distribution along the slope face.

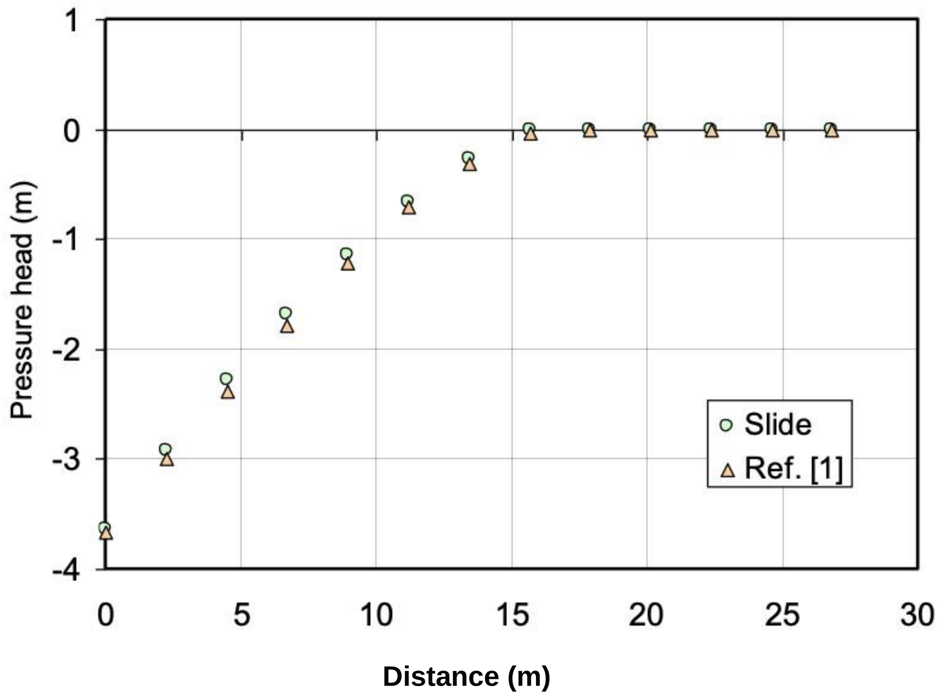


Figure 6.22 Pressure head distribution along slope face

Figure 6.23 shows a comparison between slide results and results from Ref. [1] for pressure head distribution along line 1-1.

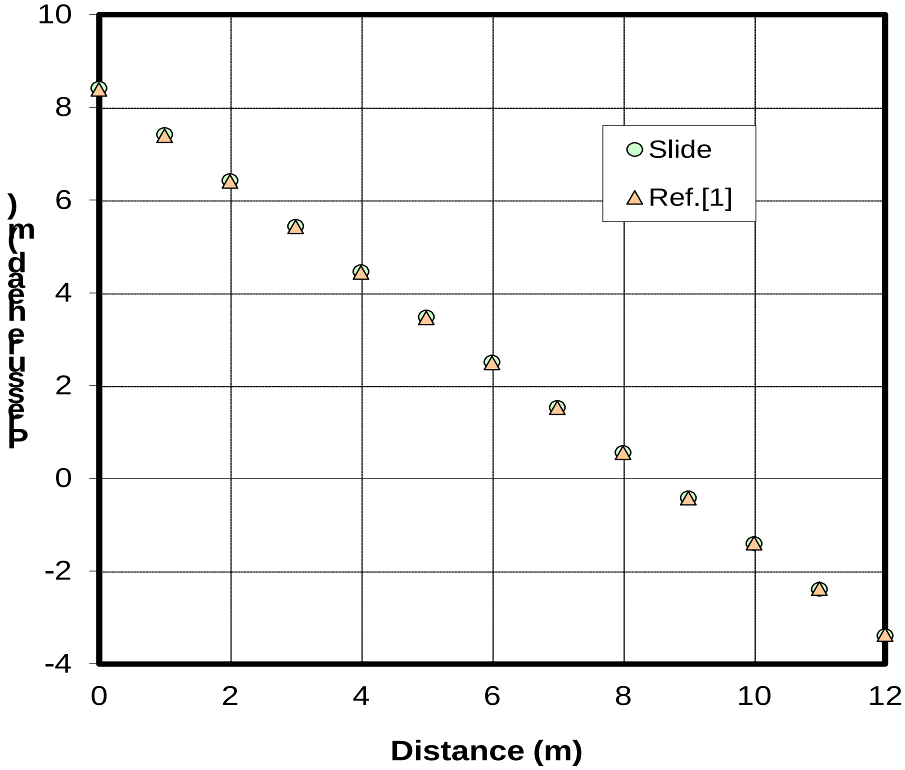


Figure 6.23: Pressure head distribution along line 1-1

Note: See file *Groundwater#06_5.sli*

6.3. References

1. Fredlund, D.G. and H. Rahardjo (1993) *Soil Mechanics for Unsaturated Soils*, John Wiley

7. Seepage within layered slope

7.1. Problem Description

This example considers the problem of seepage through a layered slope. Rulan and Freeze [1] studied this problem using a sandbox model. The material of the slope consisted of medium and fine sand. The fine sand has lower permeability than the medium sand. The geometry of the problem is shown in Figure 7.1 and the two permeability functions used to model the soil is presented in Figure 7.2. These permeability functions are similar to those presented by Fredlund and Rahardjo [2].

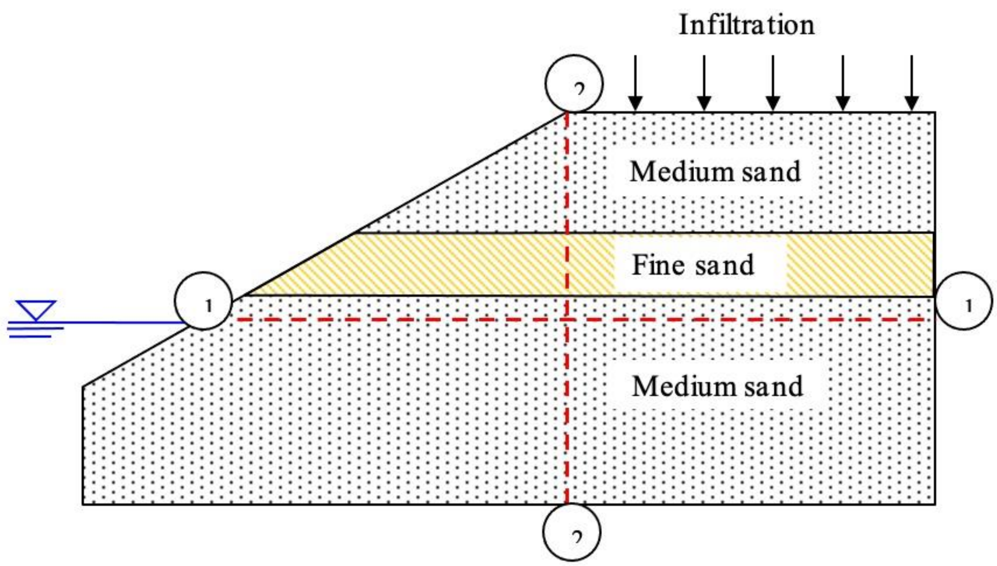


Figure 7.1: Model description

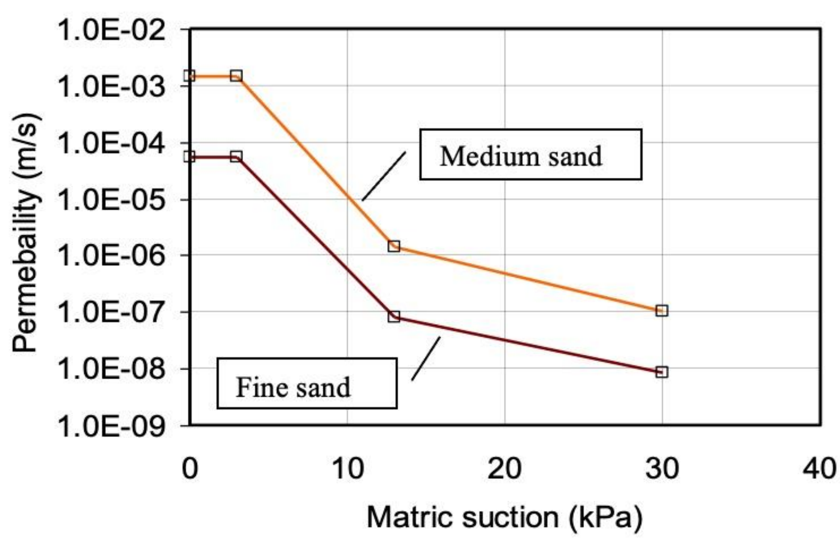


Figure 7.2: Permeability function for the fine and medium sand

7.2. Slide Model and Results

The **Slide** model geometry used in this example is shown in Figure 7.3.

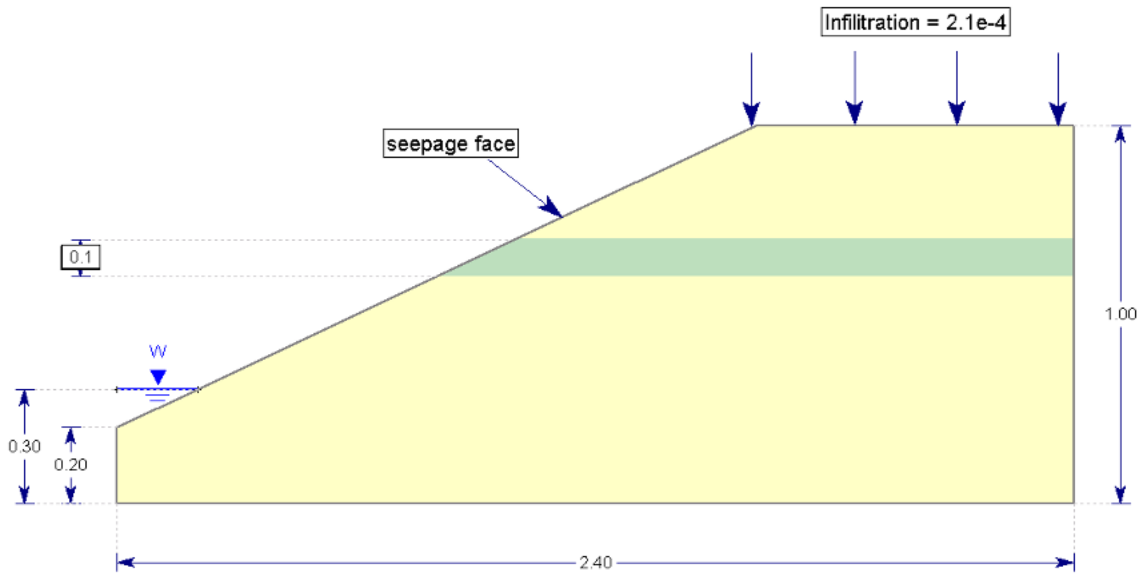


Figure 7.3: **Slide** model

A constant infiltration rate of 2.1×10^{-4} is applied to the top of the side of the slope. The water table is located at 0.3 m from the toe of the slope. The boundary condition at the slope face was assumed undefined, meaning that it initially either had flow, *Q*, or pressure head, *P*, equal to 0. Figure 7.4 shows the location of the calculated water table location and the direction of the flow vectors.

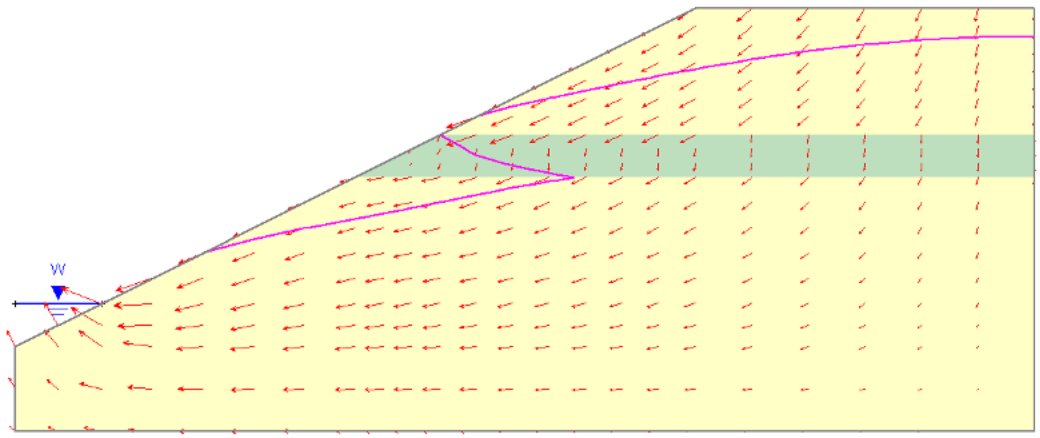


Figure 7.4: Flow vectors

Figures 7.5 and 7.6 show contours of pressure head and total head pressure from **Slide**, respectively.

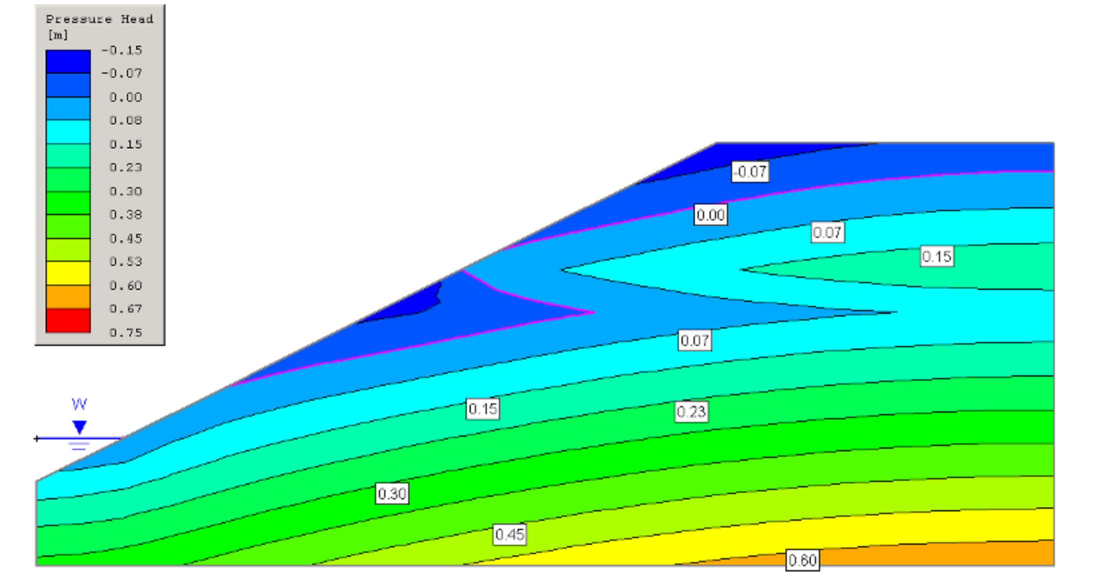


Figure 7.5: Pressure head contours

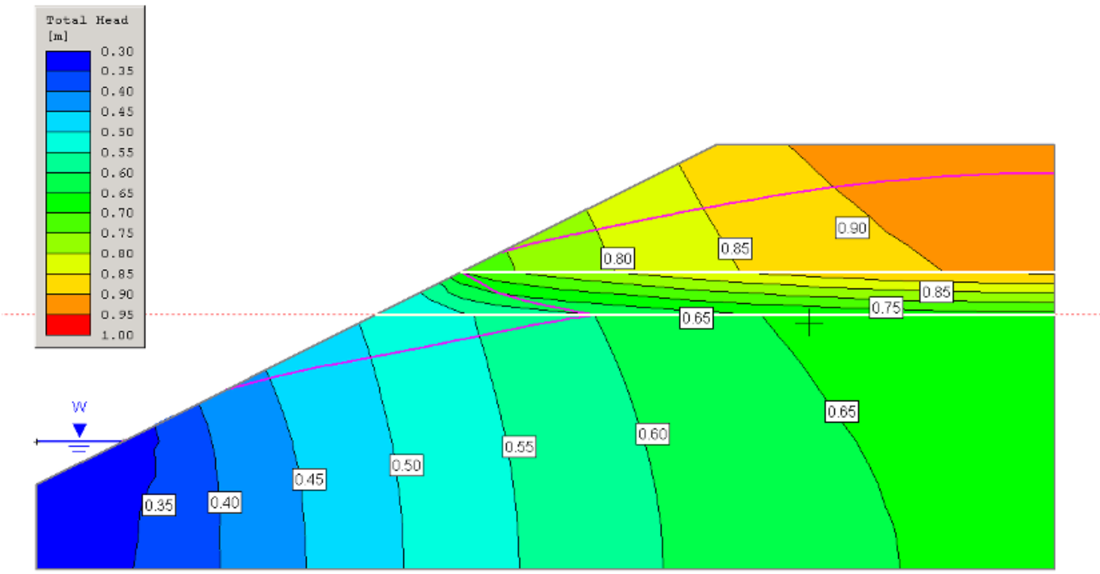


Figure 7.6: Total head contours

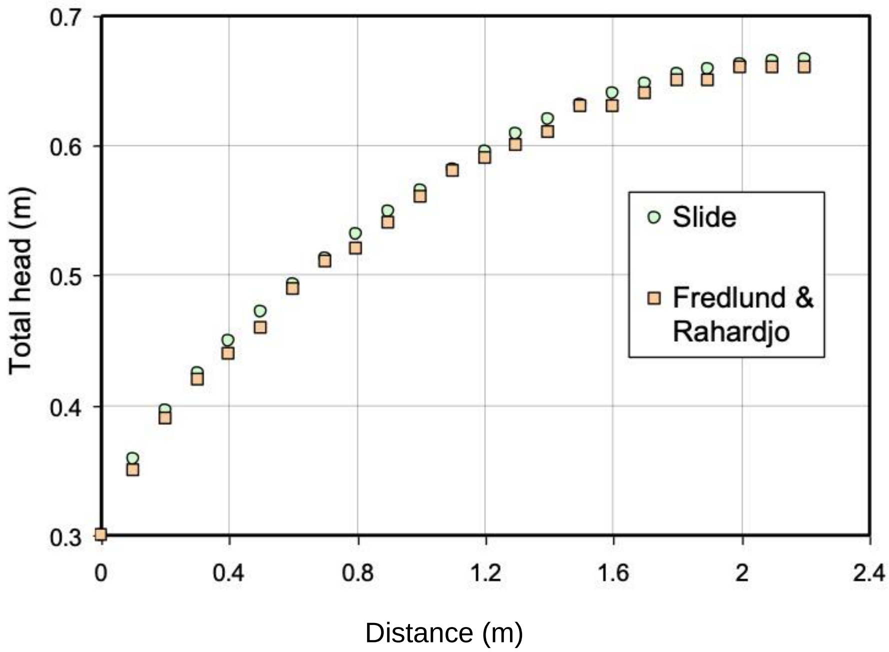


Figure 7.7: Total head variation along line 1-1

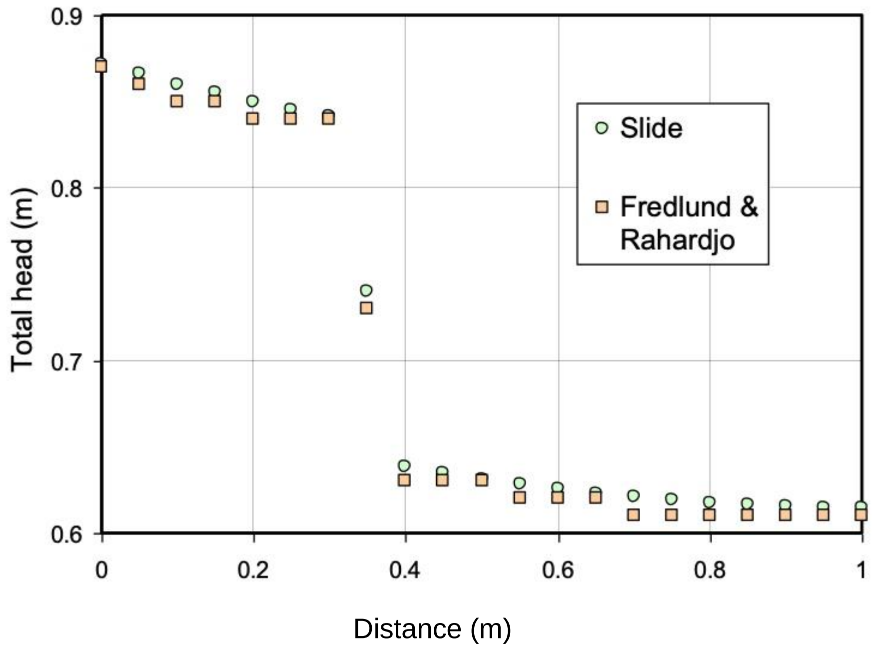


Figure 7.8: Total head variation along line 2-2

7.3. References

1. Fredlund, D.G. and H. Rahardjo (1993) *Soil Mechanics for Unsaturated Soils*, John Wiley

Note: See file *Groundwater#07.sli*

8. Flow through ditch-drained soils

8.1. Problem Description

In problems related to ditch-drained aquifers, numerical solutions are often used to predict the level of the water table and the distribution of soil-water pressure. The problem considered in this section involves the infiltration of water downward through two soil layers.

Half-drain spacing with a length of 1m and the depth of the soil to the impermeable level is 0.5m. The ditch is assumed to be water free. Figure 8.1 shows the problem description.

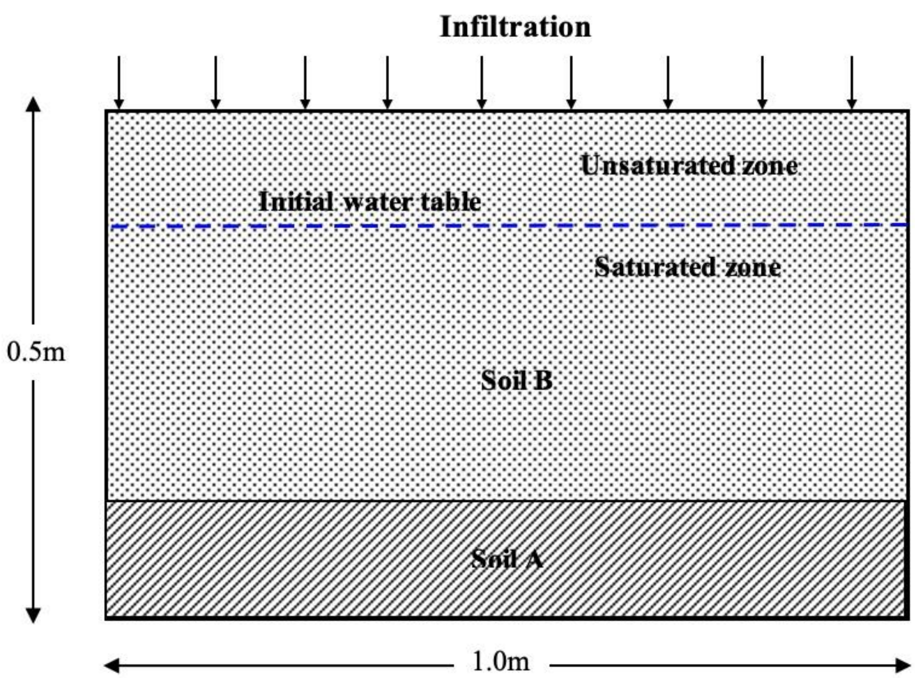


Figure 8.1: Model geometry

The soil properties of the layered system are given in the Table 8.1 simulating a coarse and a fine soil. The lower layer has a thickness of 0.1m. The rate of incident rainfall (infiltration) is taken to be equal to 4.4e-6 m/sec.

Table 8.1: Material parameters

Soil A	*Relative Conductivity*	1.11e-3 (m/s)
	Gardner's parameters	*a* = 1000, *n* = 4.5
Soil B	*Relative Conductivity*	1.11e-4 (m/s)
	Gardner's parameters	*a* = 2777.7, *n* = 4.2

8.2. Slide Model and Results

The **Slide** model for the problem is shown in Figure 8.2.

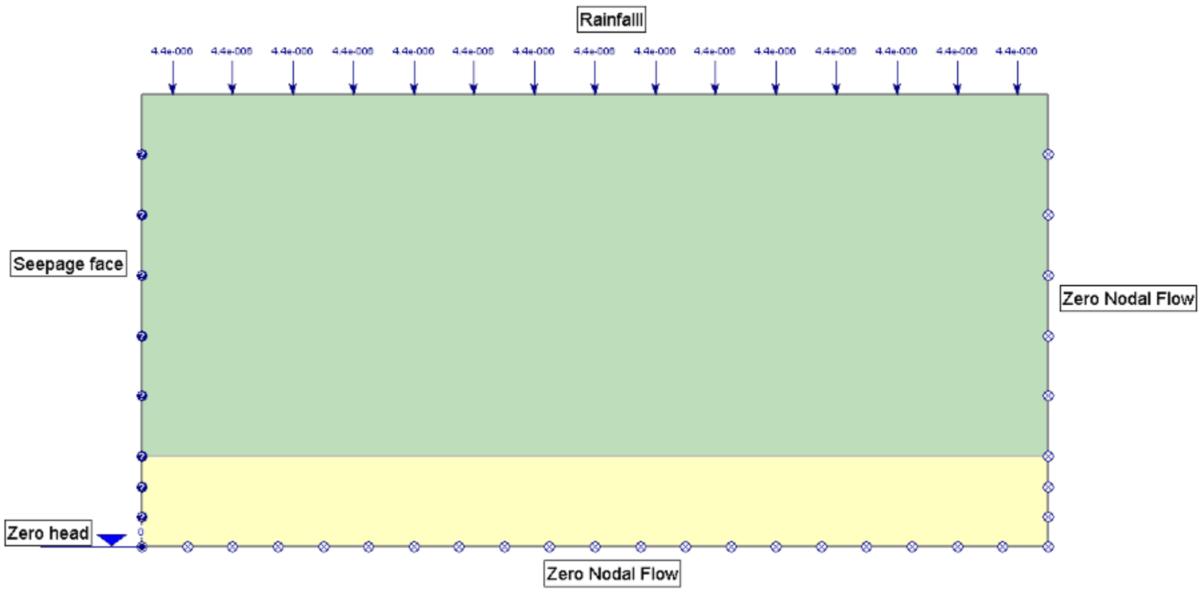


Figure 8.2: Slide model

The problem is modelled using three-noded triangular finite elements. The total number of elements used was 459 elements.

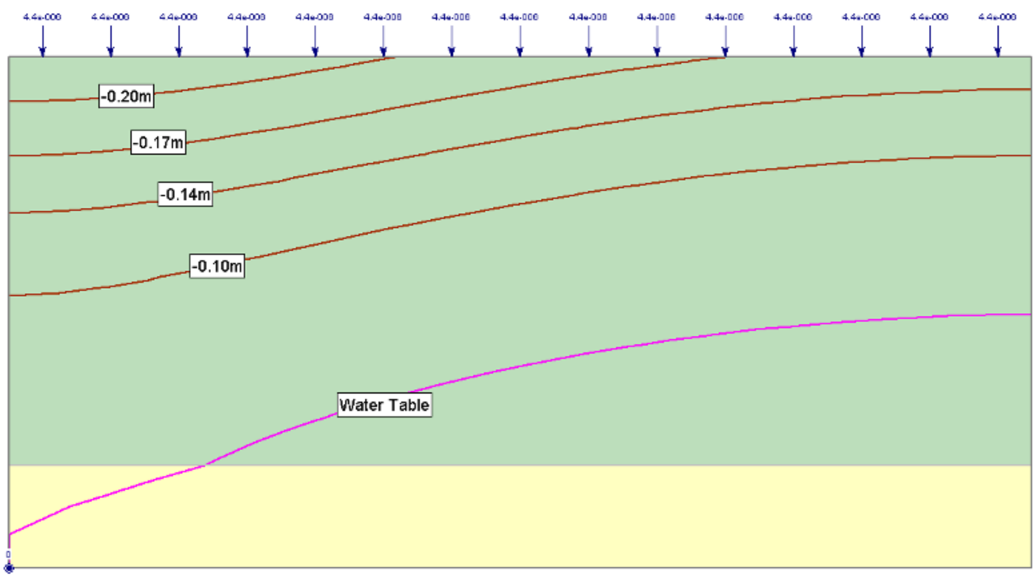


Figure 8.3: The computed unsaturated soil-water regime above the water table

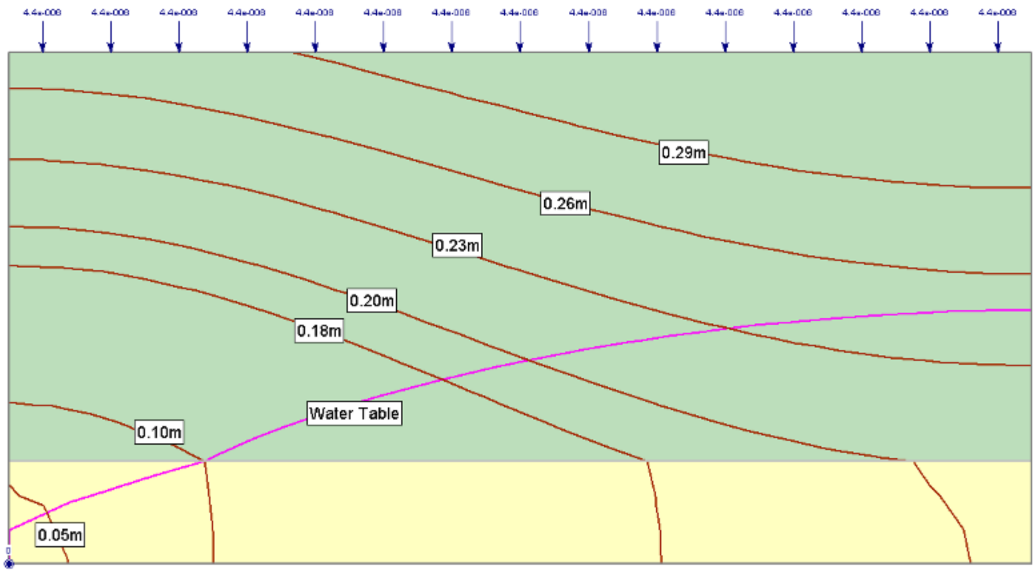


Figure 8.4: The computed total head contours for the drainage situation

Figure 8.3 gives the distribution of the soil-water pressure head for the unsaturated regime above the water table. The computer total head contours are presented in Figure 8.4. The **Slide** results are in close agreement with the solution provided by Gureghian [1].

8.3. References

1. Gureghian A., (1981) “A two-dimensional finite element solution scheme for the saturated-unsaturated flow with application to flow through ditch drained soils:” J. Hydrology. (50), 333-353.

Note: See file *Groundwater#08.sli*

9. Seepage through dam

9.1. Problem Description

Seepage flow rate through earth dams is examined in this section. The geometry and material properties for two earth dams are taken from the textbook, *Physical and geotechnical properties of soils* by Bowels [1]. Bowles calculated the leakage flow rate through these dams using flow net techniques which neglects the unsaturated flow. Chapuis et. al. [2] solved the same examples using SEEP/W, a finite element software package. In this section, **Slide** results are compared with Bowles [1] and SEEP/W [2] results.

9.2. Slide Model and Results

1. Homogeneous dam

The seepage rate of homogeneous dam is verified in this section (this example is presented in Bowles, pp.295). Figure 9.1 shows detailed geometry of the dam. The total head of 18.5 is applied on the left side of the dam and the seepage flow rate is calculated on the right side of the dam. A customized permeability function is used to model the material conductivity for the saturated-unsaturated zone (Figure 9.2). This hydraulic conductivity function is similar to the one presented in Chapius et al. [2]. The dam is discretized using 4-noded quadrilateral finite elements. A total of 391 finite elements are used for the mesh.

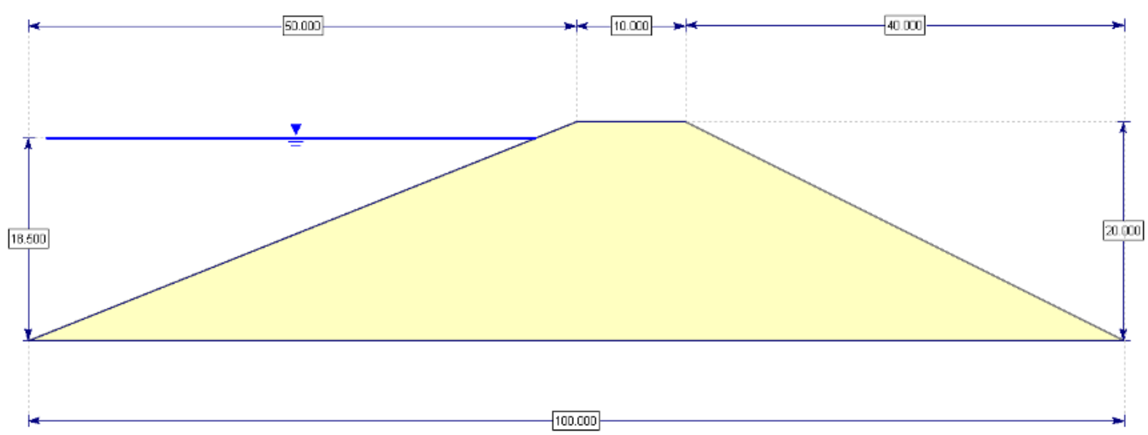


Figure 9.1: Homogenous dam geometry details

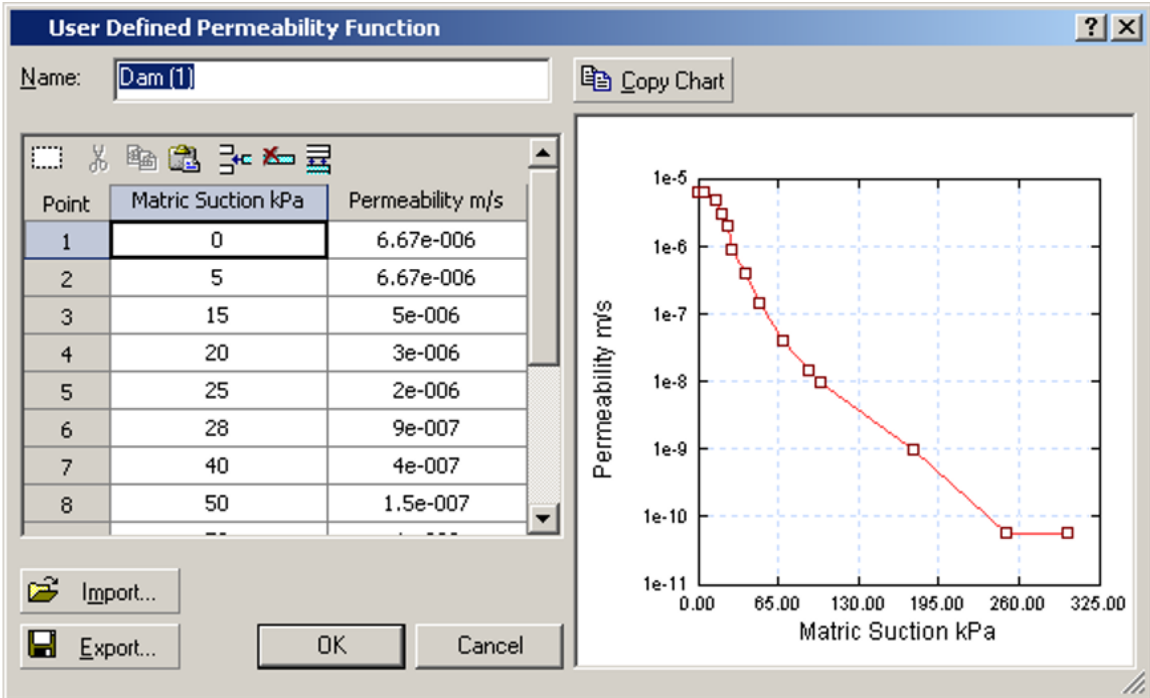


Figure 9.2: Permeability function for the isotropic earth dam

Slide gave a flow rate of Q = 1.378×10^{-3} m^3/(min.m) which compared well with the flow rate estimated by Bowels [1], which used two approximate methods that neglect the unsaturated flow. Bowels' two methods gave Q = 1.10×10^{-3} and 1.28×10^{-3} m^3/(min.m). Chapuis et al. [2] solved the same example using finite element software SEEP/W. The flow rate calculated using SEEP/W was 1.41×10^{-3} m^3/(min.m) for a mesh of 295 elements and a flow rate of 1.37×10^{-3} m^3/(min.m) for a mesh of 1145 elements.

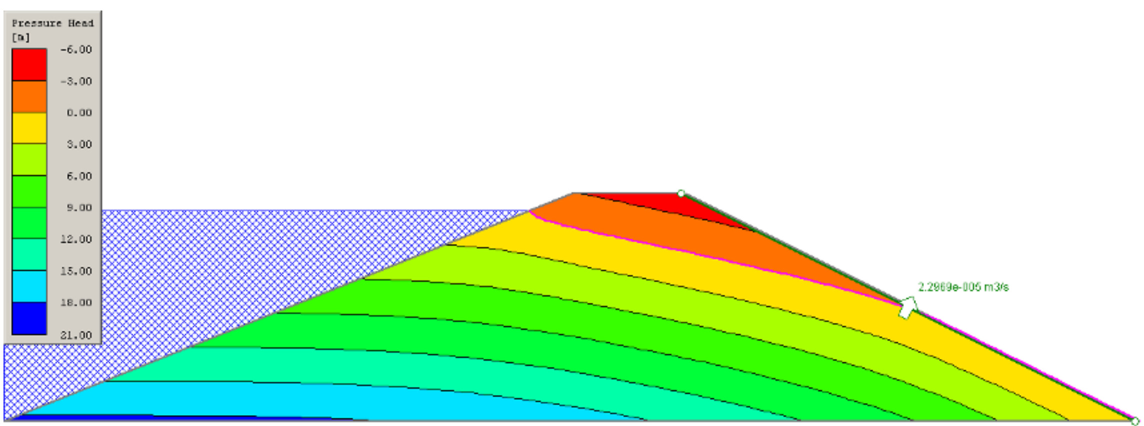


Figure 9.3: Pressure head contours

Figure 9.3 presents the flow vectors and the location of the phreatic line from **Slide** ground water model. Figure 9.4 shows the contours of total head with flow lines in the homogenous dam.

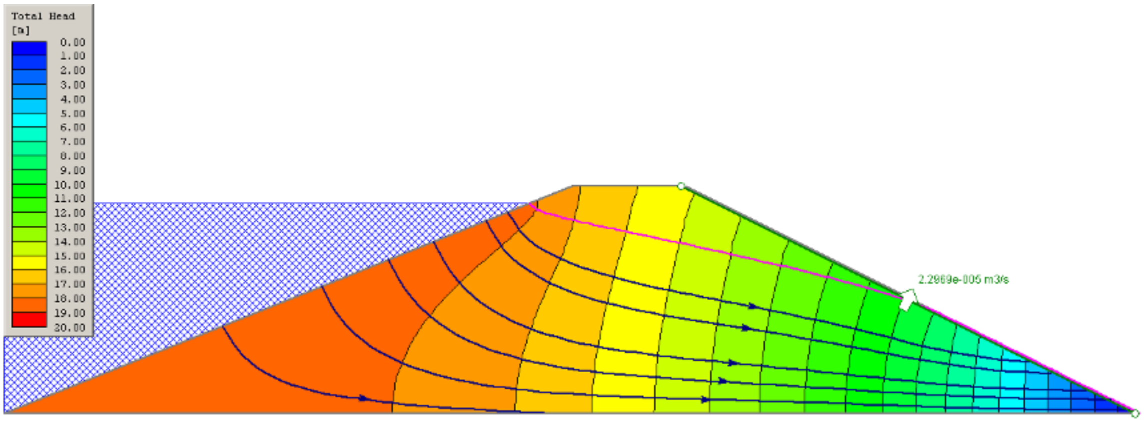


Figure 9.4: Total head contours with flow lines

Note: See file *Groundwater#09_1.sli*

2. <u>Dam with impervious core</u>

The second problem in this section considers a dam with an impervious core (Figure 9.5). The hydraulic function for the dam and the drain material are assumed to have a variation shown in Figure 9.6

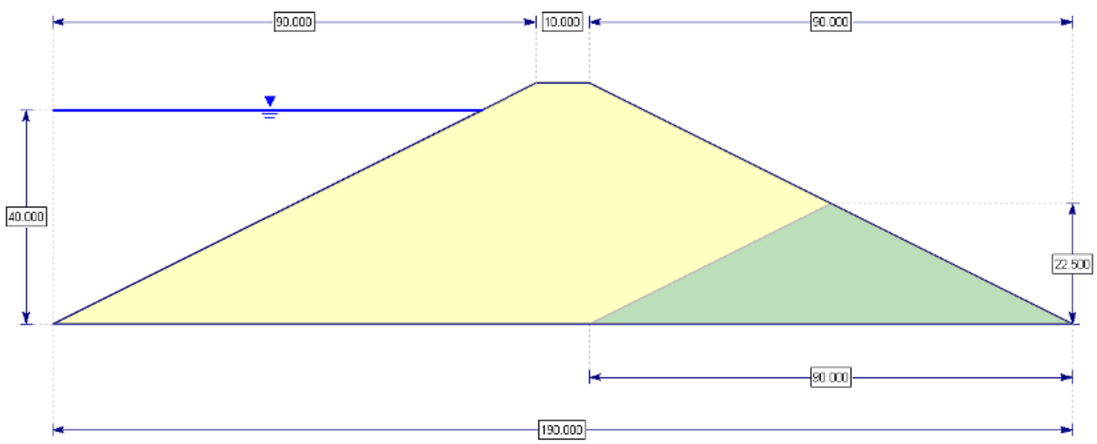


Figure 9.5: Dam with impervious core geometry detail

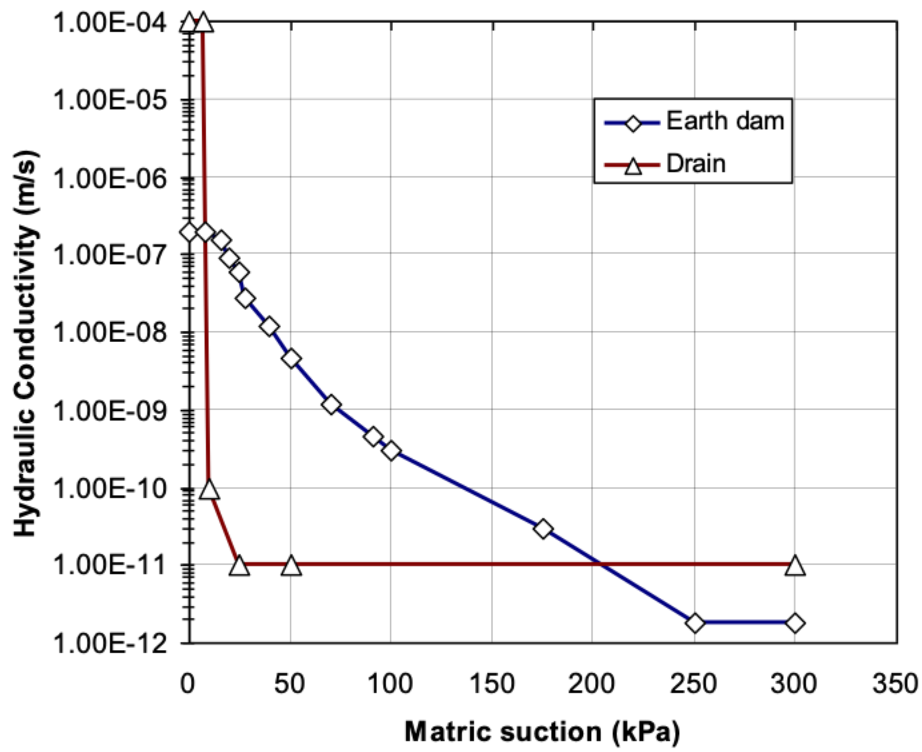


Figure 9.6

Slide gave a flow rate of Q = 4.23x10^{-6} m^3/(min.m) which compared well with the flow rate estimated by Bowels [1], Q = 3.8x10^{-6} m^3/(min.m). Chapuis et al. [2] solved the same example using finite element software SEEP/W. The flow rate calculated using SEEP/W was 5.1x10^{-6} m^3/(min.m) for a coarse mesh and a flow rate of 4.23x10^{-6} m^3/(min.m) for a finer mesh of 2328 elements.

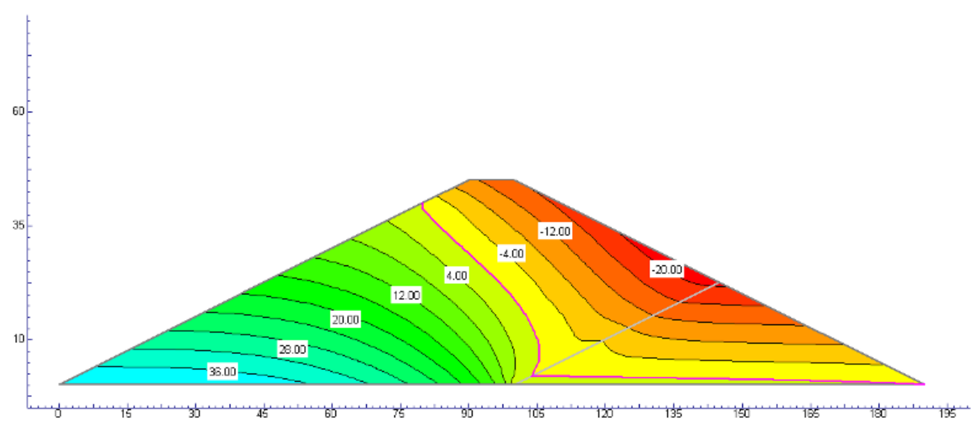


Figure 9.7: Pressure head contours

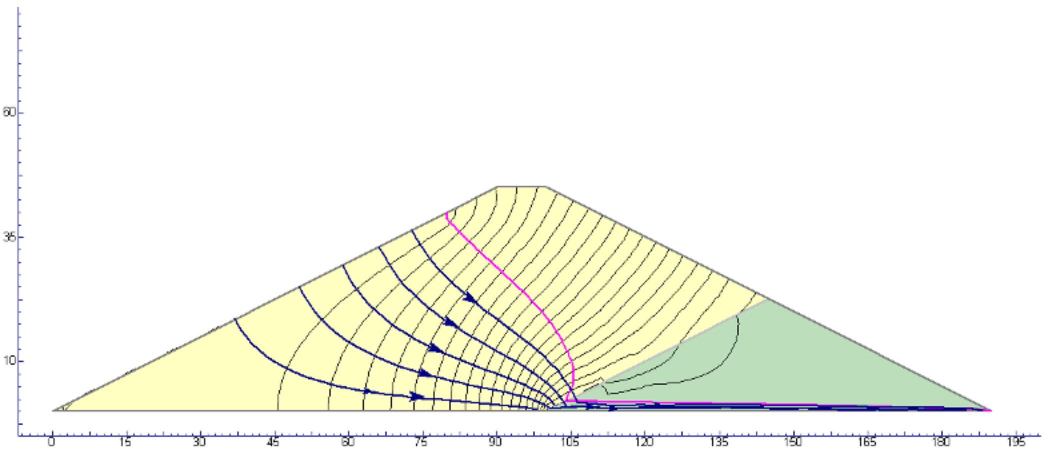


Figure 9.8: Total head contours with flow lines

Note: See file *Groundwater#09_2.sli*

9.3. References

1. Bowles J.E., (1984) *Physical and geotechnical properties of soils*. 2nd Ed. McGraw Hill, New York.
2. Chapuis, R., Chenaf D, Bussiere, B. Aubertin M. and Crespo R. (2001) "A user's approach to assess numerical codes for saturated and unsaturated seepage conditions", Can Geotech J. **38**: 1113-1126.

10. Steady-state unconfined flow using Van Genuchten permeability function

10.1. Problem Description

An unconfined flow in rectangle domain was analyzed in this section. The sensitivity of seepage face height to the downstream height is examined. Van Genuchten [1] closed form equation for the unsaturated hydraulic conductivity function is used to describe the soil properties for the soil model. A Dupuit-Forcheimer model [2], which assumes equipotential surfaces are vertical and flow is essentially horizontal, is also used for comparison.

10.2. Slide Model and Results

A 10mx10m square embankment has no-flow boundary conditions on the base and at the top. The water level at the left is 10m. Four different water levels (2, 4, 6 and 8m) at the downstream are considered. The soil has the saturated conductivity of $K_s = 1.1574\text{x}10^{-5}\,\text{m/sec}$. The values of the Van Genuchten soil parameters are $\alpha = 0.64\,\text{m}^{-1}$, $n = 4.65$. The geometry and the mesh discretization are presented in Figure 10.1.

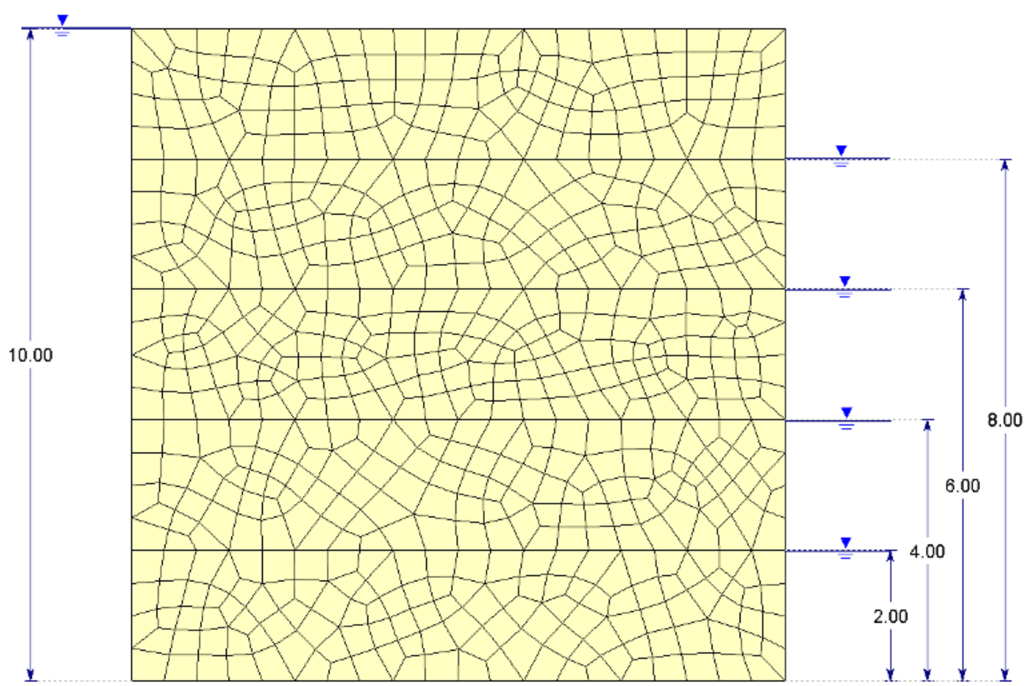
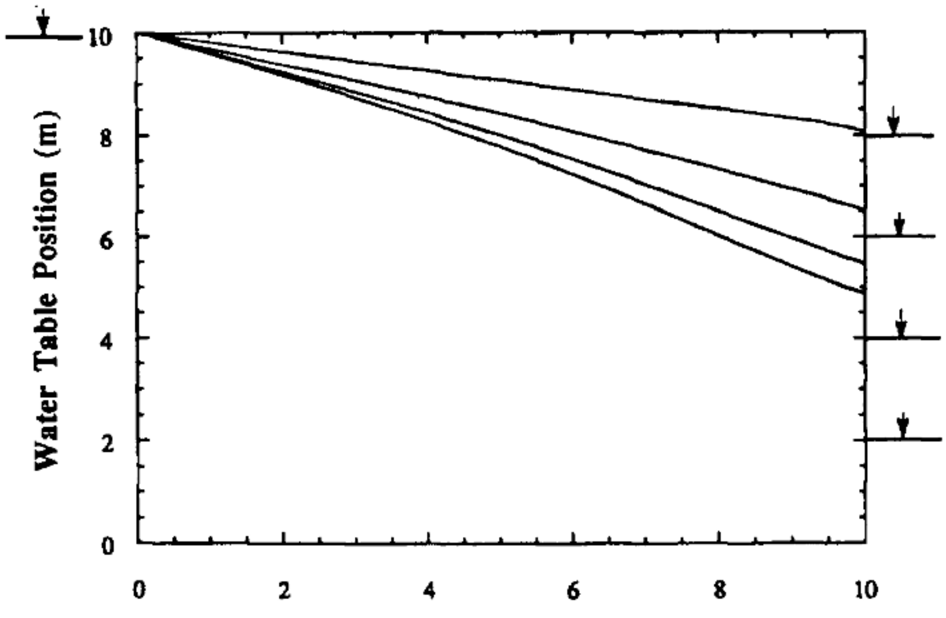


Figure 10.1: Model mesh discretization



X Coordinate (m)

Figure 10.2: Phreatic surface variation to changing downstream water level [2]

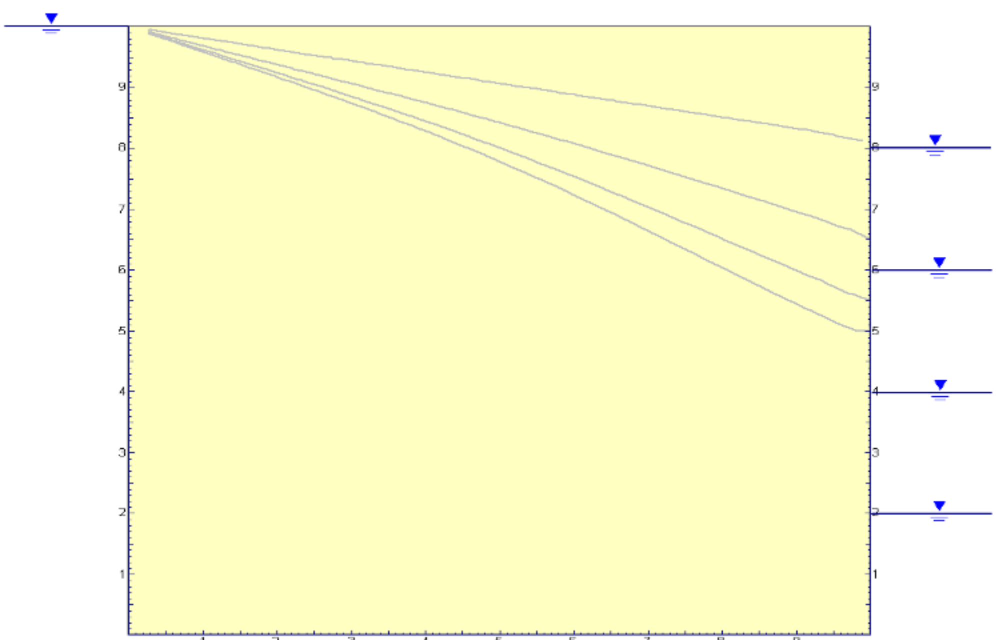


Figure 10.3: Phreatic surface variation to changing downstream water level predicted from **Slide**

Figures 10.2 – 10.3 show the variation of the phreatic surface predicted by changing downstream water level from Ref [2] and Slide respectively. These figures show that the absolute length of the seepage face decreases significantly with an increase in the water level at the downstream the results. Table 10.1 presents comparison of discharge values and seepage face from Ref. [2] and Slide.

Table 10.1: Discharge and seepage results

	MODEL DIMENSION (MXM)	TAILWATER LEVEL (M)	DISCHARGE (M/SEC)	SEEPAGE FACE (M)
Clement et. al. [2]	10x10	2	6.0764×10^{-5}	4.8
Slide	10X10	2	6.0659×10^{-5}	5.0

Note: See file *Groundwater#10_1.sli, Groundwater#10_2.sli, Groundwater#10_3.sli, Groundwater#10_4.sli*

10.3. References

1. Genuchten, V. M (1980) “A closed equation for predicting the hydraulic conductivity of unsaturated soils”, Soils Sci Soc Am J. **44**: 892-898
2. Clement, T.P, Wise R., Molz, F. and Wen M. (1996) “A comparison of modeling approaches for steady-state unconfined flow”, J. of Hydrology **181**: 189-209

11. Earth and rock-fill dam using Gardner permeability function

11.1. Problem Description

Seepage in a uniform earth and rock-fill dam is examined in this section. Nonlinear model is used to represent the seepage flow above and below the free surface. The Gardner's nonlinear equation [1] between permeability function k_w and pressure head is used in this section and it can be presented as

$$k_w = \frac{k_s}{1 + ah^n}$$

where: *a* and *n* are the model parameters

h = pressure head (suction)

k_w = permeability

k_s = saturated permeability

11.2. Slide Model and Results

1. Uniform earth and rock-fill dam

Figure 11.1 shows detailed geometry of the dam. The upstream elevation head is 40m and the downstream elevation head is 0m. The geometry of the dam is taken from Ref. [2], the slope of upstream is 1:1.98 and the slope of the downstream is 1:1.171 (Figure 11.1). The Gardner's model parameters are taken as $a = 0.15$ and $n = 6$.

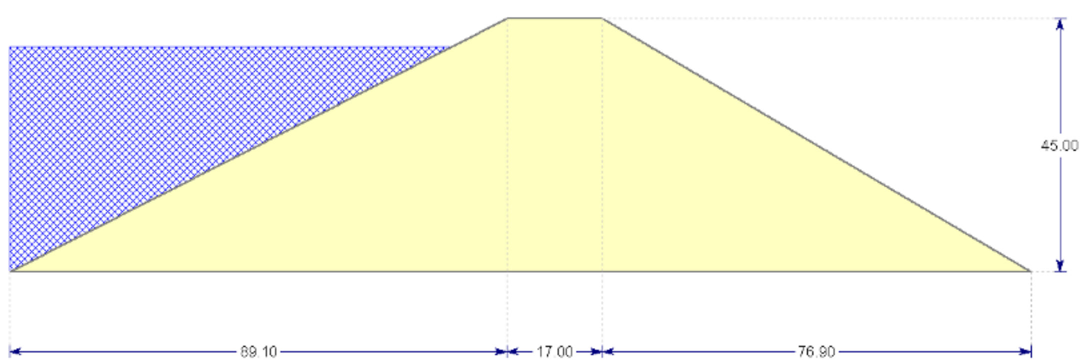


Figure 11.1: Dam geometry

Zhang et. al. [2] used general commercial software ABAQUS to analyze the earth dam and the results showed that the calculated elevation of release point is 19.64m. Same dam geometry is studied using Slide and the calculated elevation of release point is 19.397m, see Figure 11.2.

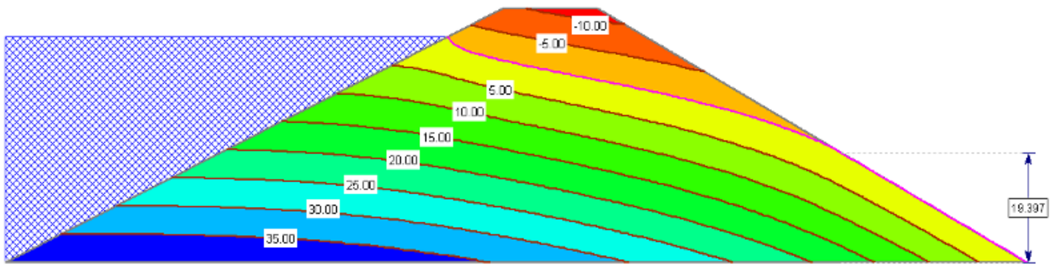


Figure 11.2: Pressure head contours

Note: See file *Groundwater#11_1.sli*

2. <u>Nonhomogeneous earth and rock-fill dam</u>

Figure 11.3 shows a dam with permeable foundation and toe drain [2]. The permeability coefficient of the foundation of sand layer is 125 times of the earth dam and blanket. The toe drain has a large value of permeability coefficient which is 10000 times larger than the permeability function of the dam. Table 11.1 shows the Gardner's parameters for the different model layers.

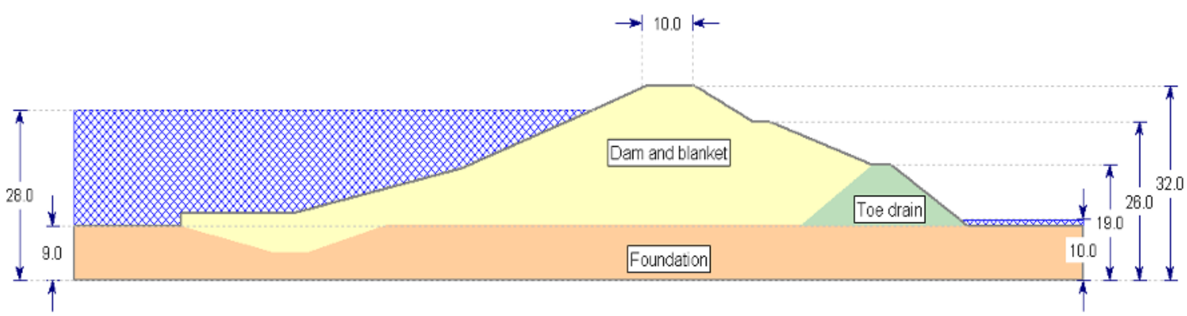


Figure 11.3: Dam geometry [2]

Table 11.1: Layers material parameters

Layer	**K_s (m/sec)**	***a***	***n***
Dam	$1x10^{-7}$	0.15	2
Foundation	$1.25x10^{-5}$	0.15	6

Figures 11.4 – 11.5 shows the distribution of the total head contours from Ref. [2] and Slide respectively. Slide results were in a good agreement with those obtained from ABAQUS.

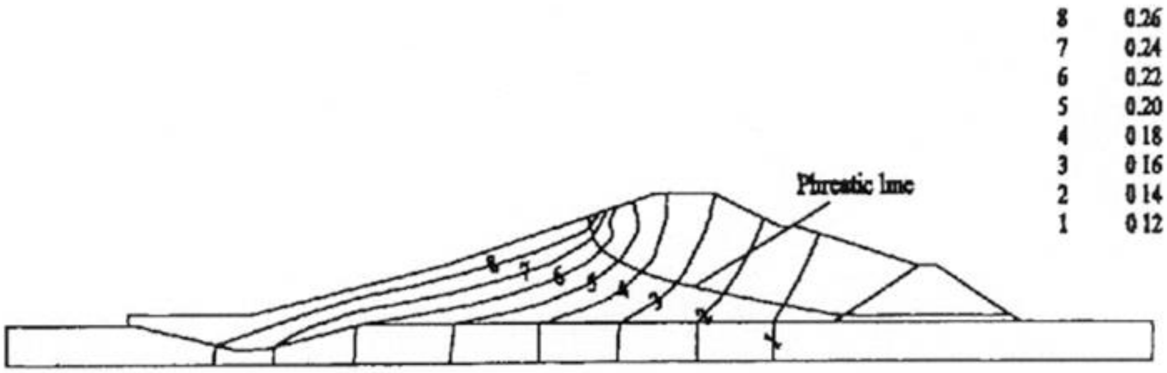


Figure 11.4: Total head (unit 10^2m) from Zhang et. al. [2]

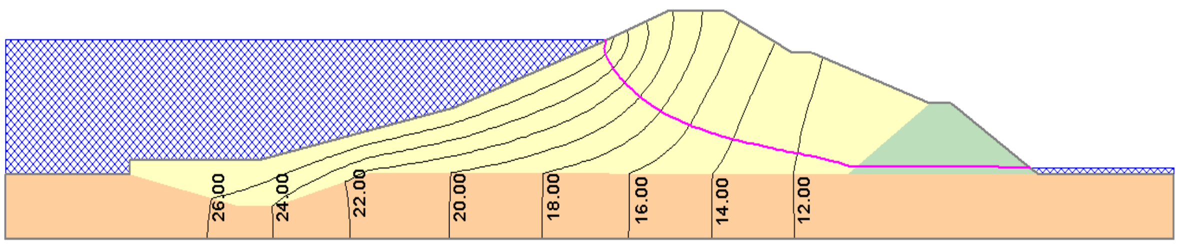


Figure 11.5: Total head contours using Slide

Note: See file *Groundwater#11_2.sli*

11.3. References

1. Gardner, W. (1956) “Mathematics of isothermal water conduction in unsaturated soils.” Highway Research Board Special Report 40 International Symposium on Physico-Chemical Phenomenon in Soils, Washington D.C. pp. 78-87.
2. Zhang, J, Xu Q. and Chen Z (2001) “Seepage analysis based on the unified unsaturated soil theory”, Mechanics Research Communications, **28** (1) 107-112.

12. Seepage from a trapezoidal ditch into a deep horizontal drainage layer

12.1. Problem Description

Seepage from a trapezoidal ditch into a deep horizontal drainage layer is analyzed in this section. The geometry of the problem is depicted in Figure 12.1.

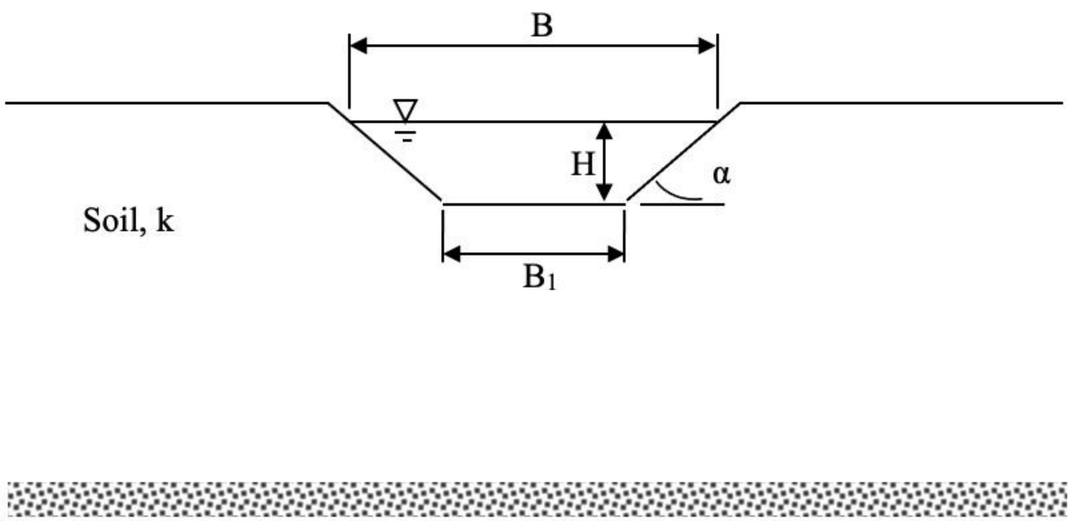


Figure 12.1: Model geometry

Vedernikov (1934) proposed a direct method to solve for the seepage from such a ditch. He proposed the following equation for calculating the flow:

$$q = k(B + AH)$$

where A is a function of B/H and cot α. In this example, we will use B=50m, H=10m and α=45° which will yield a value of A = 3. [1]

He also proposed the following equation for calculating the width of the flow at an infinite distance under the bottom of the ditch:

$$L = B + AH$$

12.2. Slide Model

The **Slide** model for the problem described in the previous section is shown in Figure 12.2. Only half of the problem was modelled because of symmetry.

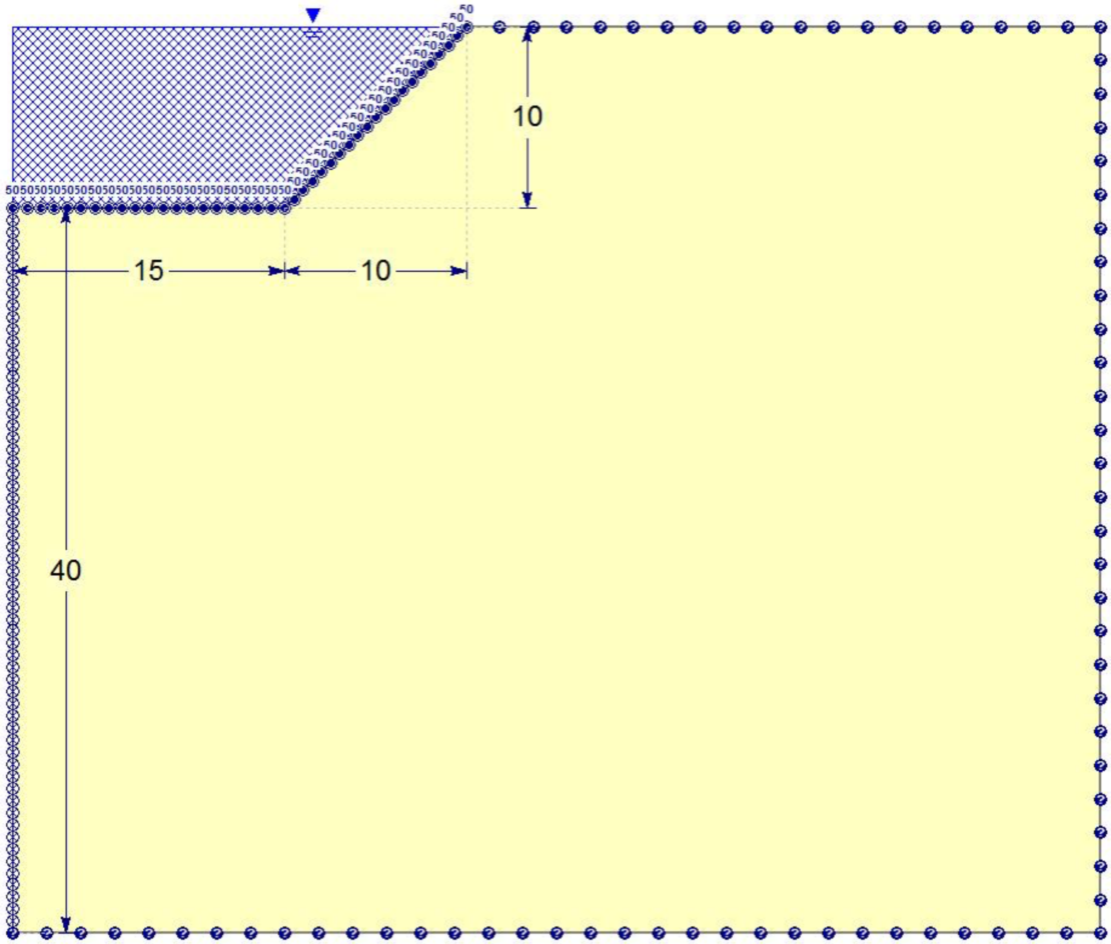


Figure 12.2: Slide model

The **Slide** model uses the following input parameters:

- B/2 = 25 m
- H = 10 m
- α = 45°
- k = 10^{-5} m/s

12.3. Slide Results

A discharge section was added to the model to compute the amount of flow and compare it to the Vedernikov solution. The output is depicted in Figure 12.3.

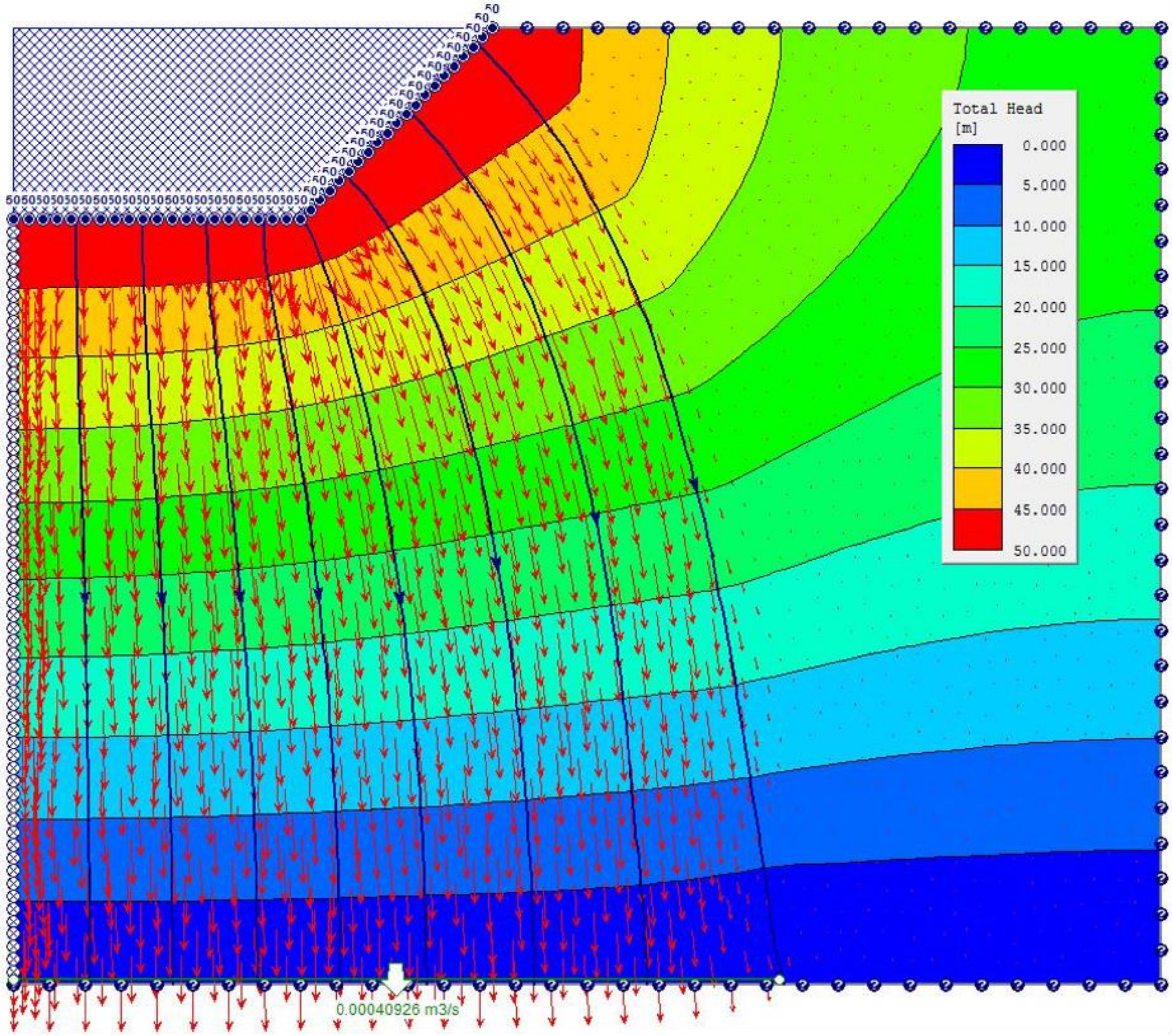


Figure 12.3: Flow net and flow vectors generated with Slide

The discharge section shows a flow of 0.00040926 m^3/s through the model. This value must be doubled in order to obtain the amount of flow through the whole system. Therefore, the amount of total flow from the trapezoidal ditch is **0.000819 m^3/s**.

When analyzing the flow vectors, the seepage zone appears to be approximately 41 m wide on the model, but this must be doubled to obtain the seepage zone for the entire ditch. Thus, the seepage zone is approximately **82m** wide.

12.4. Vedernikov's solution

Using the equations presented in section 12.1, the flow through the system was calculated to be **0.0008 m^3/s**. This is in very close agreement with the **Slide** output. The width of the seepage zone was calculated to be **80 m**, which is also in very close agreement with the Slide output.

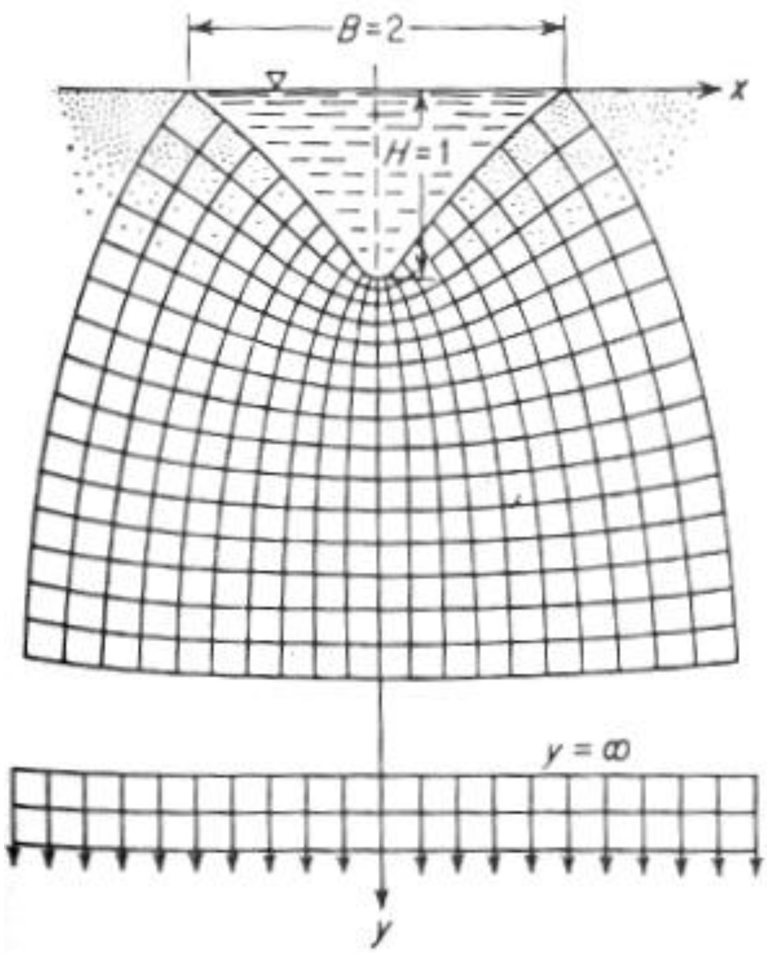


Figure 12.4: Theoretical flow net (from Harr, 1990)

12.5. Total head

A material query was added at the center of the ditch to obtain the total head through the depth and the total head was plotted against the depth. An analytical solution was also obtained, and total head values were plotted against depth as well. The analytical solution was a flow net drawn by hand using Vedernikov's boundary conditions (width of seepage zone, depth to horizontal equipotential lines). Figure 12.5 shows the flow net used to obtain the analytical solution and Figure 12.6 depicts the comparison between the values obtained using **Slide** and the analytical solution.

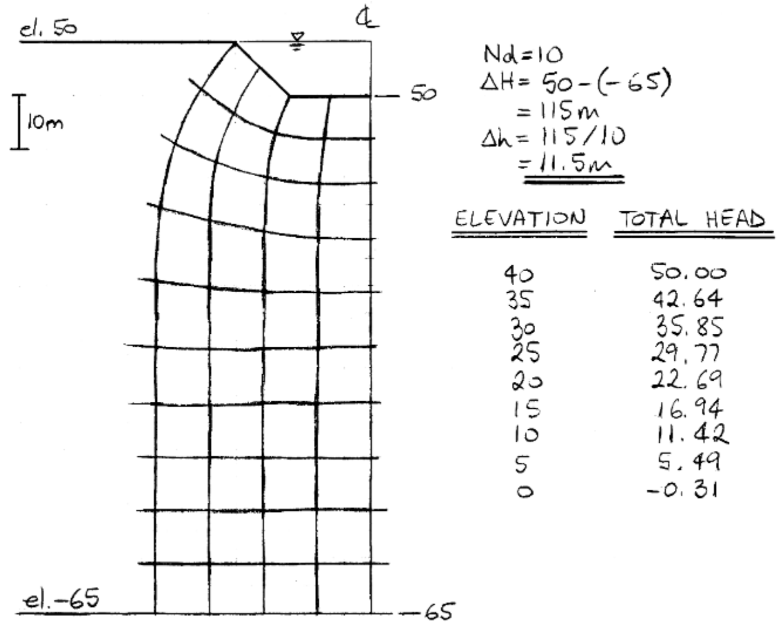


Figure 12.5: Analytical solution (flow net)

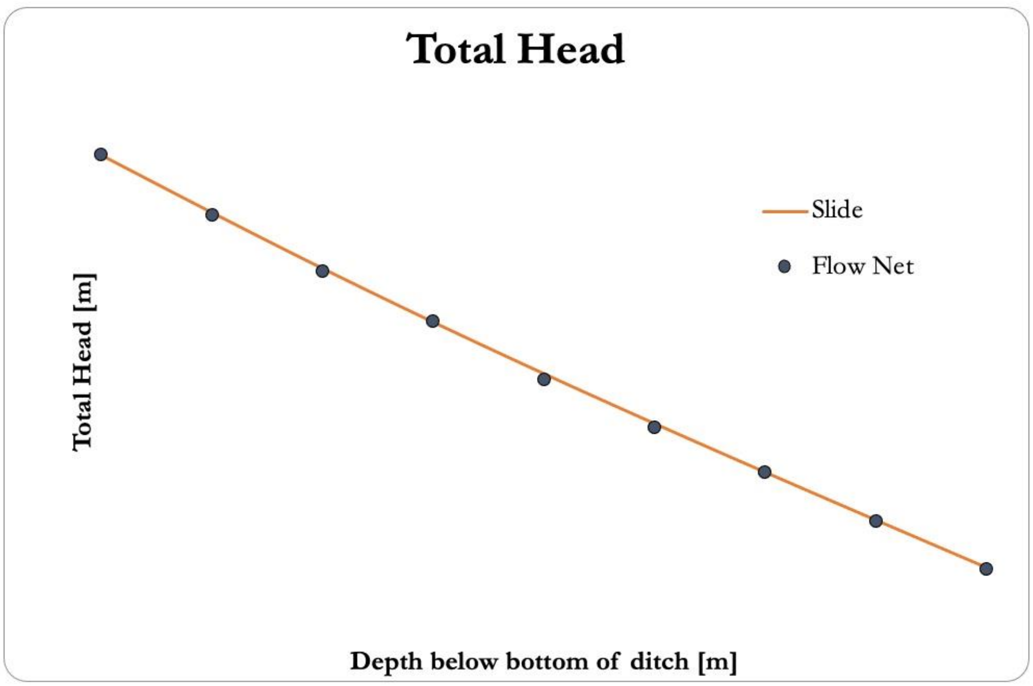


Figure 12.6: Comparison of **Slide** and analytical solution of total head

12.6. References

1. Haar, M. E. (1990) *Groundwater and Seepage*, 2nd Edition, Dover.

Note: See file *Groundwater#12.sli*

13. Seepage from a triangular ditch into a deep horizontal drainage layer

13.1. Problem Description

Seepage from a triangular ditch into a deep horizontal drainage layer is analyzed in this section. The geometry of the problem is depicted in Figure 13.1.

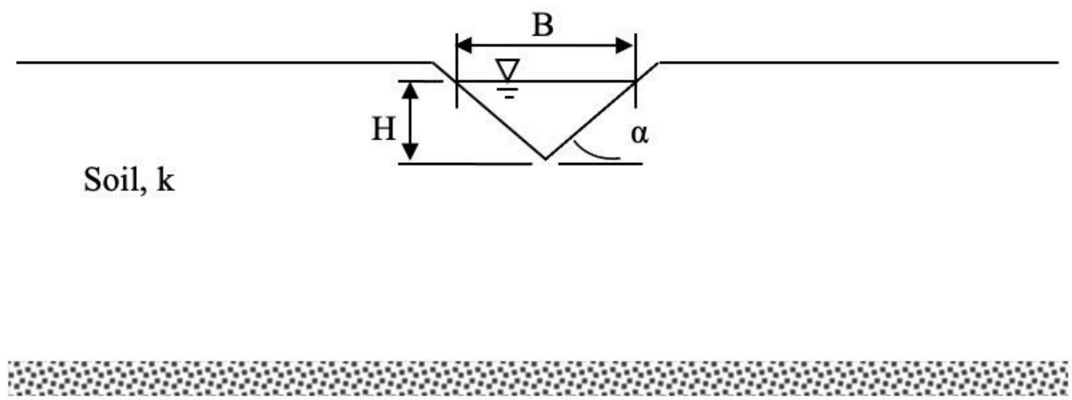


Figure 13.1: Model geometry

Vedernikov (1934) proposed a direct method to solve for the seepage from such a ditch. He proposed the following equation for calculating the flow:

$$q = k(B + AH)$$

where A is a function of α. In this example, we will use B=20m, H=10m and α=45° which will yield a value of A = 2 [1].

He also proposed the following equation for calculating the width of the flow at an infinite distance under the bottom of the ditch:

$$L = B + AH$$

13.2. Slide Model

The **Slide** model for the problem described in the previous section is shown in Figure 13.2. Only half of the problem was modelled because of symmetry.

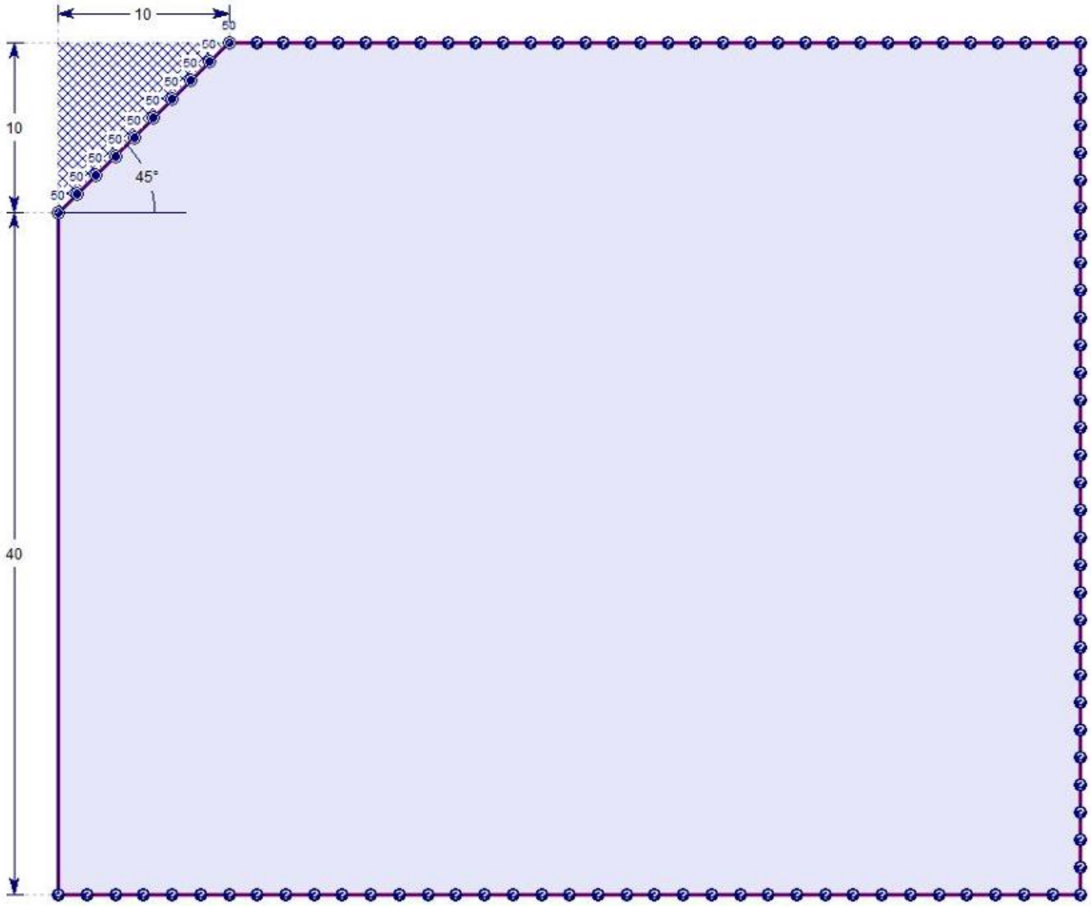


Figure 13.2: Slide model

The **Slide** model uses the following input parameters:

- B/2 = 10 m
- H = 10 m
- α = 45°
- k = 10^{-3} m/s

13.3. Slide Results

A discharge section was added to the model to compute the amount of flow and compare it to the Vedernikov solution. The output is depicted in Figure 13.3.

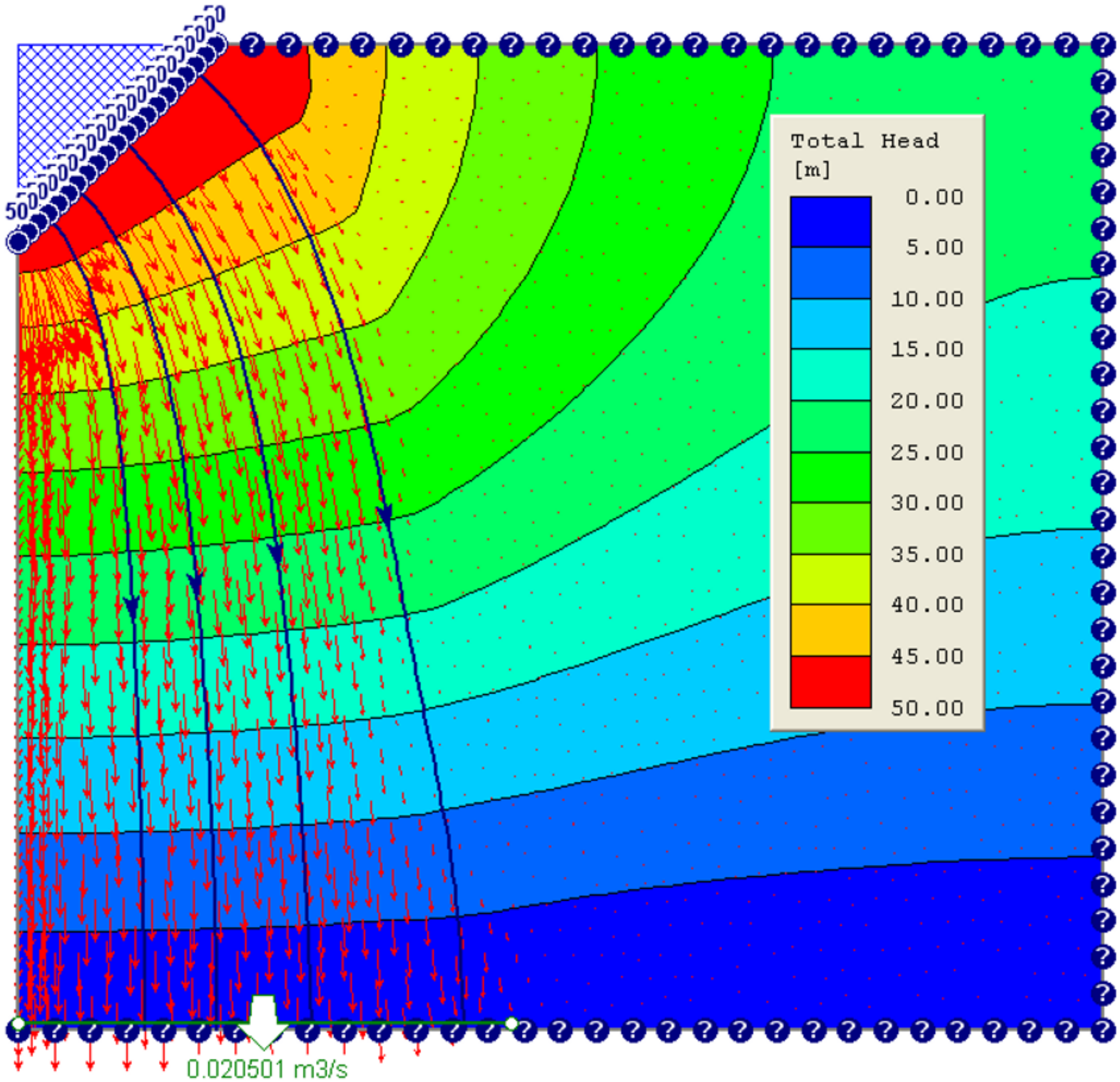


Figure 13.3: Flow net and flow vectors generated with Slide

The discharge section shows a flow of 0.020501 m^3/s through the model. This value must be doubled in order to obtain the amount of flow through the whole system. Therefore, the amount of total flow from the triangular ditch is **0.0410 m^3/s**.

When analyzing the flow vectors, the seepage zone appears to be approximately 22 m wide on the model, but this must be doubled to obtain the seepage zone for the entire ditch. Thus, the seepage zone is approximately **44 m** wide.

13.4. Vedernikov's solution

Using the equations presented in section 13.1, the flow through the system was calculated to be **0.04 m^3/s**. This is in very close agreement with the **Slide** output. The width of the seepage zone was calculated to be **40 m**, which is also in very close agreement with the Slide output.

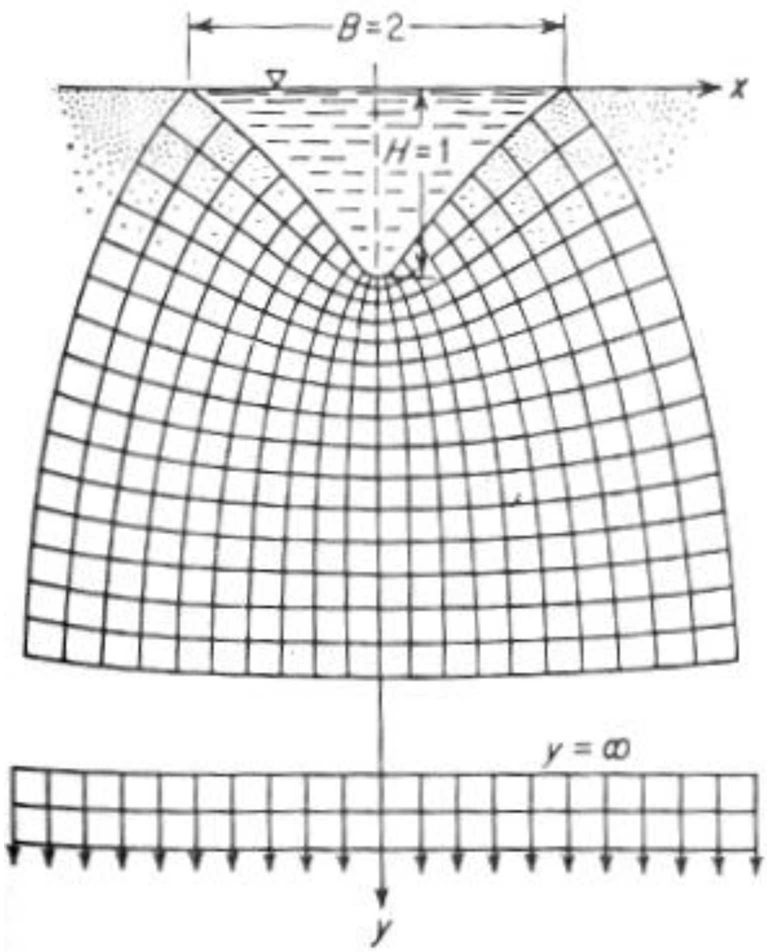


Figure 13.4: Theoretical flow net (from Harr, 1990)

13.5. Total head

A material query was added at the center of the ditch to obtain the total heads through the depth and the total head was plotted against the depth. An analytical solution was also obtained, and total head values were plotted against depth as well. The analytical solution was a flow net drawn by hand using Vedernikov's boundary conditions (width of seepage zone, depth to horizontal equipotential lines). Figure 13.5 shows the flow net used to obtain the analytical solution and Figure 13.6 depicts the comparison between the values obtained using *Slide* and the analytical solution.

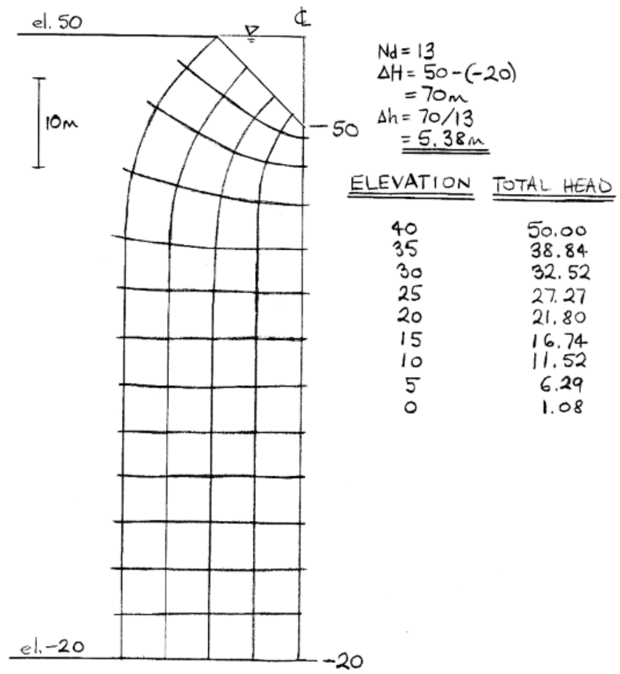


Figure 13.5: Analytical solution (flow net)

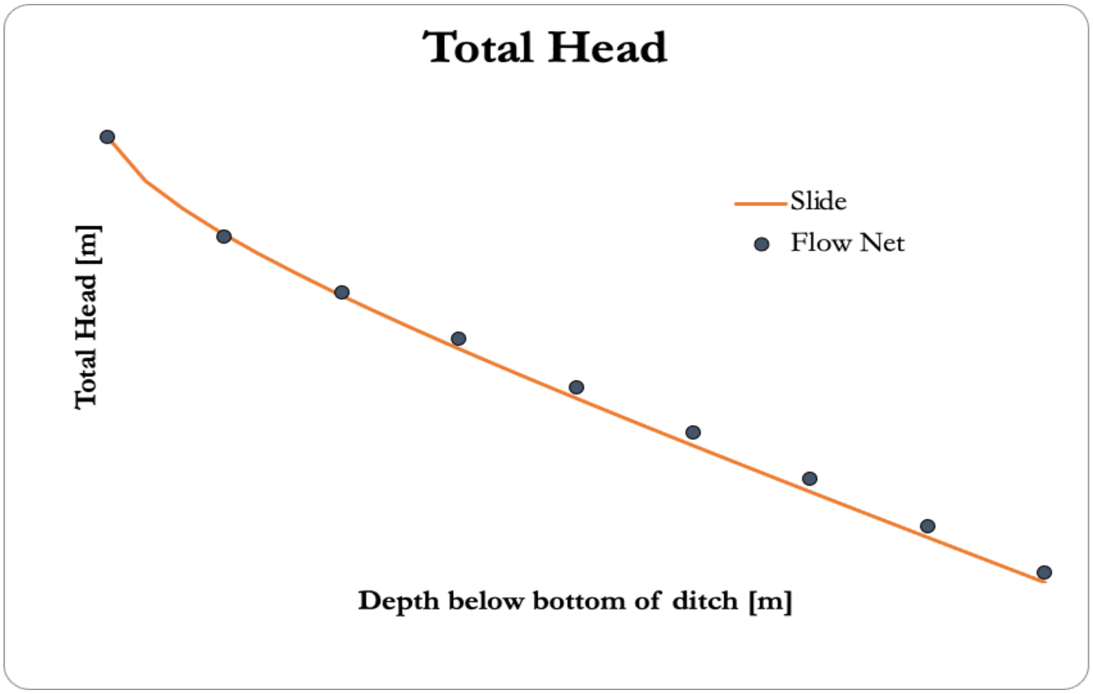


Figure 12.6: Comparison of Slide and analytical solution of total head

13.6. References

1. Haar, M. E. (1990) *Groundwater and Seepage*, 2nd Edition, Dover.

   Note: See file *Groundwater#13.sli*

14. Unsaturated soil column

14.1. Problem Description

Steady-state capillary head distribution above the water table in a soil column is analyzed in this example. The geometry of the problem is depicted in Figure 14.1.

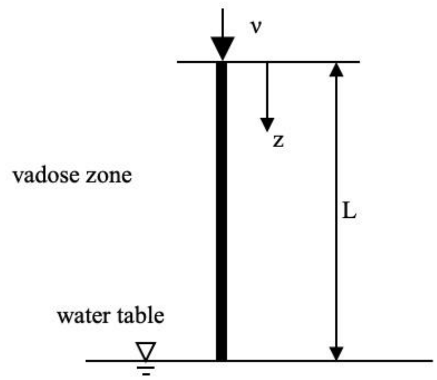


Figure 14.1: Model geometry

Gardner (1958) proposed an analytical solution to this problem. He proposed the following equation for calculating capillary head:

$$\psi(z) = -\frac{1}{\alpha}\ln\left[\frac{1}{K_s}\left(\nu + \left(K_s - \nu\right)e^{-\alpha(L-z)}\right)\right]$$

where *z* is the vertical coordinate (m), *v* is the infiltration/exfiltration rate (m/day), K_s is the saturated conductivity (m/s), *L* is the height of the column (m) and *α* is the sorptive number.

In this example, we will use L=1 m, K_s=10^{-7} m/s, v=±8.64×10^{-4} m/d and α=1 m^{-1}.

14.2. Slide Model

The **Slide** model for the problem described in the previous section is shown in Figure 14.2. The model is a very thin soil column (2 mm wide), 1 meter deep to the water table.

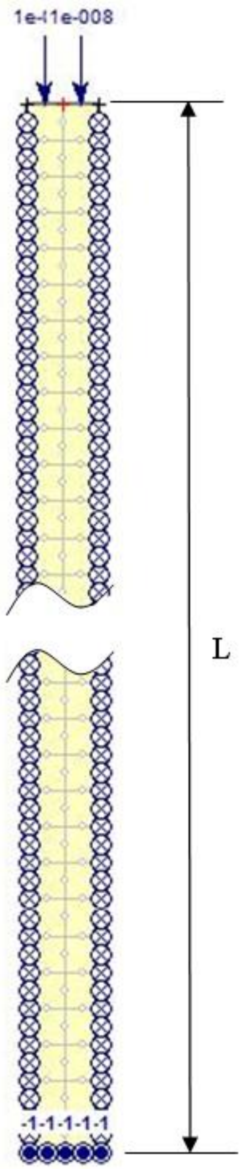


Figure 14.2: **Slide** model (infiltration)

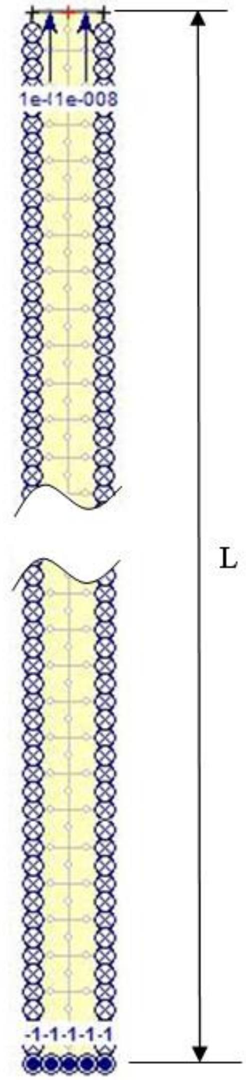


Figure 14.3: **Slide** model (exfiltration)

14.3. Results

A material query was added throughout the depth to plot the pressure head values. The output is depicted in Figure 14.3 for the constant infiltration case and in Figure 14.4 for the constant exfiltration case. The **Slide** results are in good agreement with the analytical solution presented by Gardner.

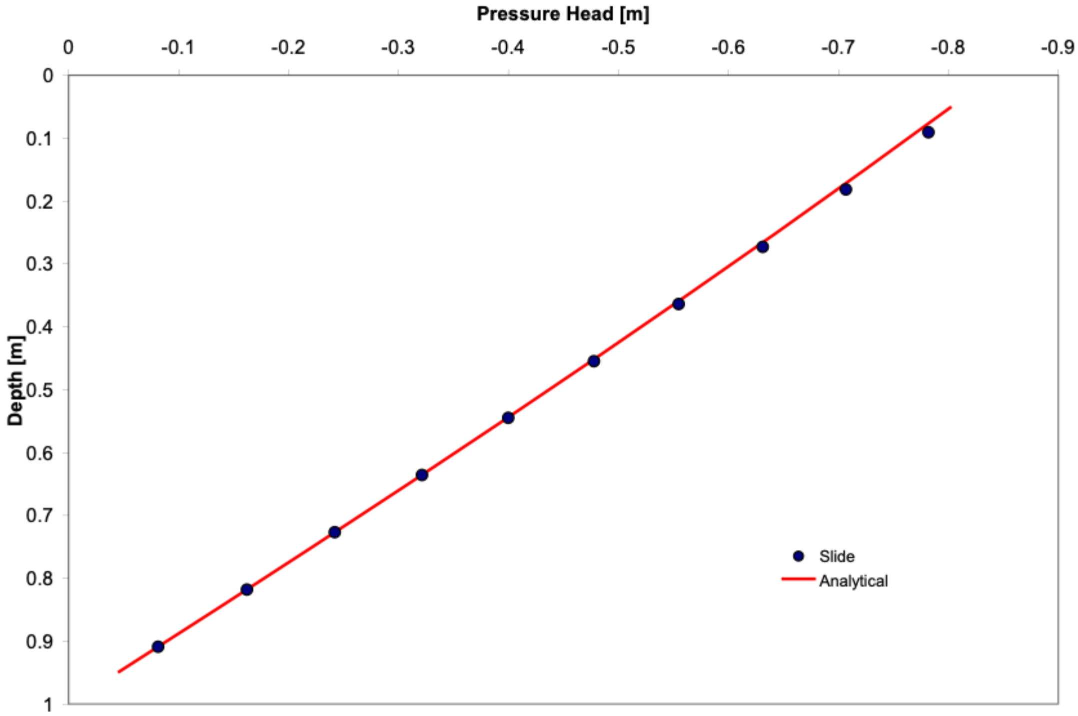


Figure 14.3: Pressure head vs. depth comparing the Gardner analytical results to the results from **Slide** for the constant infiltration case

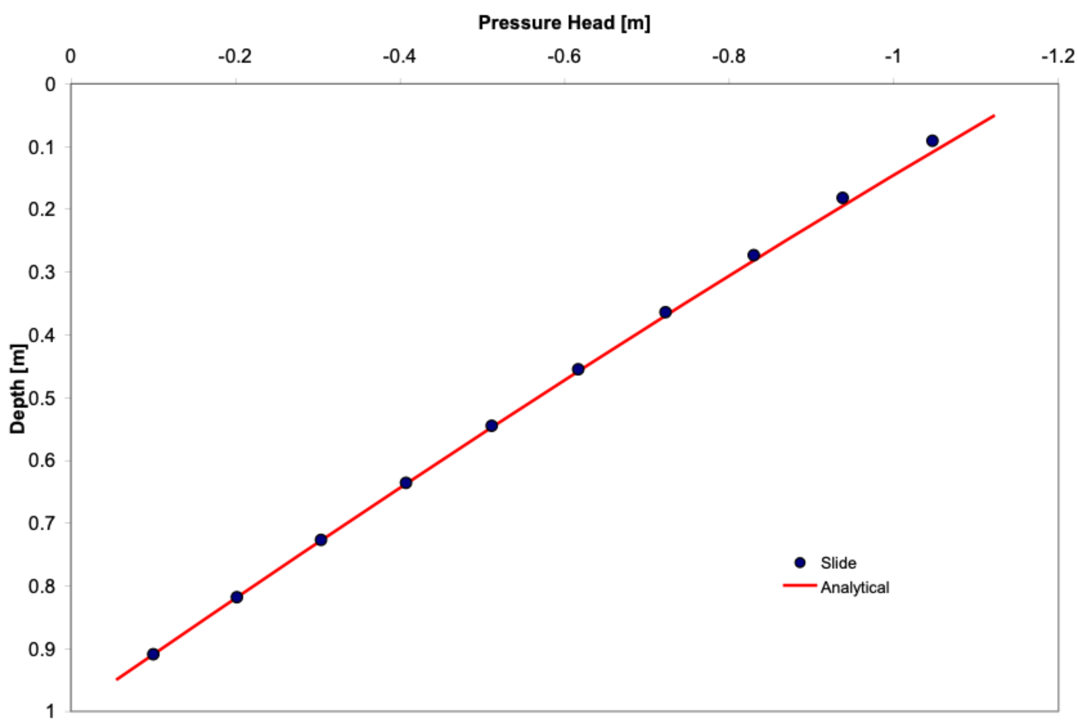


Figure 14.4: Pressure head vs. depth comparing the Gardner analytical results to the results from **Slide** for the constant exfiltration case

14.4. References

1. Gardner, W.R., *Some Steady-State Solutions of the Unsaturated Moisture Flow Equation with Application to Evaporation from a Water Table*, Soil Science 35 (1958) 4, 228-232.

   Note: See file *Groundwater#14_1.sli* and *Groundwater#14_2.sli.*

15. 1-D Consolidation with Uniform Initial Excess Pore Pressure

15.1. Problem Description

In this problem, a 1-D soil column with a height of one metre is considered. Two boundary condition cases are considered. The first case allows flow along the top and bottom edges, while the second case only allows flow along the top edge. An initial pressure head of $P =$ 100 m is applied uniformly throughout the column. This geometry is shown in Figure 15.1.

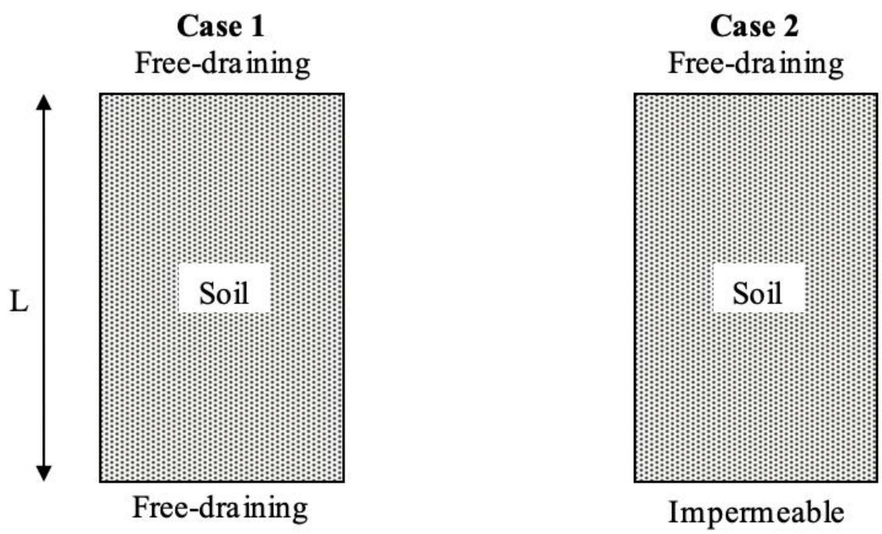


Figure 15.1: Model geometry

Terzaghi's consolidation equation can be written as:

(15.1a) $$\frac{\partial^2 u_e}{\partial Z^2} = \frac{\partial u_e}{\partial T}$$

using the dimensionless variables

(15.2a) $$Z = \frac{z}{H}$$

and

(15.2b) $$T = \frac{C_v t}{H^2}$$

where

z	=	depth from the top of the column
H	=	maximum drainage path
C_v	=	coefficient of consolidation

t = time

u_e = excess pore pressure

An initial condition is imposed at $t = 0$:

$$u_e = u_0 \text{ for } 0 \le Z \le 1$$

where

u_0 = initial excess pore pressure

Along edges where flow is allowed to occur, a boundary condition is imposed for all *t*:

$$u_e = 0$$

where

$$M = \frac{\pi}{2}(2m+1)$$

15.2. Slide Model and Results

Case 1

The **Slide** model for Case 1 is shown in Figure 15.2. A uniform initial excess pore pressure of 100 m is set.

The following properties are assumed for the soil:

- $m_w =$ 0.01 /kPa
- $C_v =$ 1.02e-4 m²/s
- $k = C_v \gamma_w m_w =$ 1e-5 m/s

The maximum drainage path is taken as L/2 = 0.5 m. The problem is modeled in **Slide** with three-noded triangular finite elements. The total number of elements used is 1580 elements.

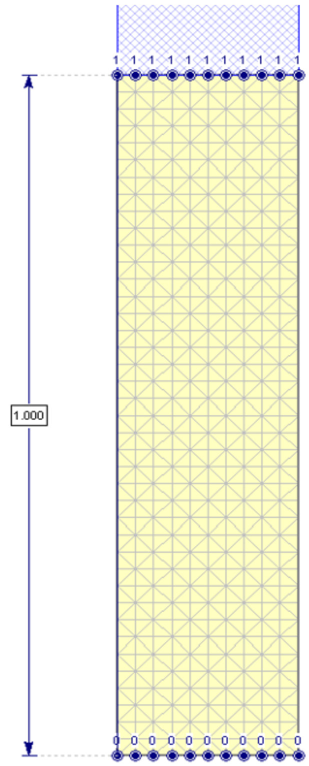


Figure 15.2: **Slide** model for Case 1

Figure 15.3 shows excess pore pressure along the soil column at different times. The single data points represent the **Slide** interpretations, while the solid lines represent values calculated using Equation 15.3. The Slide curves take the same form as published graphs such as in Ref [1]. As seen, the Slide results are in close agreement with values calculated using Equation 15.3.

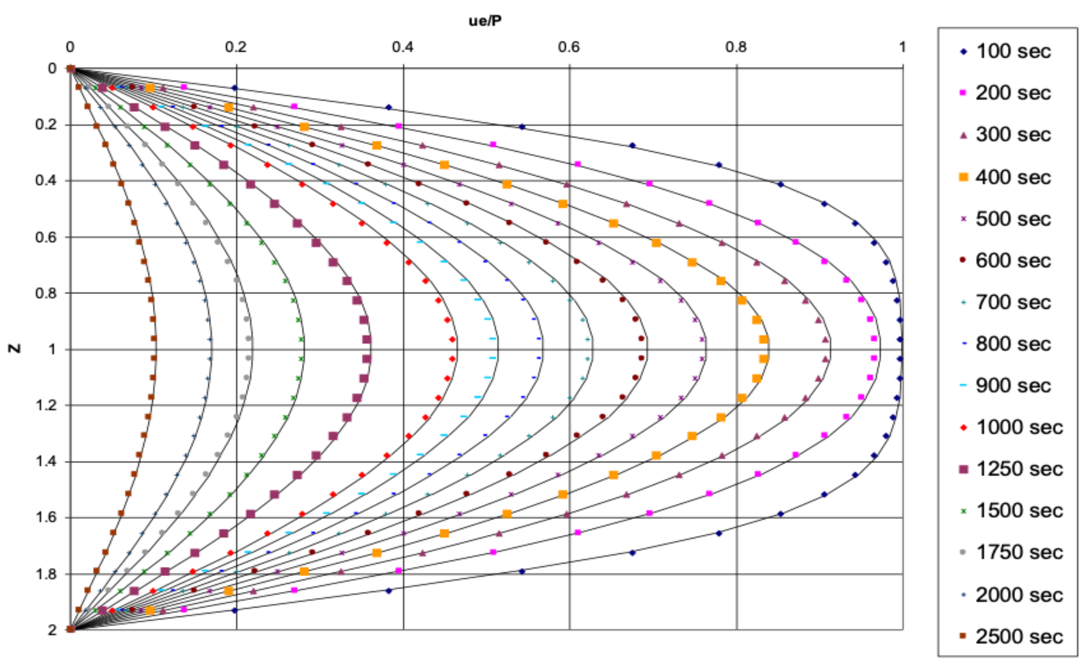


Figure 15.3: Phreatic surface variation to changing downstream water level predicted from **Slide**

Case 2

The **Slide** model for Case 2, shown in Figure 15.4, uses properties similar to Case 1. The maximum drainage path is taken as L = 1 m.

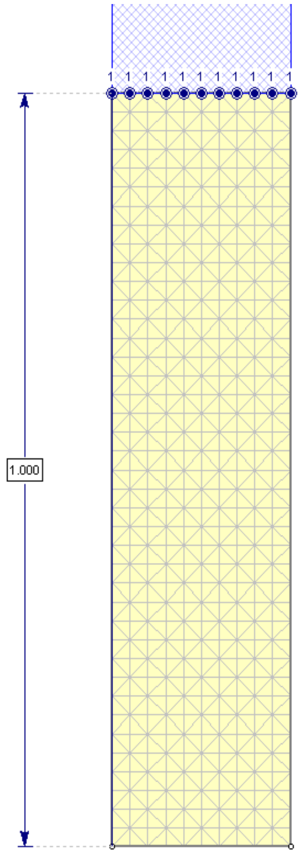


Figure 15.4: **Slide** model for Case 2

The **Slide** results for Case 2 shown in Figure 15.5 are again in close agreement with the Terzaghi consolidation equation values.

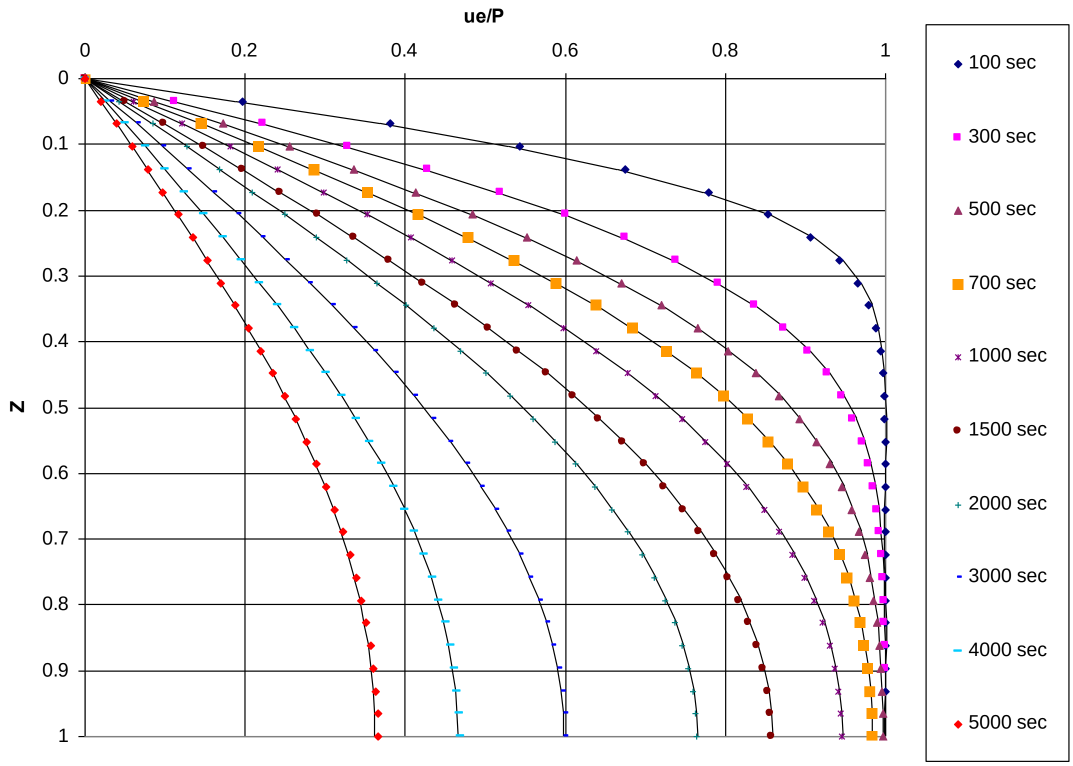


Figure 15.5: Comparison of Pore Pressure Dissipation for Case 2

15.3. References

1. T.W. Lambe and R.V. Whitman (1979) *Soil Mechanics, SI Version*, New York: John Wiley & Sons

16. Pore Pressure Dissipation of Stratified Soil

16.1. Problem Description

The problem deals with 1D consolidation of stratified soils. Three cases are considered, which are shown in Figure 16.1. The properties for Soil A and Soil B are shown in Table 16.1. Both the pore fluid specific weight (γ_w) and the height of the soil profiles are assumed to be one unit. An initial pressure head of $P =$ 1000 m is applied uniformly throughout the column.

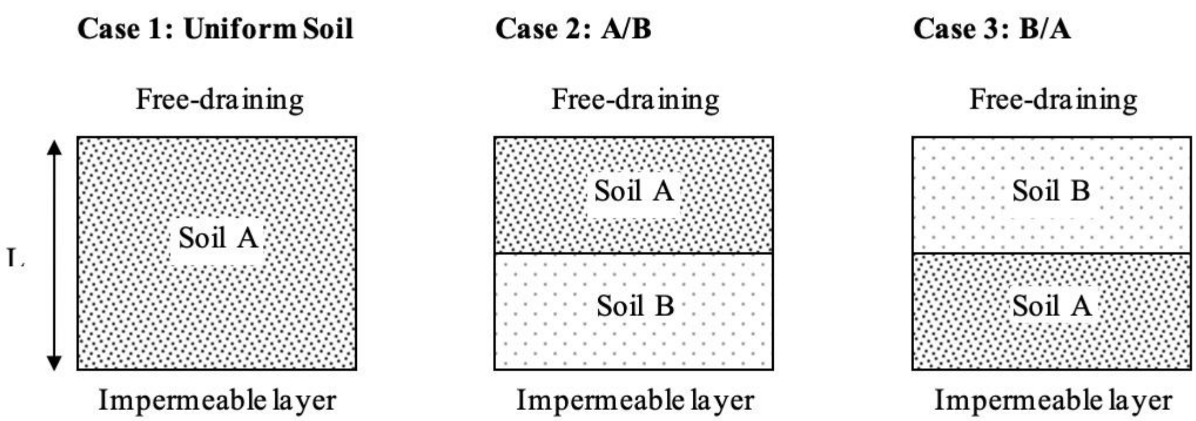


Figure 16.1: Model geometry

Table 16.1: Soil parameters

	Soil A	Soil B
k	1	10
m_v	1	10
c_v	1	1

16.2. Slide Model Results

Figures 16.2 to 16.4 show comparisons between excess pore pressures in the **Slide** model and values from the analytical solution presented in Ref [1]. The single data points represent the Slide interpretations, while the solid lines represent analytical values from Ref [1]. As shown, the Slide results are in close agreement with the analytical solutions.

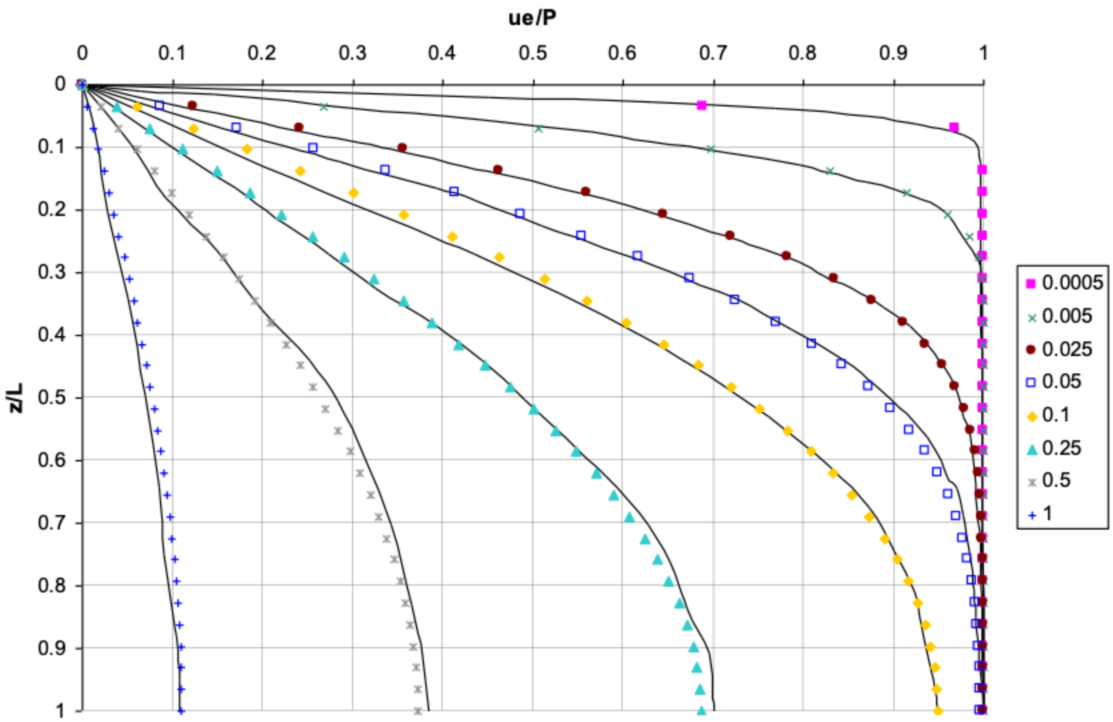


Figure 16.2: Comparison of Excess Pore Pressure for Case 1

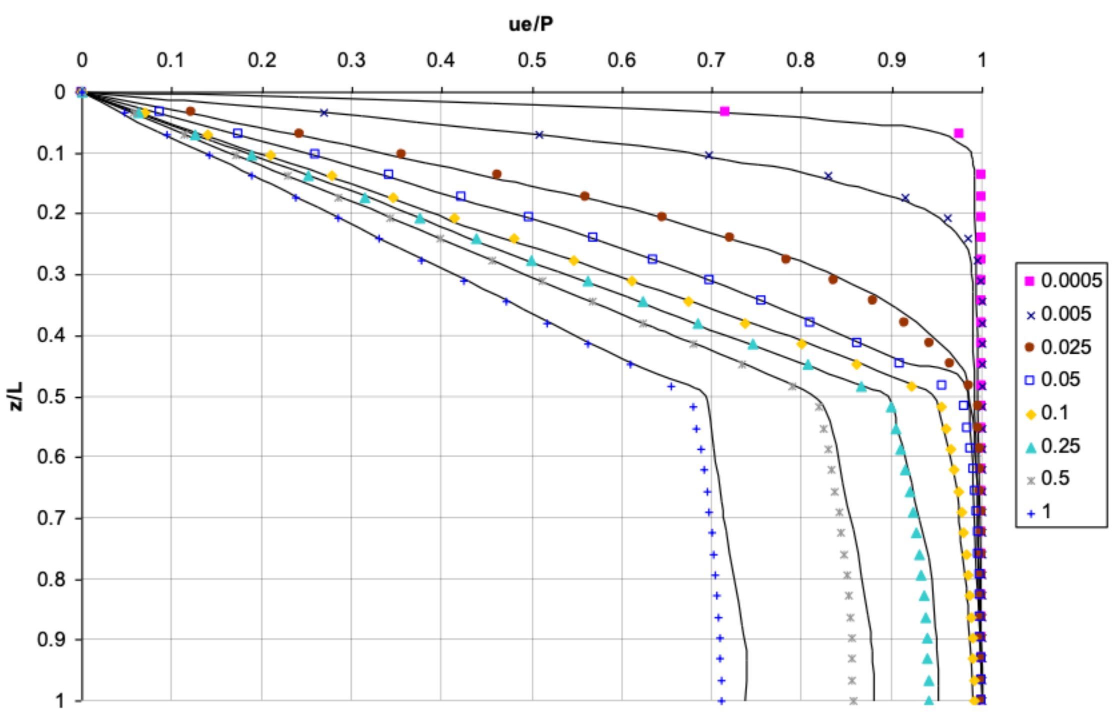


Figure 16.3: Comparison of Excess Pore Pressure for Case 2

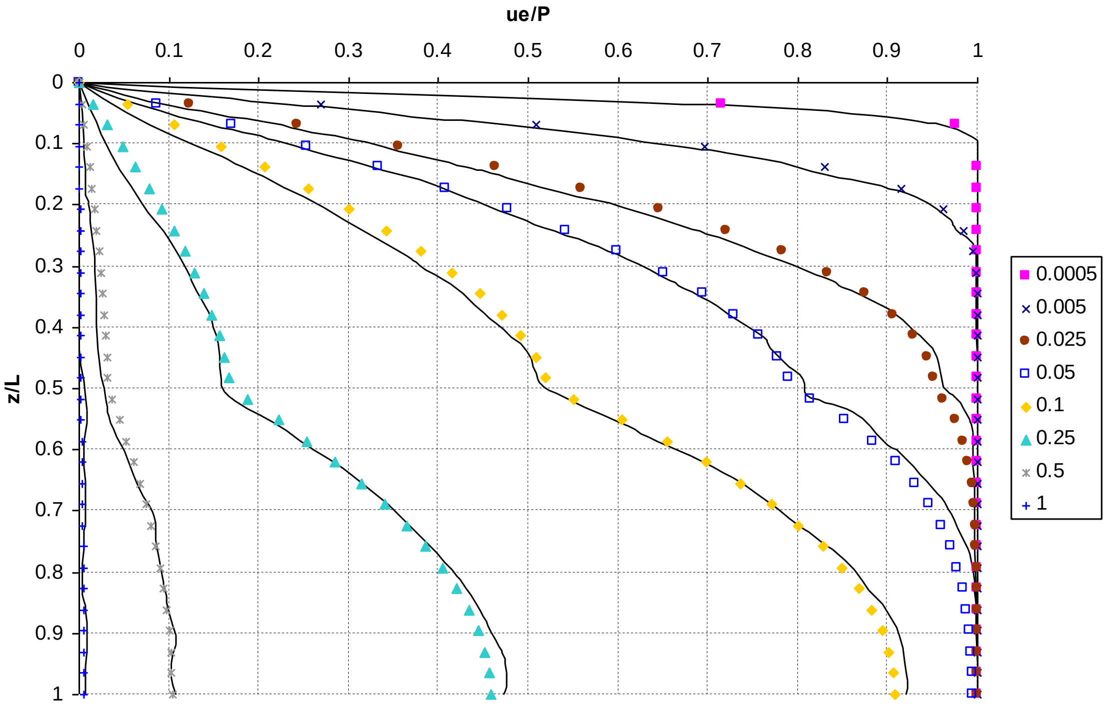


Figure 16.4: Comparison of Excess Pore Pressure for Case 3

16.3. References

1. Pyrah, I.C. (1996), “One-dimensional consolidation of layered soils”, Géotechnique, Vol. 46, No. 3, pp. 555-560.

17. Transient Seepage through an Earth Fill Dam with Toe Drain

17.1. Problem Description

In this problem, an earth fill dam with a reservoir on one side is modeled. The reservoir level is quickly raised, and transient seepage is investigated.

The base of the earth fill dam is 52 m wide and there is a 12 m wide toe drain installed at the downstream side. The initial steady-state reservoir level is 4 m. For transient analysis, the reservoir level is quickly raised to a height of 10 m. Isotropic conditions and a m_v value of 0.003 /kPa are assumed. Figure 17.1 shows the coefficient of permeabilities used for dam material.

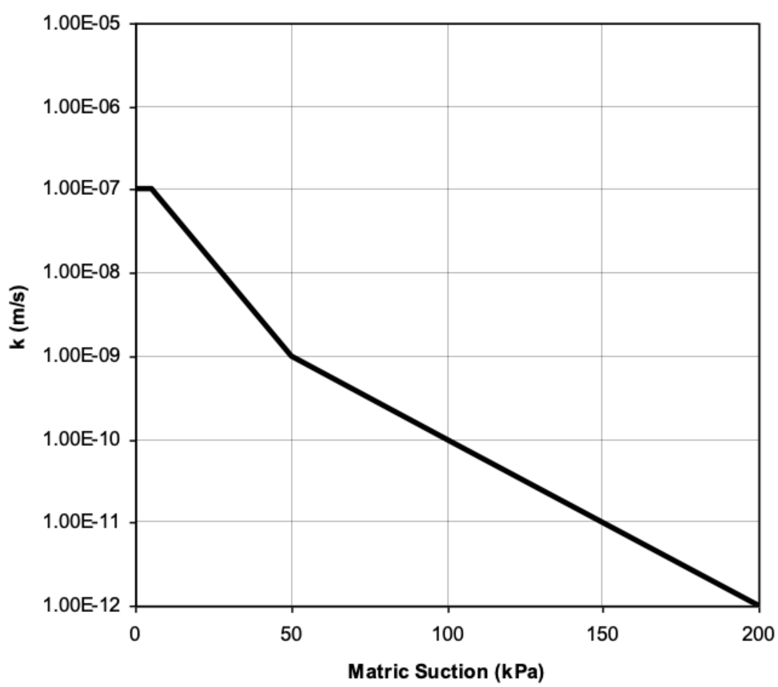


Figure 17.1: Coefficient of Permeability Function

17.2. Slide Model

The **Slide** models for initial steady state and transient analysis are shown in Figure 17.2 and 17.3, respectively. The boundary conditions simulate the rise in the reservoir water level and the installed toe drain.

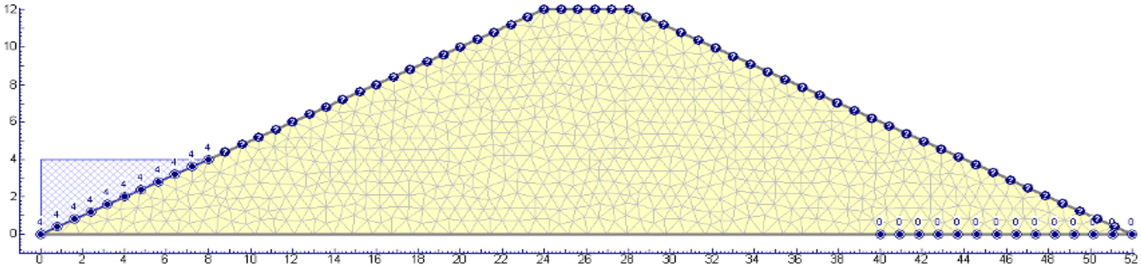


Figure 17.2: Slide Model – Initial Steady State

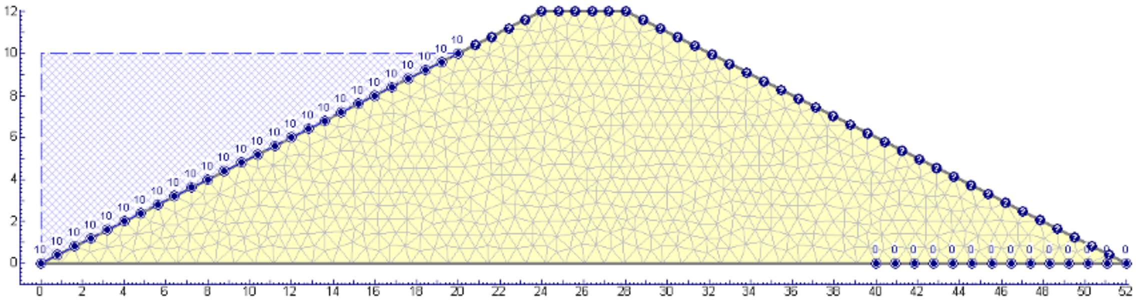


Figure 17.3: Slide Model – Transient

17.3. Results

The **Slide** model results are shown at times 15 hr and 16383 hr in Figures 17.4 and 17.5, respectively. The solid lines represent total head contour results from Slide*.* The dotted lines are solutions taken from FlexPDE results in Ref [1], while the dashed lines are SEEP/W results from Ref [1].

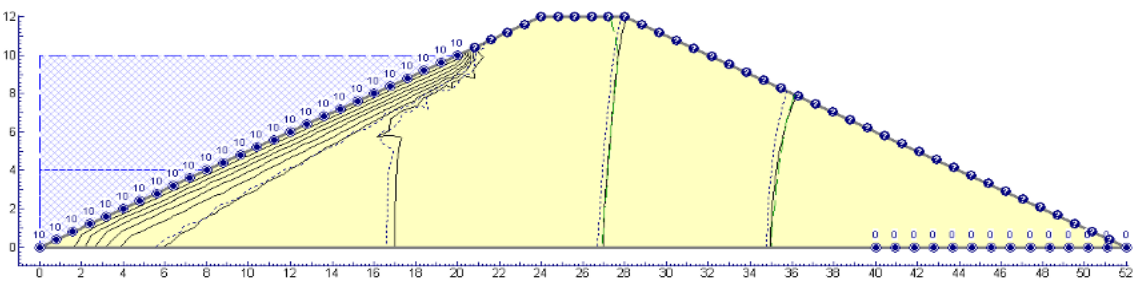


Figure 17.4: Comparison of Total Head Contours for Time 15 hr

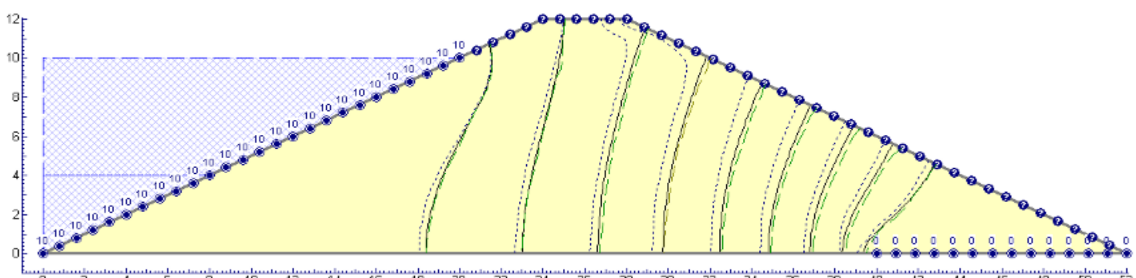


Figure 17.5: Comparison of Total Head Contours for Time 16383 hr

Figures 17.6 and 17.7 show pressure head contours at times 15 hr and 16383 hr, respectively.

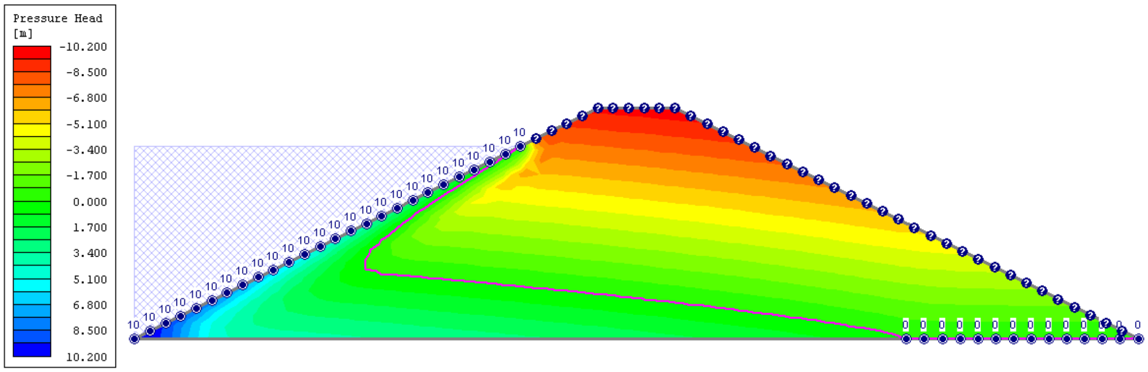


Figure 17.6: Pressure Head Contours for Time 15 hr

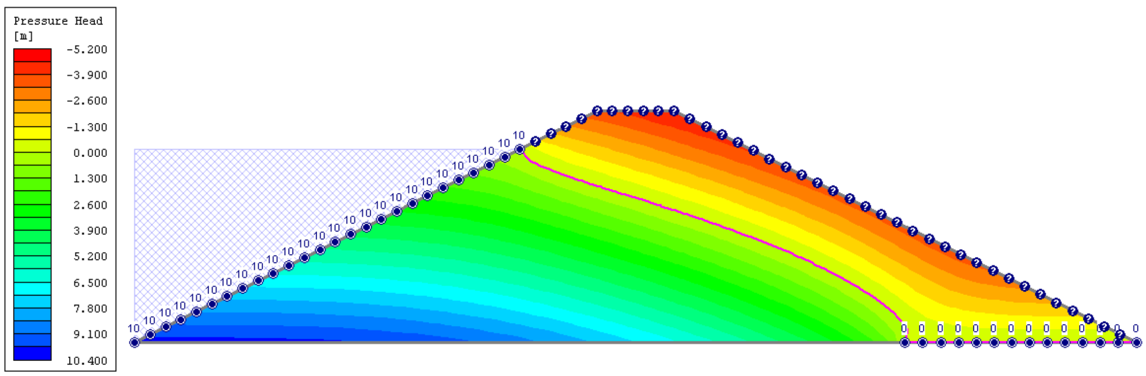


Figure 17.7: Pressure Head Contours for Time 16383 hr

17.4. References

1. Pentland, et. al (2001), "Use of a General Partial Differential Equation Solver for Solution of Mass and Heat Transfer Problems in Geotechnical Engineering", 4th Brazilian Symposium on Unsaturated Soil, pp. 29-45

18. Transient Seepage through an Earth Fill Dam

18.1. Problem Description

This problem is similar to Verification Example 17.

The base of the earth fill dam is 52 m wide but there is no toe drain. The reservoir level is raised from 4 m to 10 m at the start of analysis time. Isotropic conditions and a m_v value of 0.003 /kPa are assumed for the earth fill. Figure 18.1 shows the coefficient of permeabilities used for the dam material.

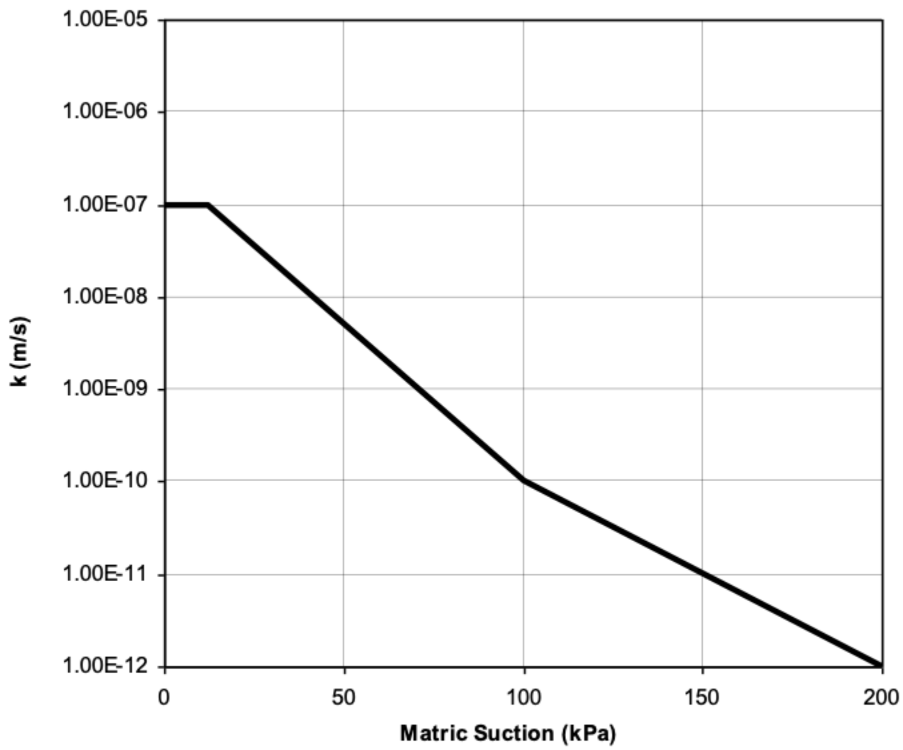


Figure 18.1: Coefficient of Permeability Function for Dam Material

18.2. Slide Model

The **Slide** models for initial steady state and transient analysis are shown in Figure 18.2 and 18.3, respectively. The boundary conditions simulate the rise in the reservoir water level.

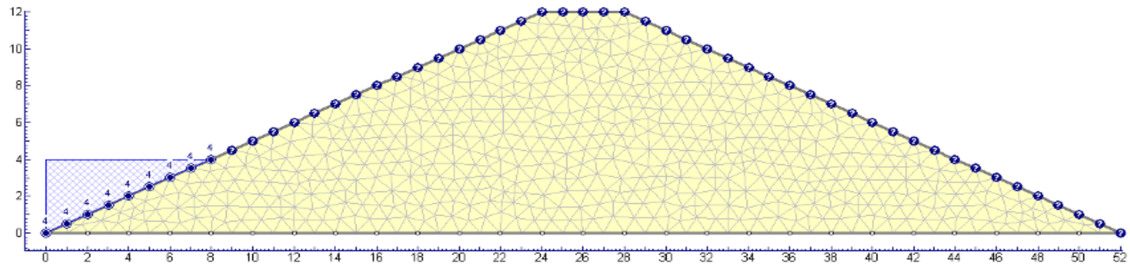


Figure 18.2: Slide Model – Initial Steady State

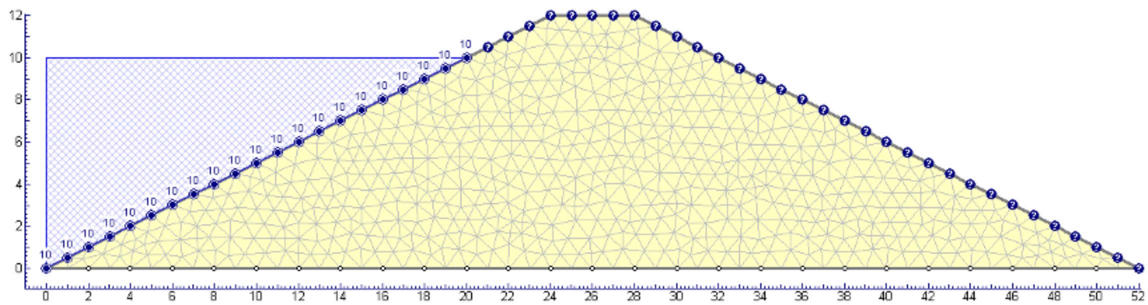


Figure 18.3: Slide Model – Transient

18.3. Results

Total head values are sampled along the toe slope as shown in Figure 18.4. These values are compared with values taken from Ref [1] in Figure 18.5. As can be seen, the values are in agreement.

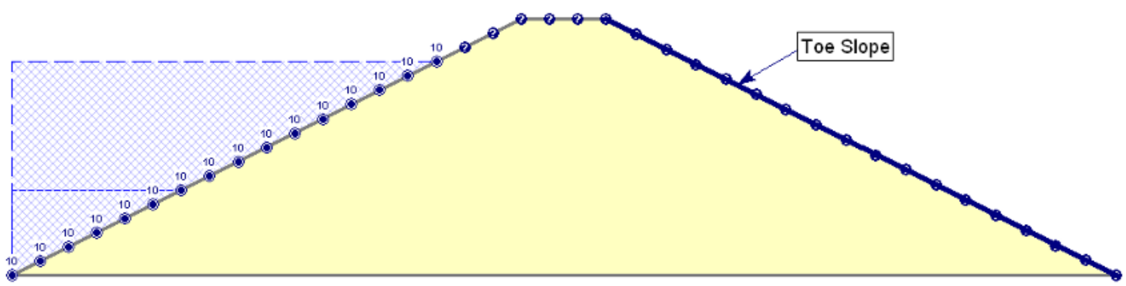


Figure 18.4: Toe Slope

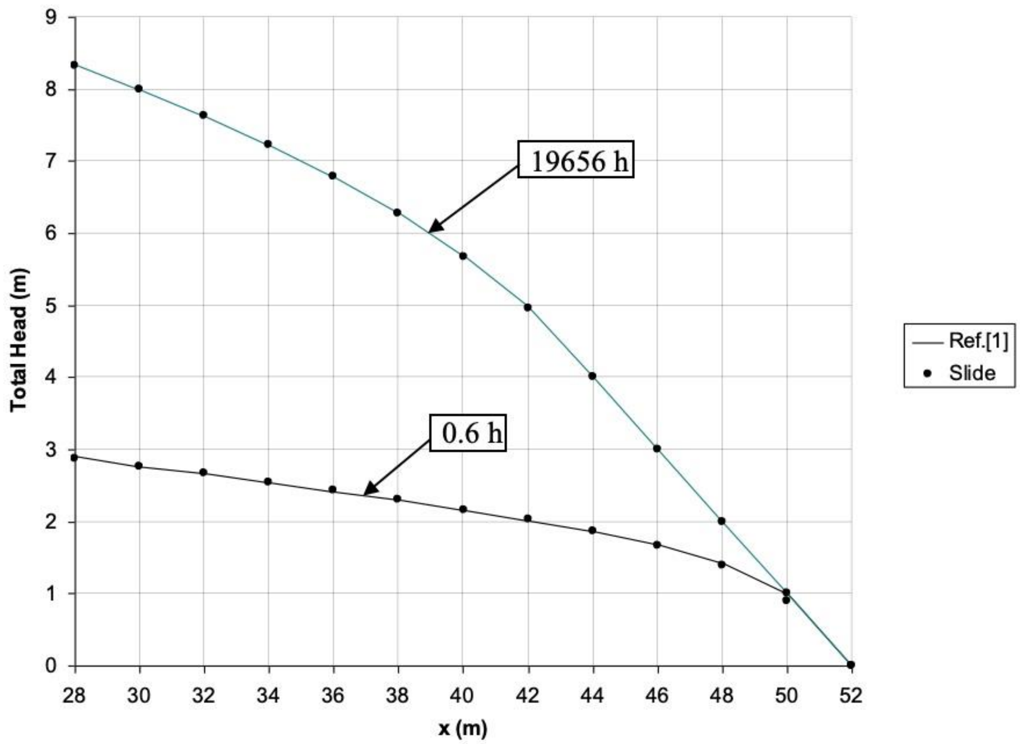


Figure 18.5: Total Head Comparison

Figures 18.6 and 18.7 show total head contours for times of 0.6 h and 19656 h, respectively. Figures 18.8 and 18.9 show pressure head contours for the same times.

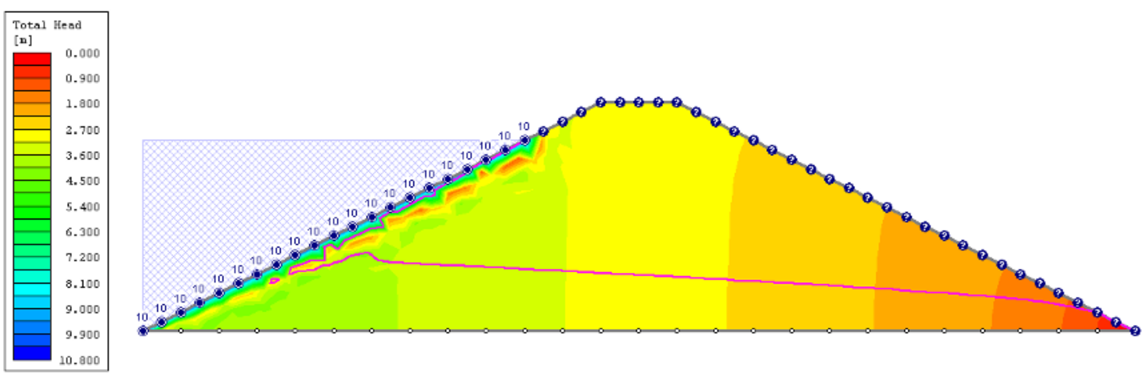


Figure 18.6: Total Head Contours at 0.6 h

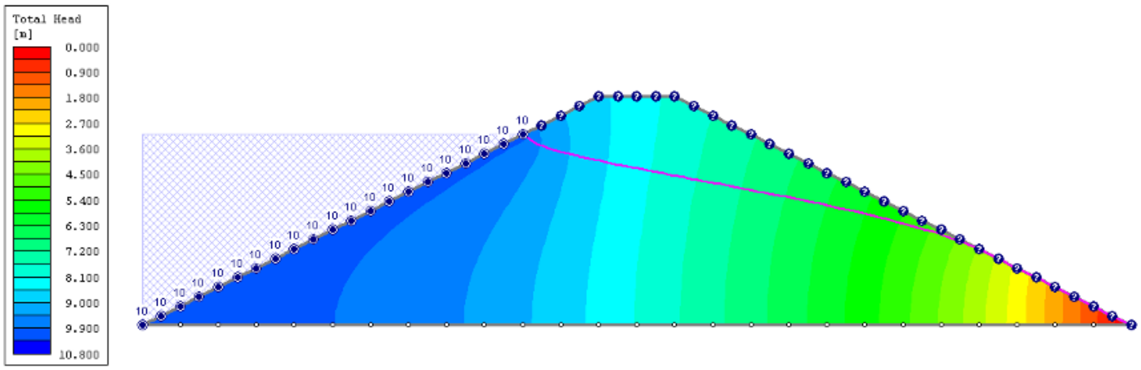


Figure 18.7: Total Head Contours at 19656 h

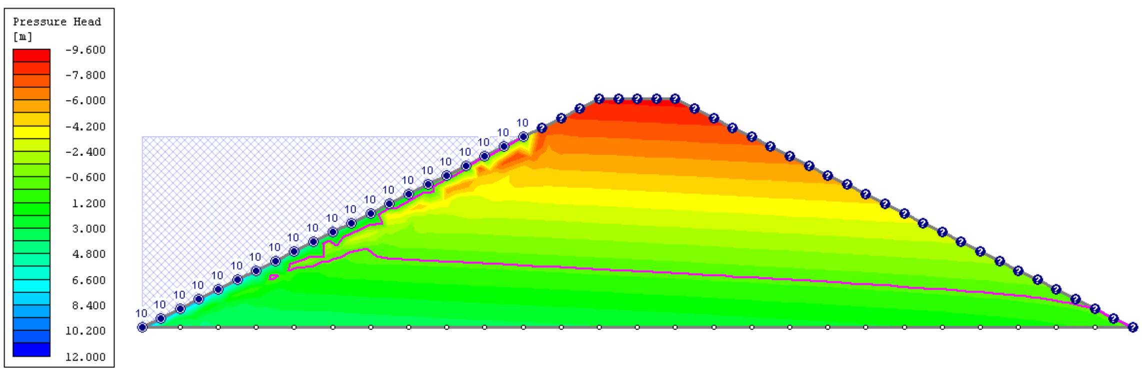


Figure 18.8: Pressure Head Contours at 0.6 h

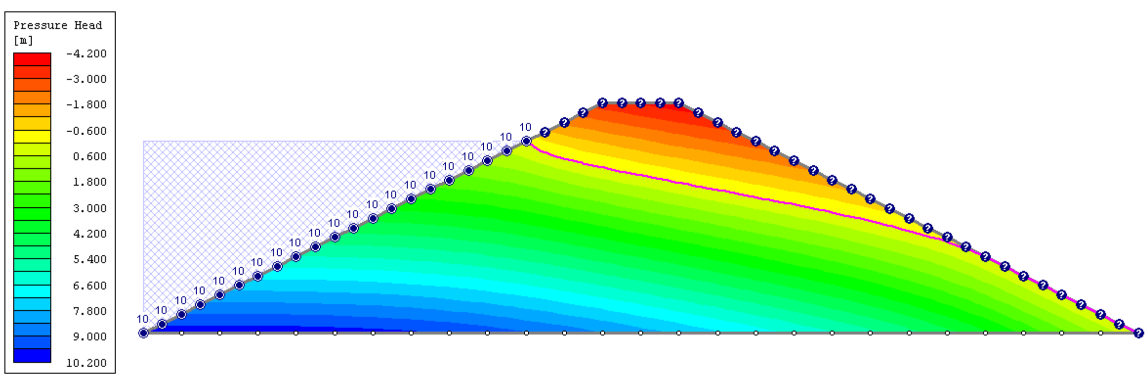


Figure 18.9: Pressure Head Contours at 19656 h

18.4. References

1. Fredlund, D.G. and Rahardjo, H. (1993), *Soil Mechanics for Unsaturated Soils*, New York: John Wiley & Sons.

19. Transient Seepage Below a Lagoon

19.1. Problem Description

This example deals with transient seepage below a lagoon. One half of the model geometry is considered since it is symmetrical. The section of the lagoon considered is 2 m wide. A 1 m deep soil liner is directly under the lagoon and the soil is assumed to extend 9 m below the soil liner before an impermeable boundary is encountered. An initial steady-state water table at a depth of 5 m from the ground surface is assumed. At analysis time zero, the water level in the lagoon is instantaneously raised to a height of 1 m. The model geometry for transient analysis at time zero is shown in Figure 19.1.

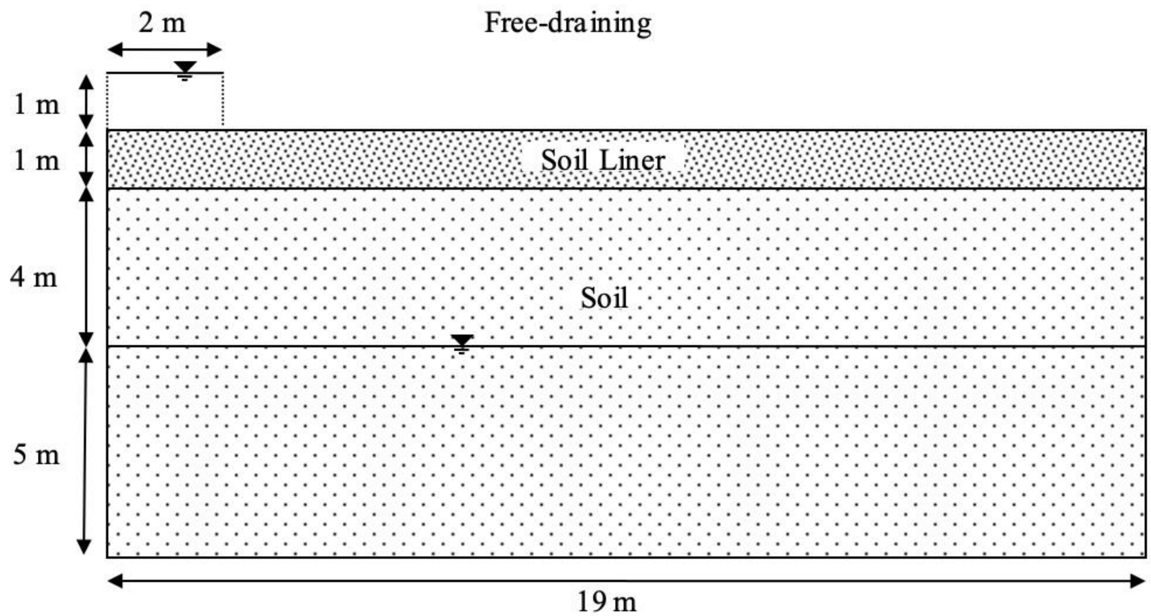


Figure 19.1: Model Geometry

An m_v value of 0.002 /kPa was assumed for both the soil and the liner. The permeability functions for the sands are shown in Figure 19.2.

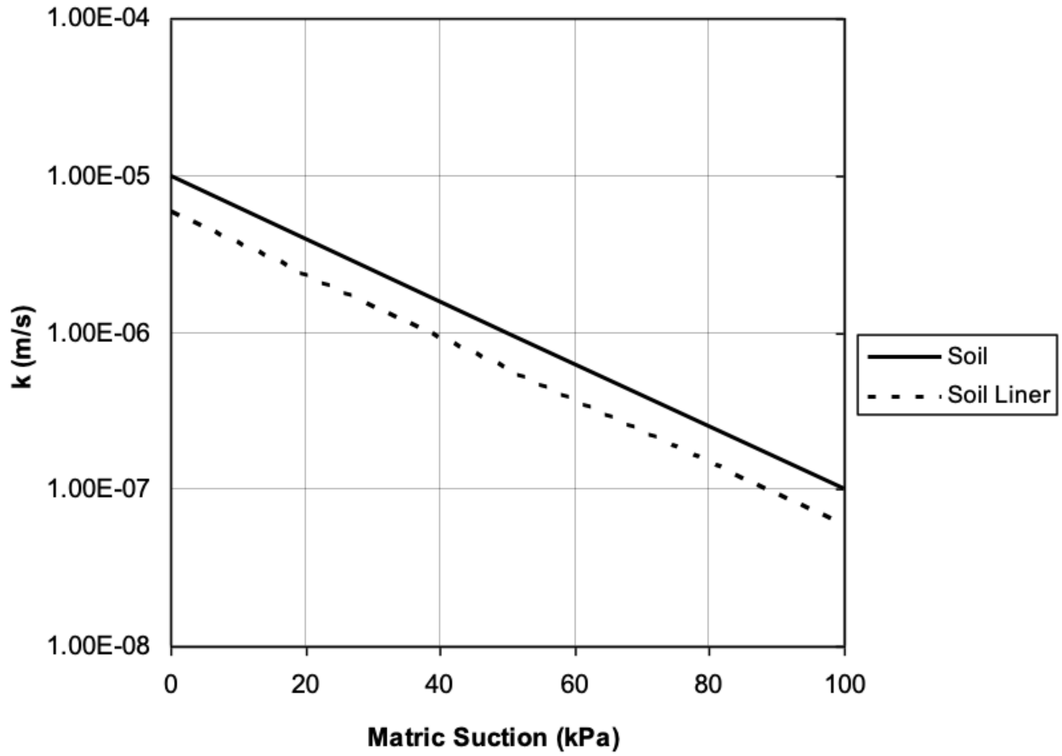


Figure 19.2: Coefficient of Permeability Functions

19.2. Slide Model

The **Slide** models for initial steady state and transient analysis are shown in Figures 19.3 and 19.4, respectively. The boundary conditions model the rise in water level in the lagoon. No flow is assumed across the lagoon centerline.

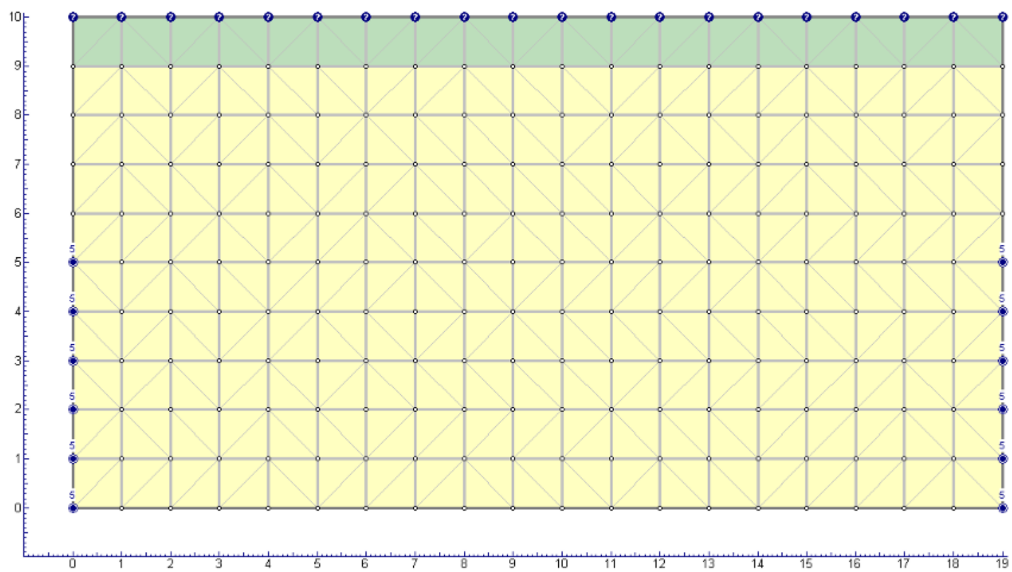


Figure 19.3: Slide Model – Initial Steady State

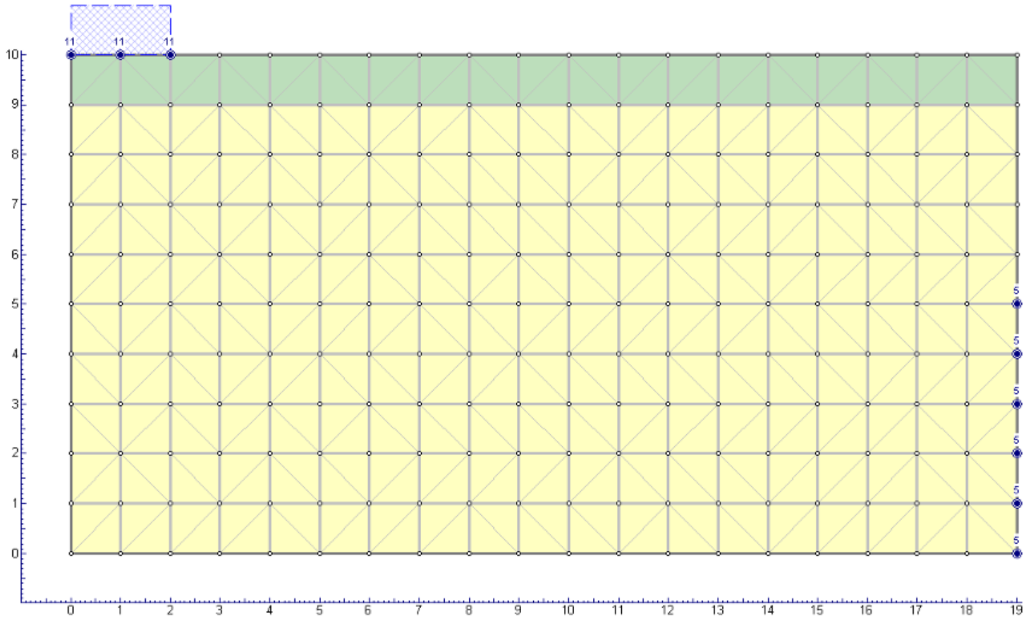


Figure 19.4: Slide Model – Transient

19.3. Results

Figures 19.4 to 19.7 show pressure head contours for different transient analysis times.

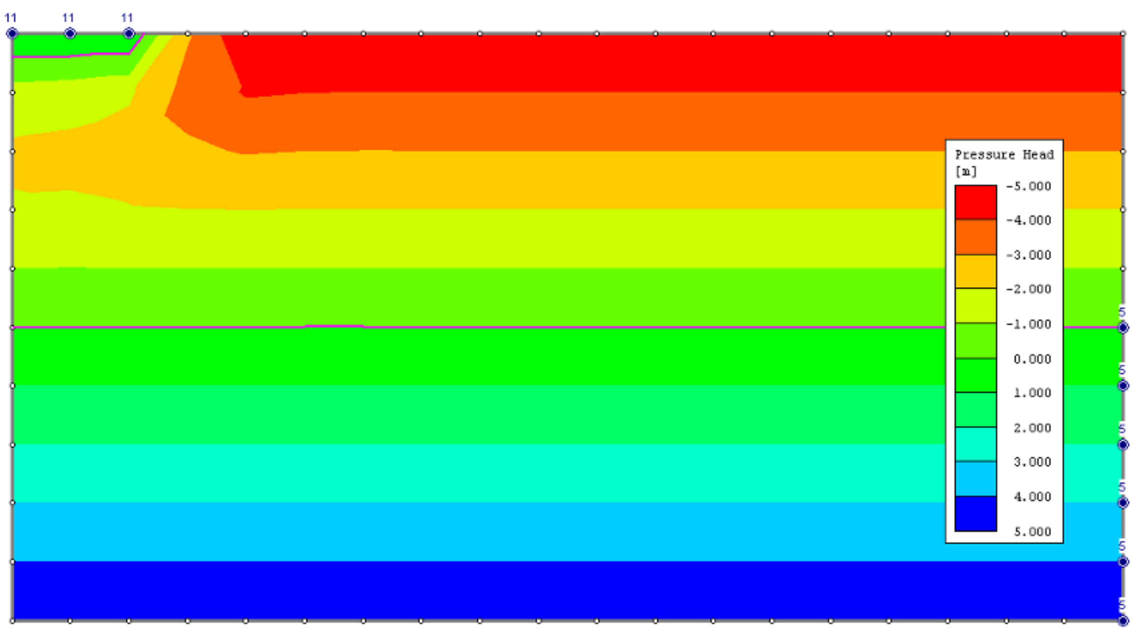


Figure 19.4: Pressure Head Contours at 73 minutes

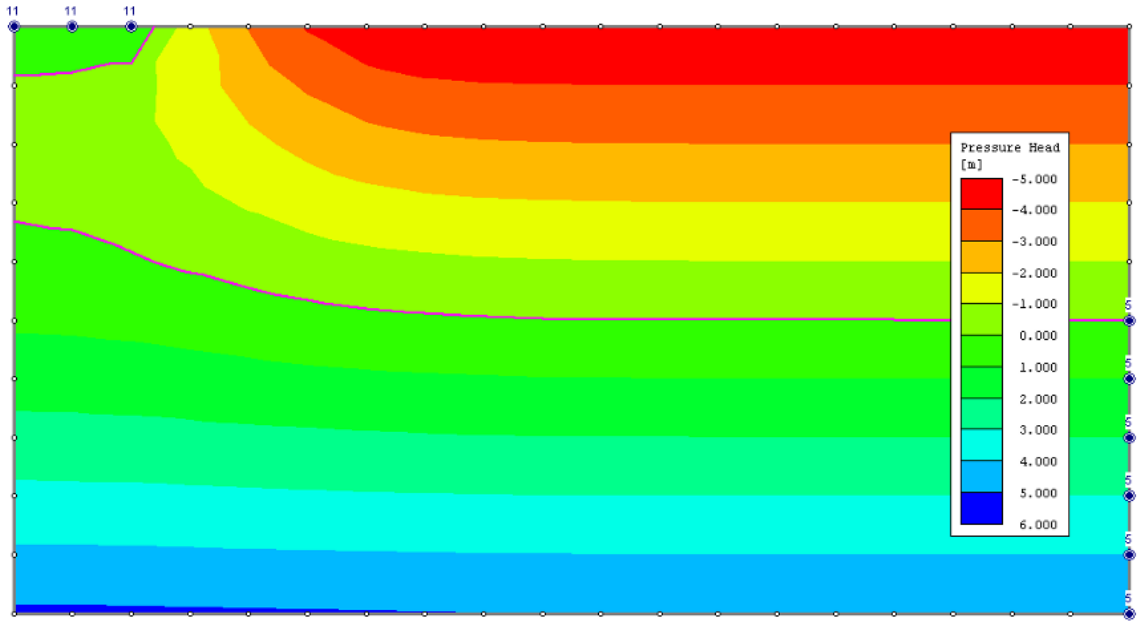


Figure 19.5: Pressure Head Contours at 416 minutes

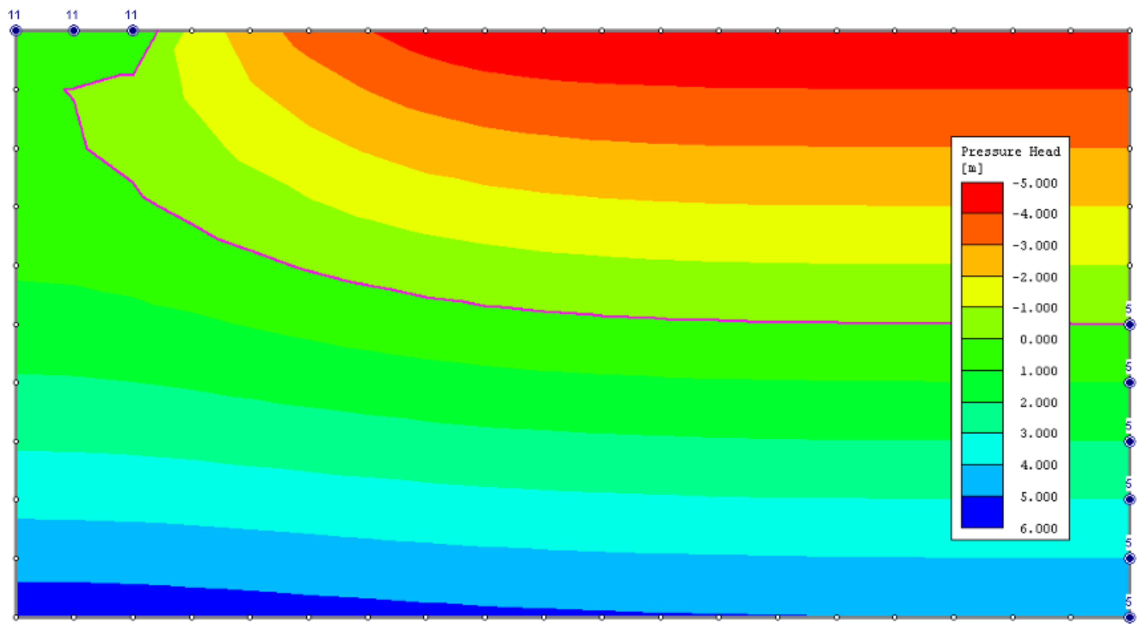


Figure 19.6: Pressure Head Contours at 792 minutes

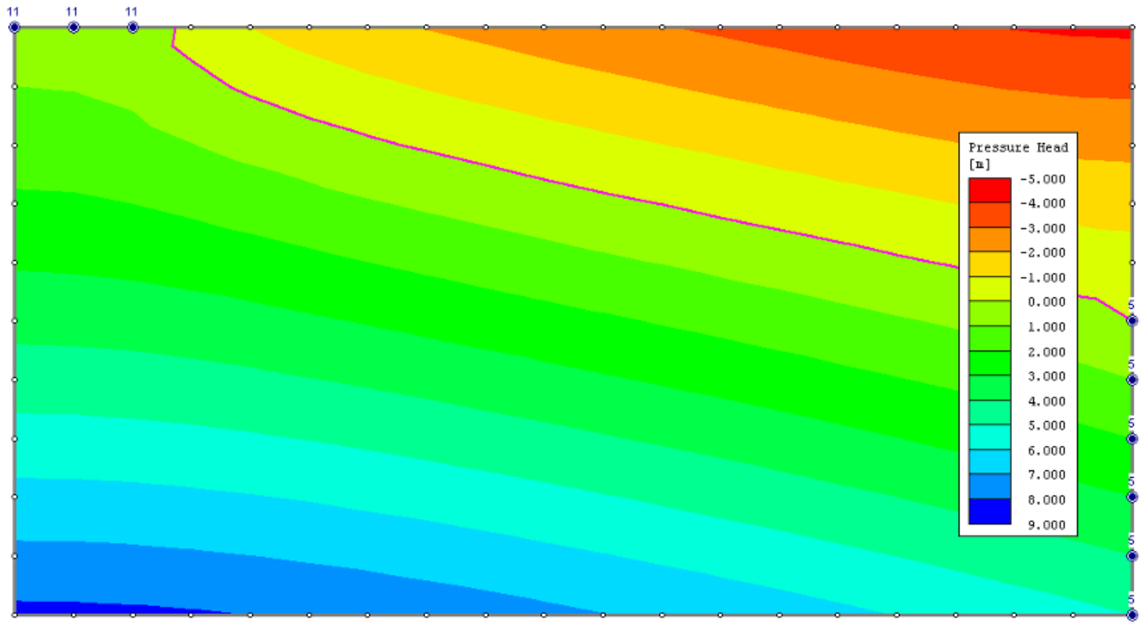


Figure 19.7: Pressure Head Contours at 11340 minutes

Pressure head values are sampled along the top boundary as shown in Figure 19.8. These values from **Slide** are plotted in comparison to values from Ref [1] in Figure 19.9.

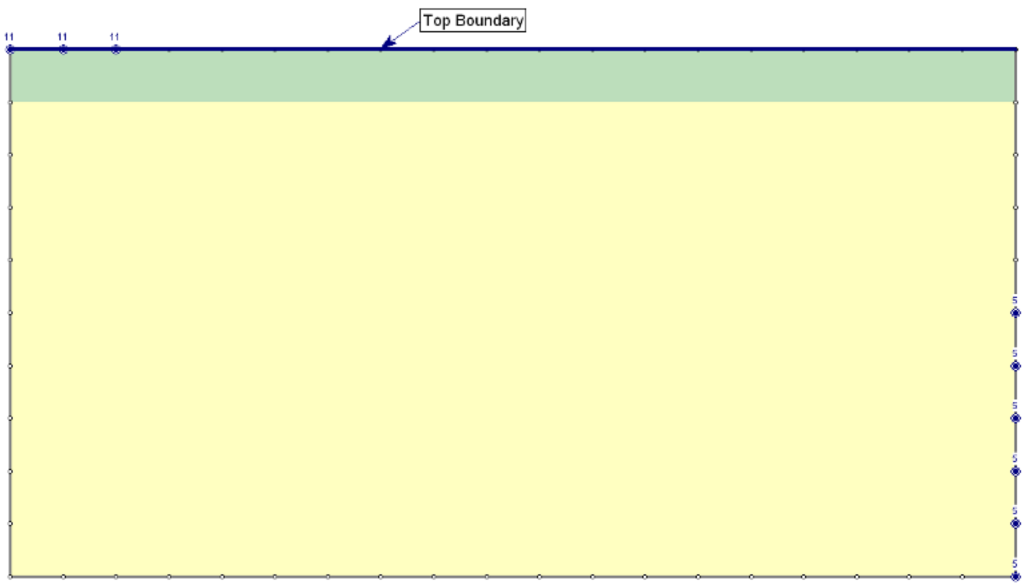


Figure 19.8: Query Lines

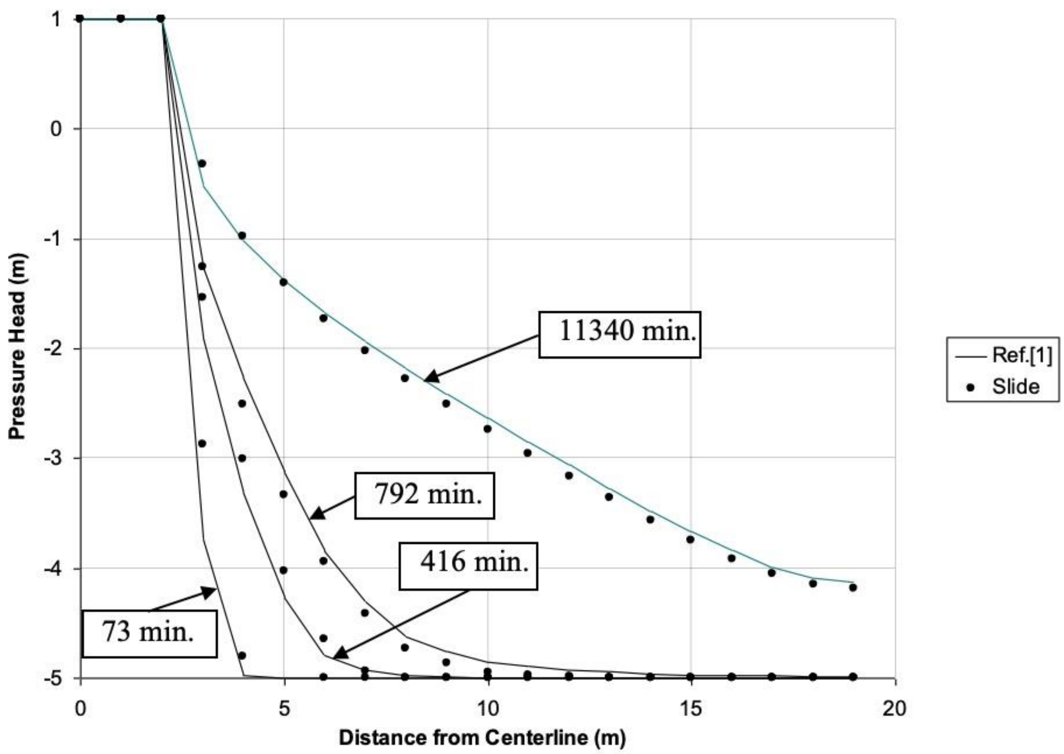


Figure 19.9: Comparison of Pressure Head Values along Top Boundary

19.4. References

1. Fredlund, D.G. and Rahardjo, H. (1993), *Soil Mechanics for Unsaturated Soils*, New York: John Wiley & Sons.

20. Transient Seepage in a Layered Slope

20.1. Problem Description

This problem deals with transient seepage in a layered slope. The slope consists of medium sand with a horizontal fine sand layer. At initial steady-state conditions, the water table is located at a height of 0.1 m from the toe of the slope. A constant infiltration of 2.1×10^{-4} m/s is applied at the top of the slope at time zero. An m_v value of 0.002 /kPa is assumed for both materials, and the permeability functions for the sands are shown in Figure 20.1.

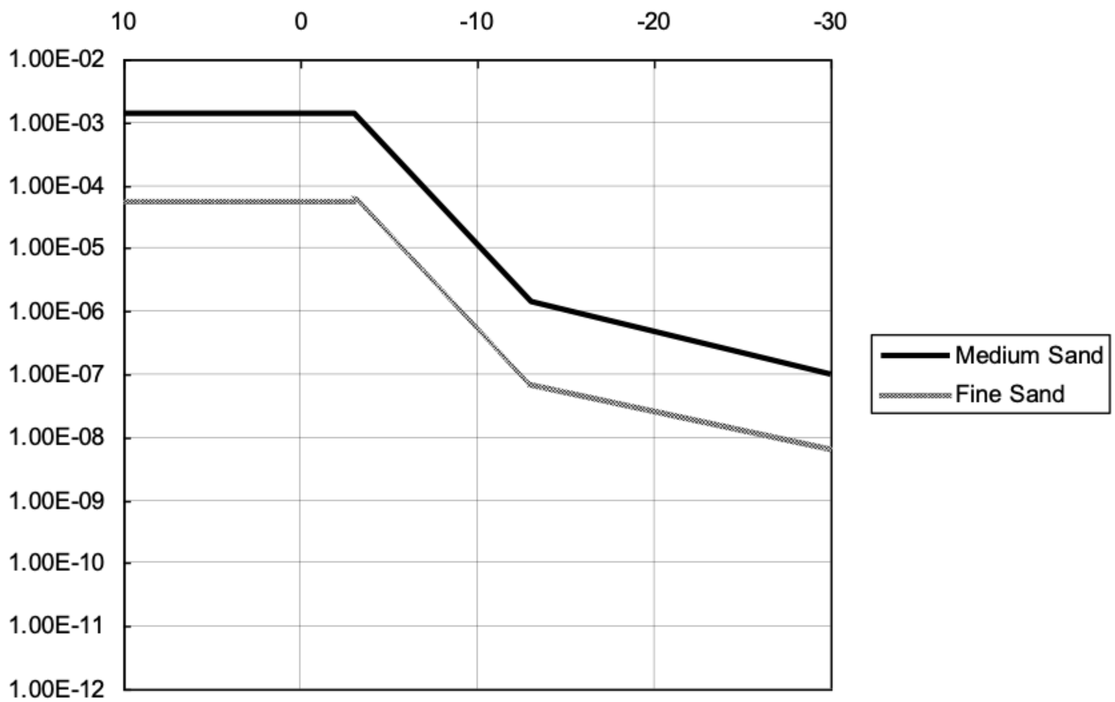


Figure 20.1: Coefficient of Permeability Functions

20.2. Slide Model

Figure 20.2 shows the **Slide** model used to perform transient analysis with constant infiltration.

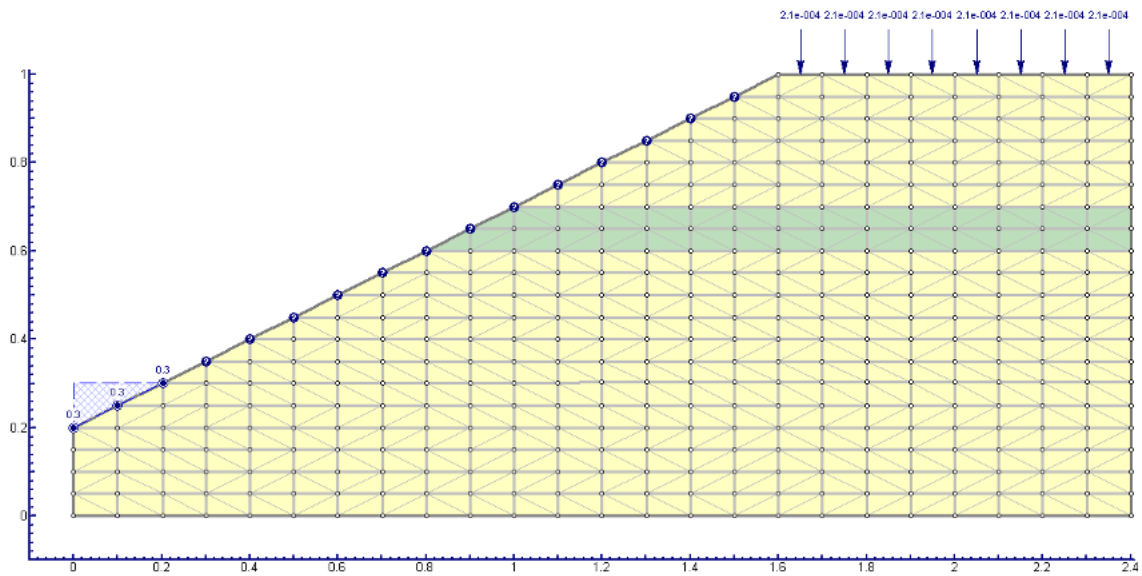


Figure 20.2: Slide Model

20.3. Results

Figures 20.3 to 20.5 show the total head contour results from **Slide**.

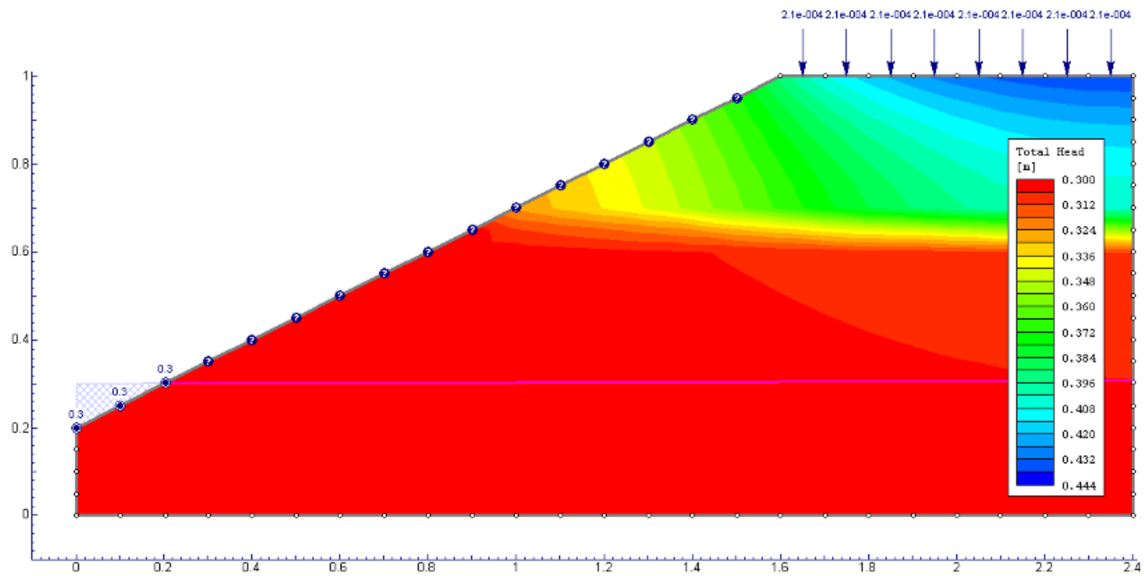


Figure 20.3: Total Head Contours for 4.6 seconds

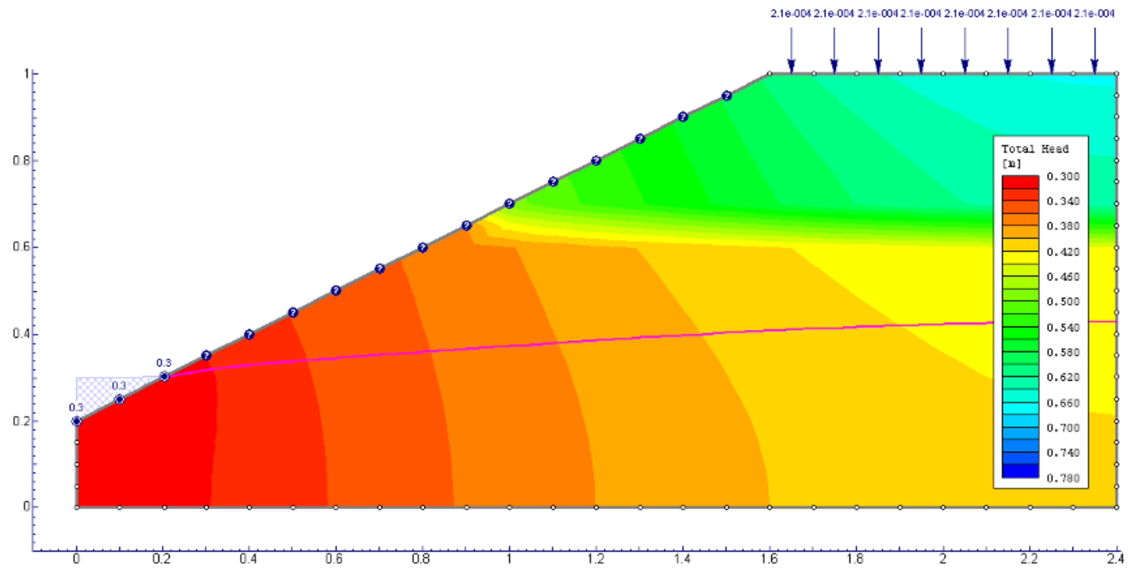


Figure 20.4: Total Head Contours for 31 seconds

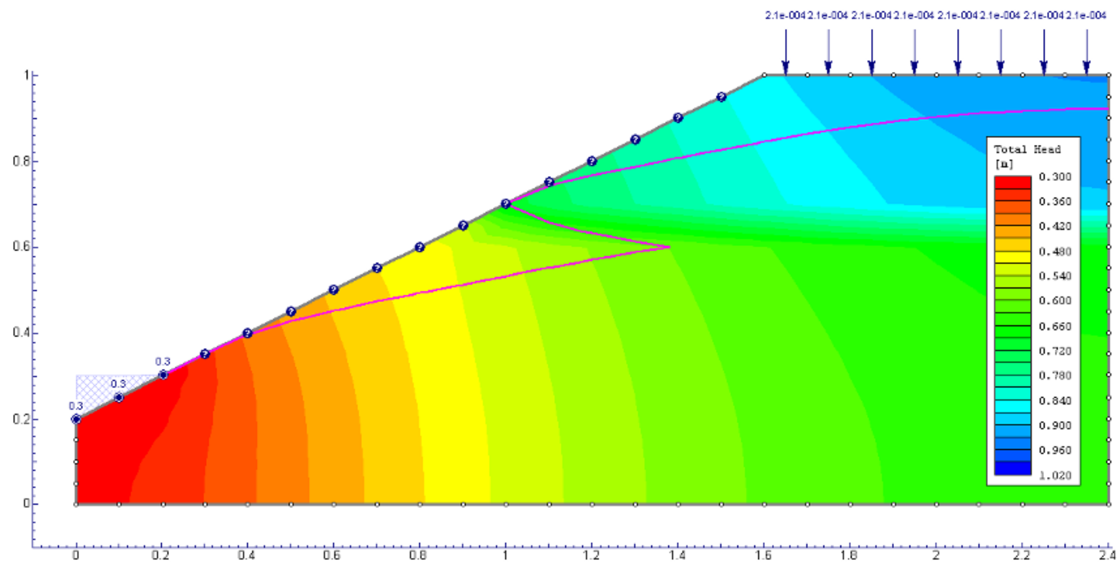


Figure 20.5: Total Head Contours for 208 seconds

Values of total head are taken along the query line shown in Figure 20.6. Figure 20.7 compares **Slide** results with those taken from Ref [1].

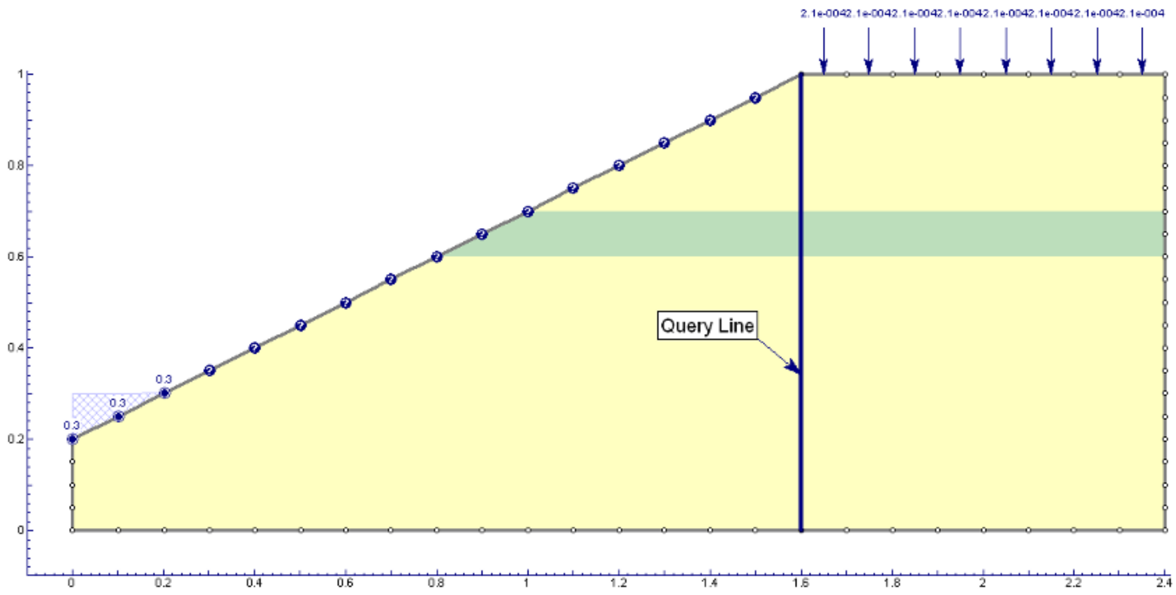


Figure 20.6: Query Line

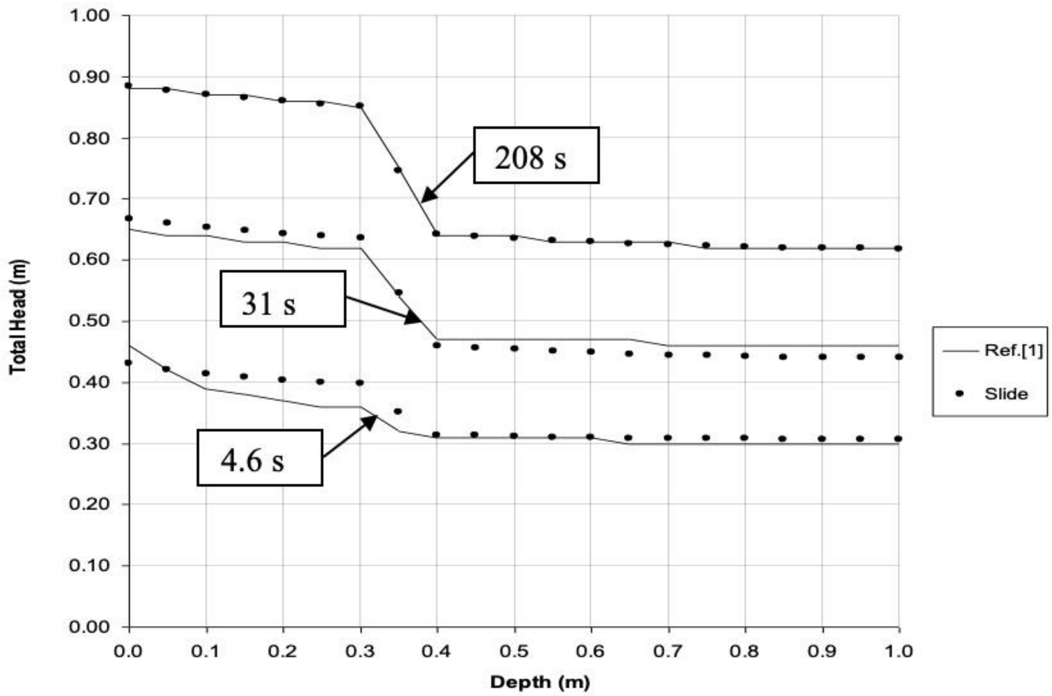


Figure 20.7: Comparison of Total Head Values

20.4. References

1. Fredlund, D.G. and Rahardjo, H. (1993), *Soil Mechanics for Unsaturated Soils*, New York: John Wiley & Sons.

21. Transient Seepage through a Fully Confined Aquifer

21.1. Problem Description

This problem deals with transient seepage through a fully confined aquifer. Two head conditions are examined. In both cases, the aquifer has an initial pore-water distribution that is changed through the introduction of five feet of hydraulic head to the left side of the aquifer. Seepage is then examined in the x-direction with time. The aquifer is 100 feet long and five feet thick. An illustration of the problem is presented in Figure 21.1.

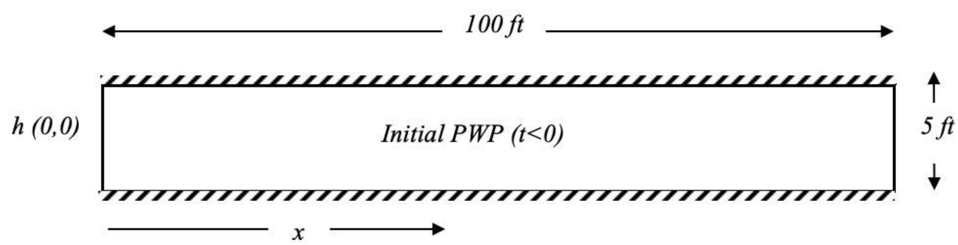


Figure 21.1: Model geometry

The soil has a hydraulic conductivity of 4 ft/hr and an m_v of 0.1. The hydraulic property is assumed to be fully saturated.

The equation for transient seepage through a fully confined aquifer can be expressed through the J.G. Ferris Formula [1] as:

$$h(x,t) = h(x,0) + \Delta H \cdot \mathrm{erfc}\left(\frac{x}{\sqrt{4t(T/S)}}\right)$$

$$T/S = \frac{k}{\gamma_w \cdot m_v}$$

Where *h(x,t)* is the hydraulic head at position *x* at time *t*; Δ*H* is the head difference between the initial pore-water distribution and the introduced hydraulic head; and *erfc* is the complimentary error function.

21.2. Slide Model

1. No initial pore-water distribution

Figure 21.2 shows the **Slide** model used to perform a transient analysis with 0 feet of initial pore-water pressure.

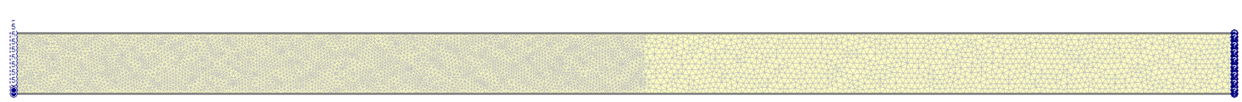


Figure 21.2: Slide Model – 0 feet of Initial PWP

2. Initial pore-water distribution of 5 feet

Figure 21.3 shows the **Slide** model used to perform a transient analysis with 5 feet of initial head (assigned by setting the steady state boundary condition of the problem to 5 feet of head). Note that the boundary condition on the left face is set to 10 feet (5 feet of initial PWP plus 5 feet of introduced hydraulic head).

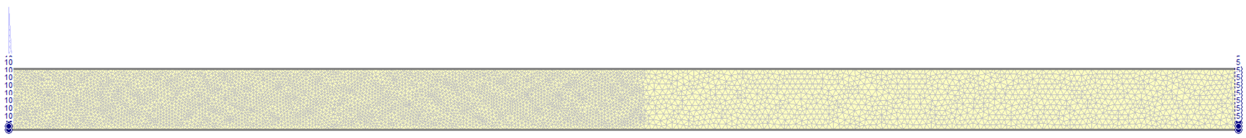


Figure 21.3: Slide Model – 5 feet of Initial PWP

21.3. Results

Figures 21.4 and 21.5 show the total head contour results from **Slide** at 600 hours.

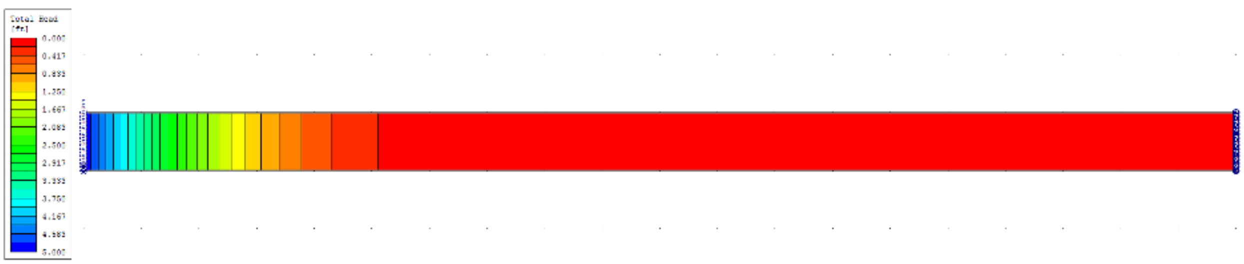


Figure 21.4: Total Head Contours, 600 hours, no initial PWP

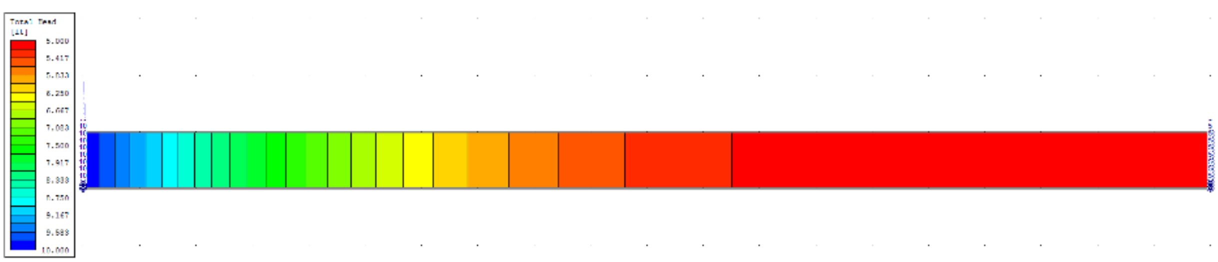


Figure 21.5: Total Head Contours, 600 hours, 5 feet of initial PWP

A comparison of the **Slide** results and the analytical solution for Case 1 is presented in Figure 21.6. A comparison of the Slide results and the analytical solution for Case 2 is presented in Figure 21.7.

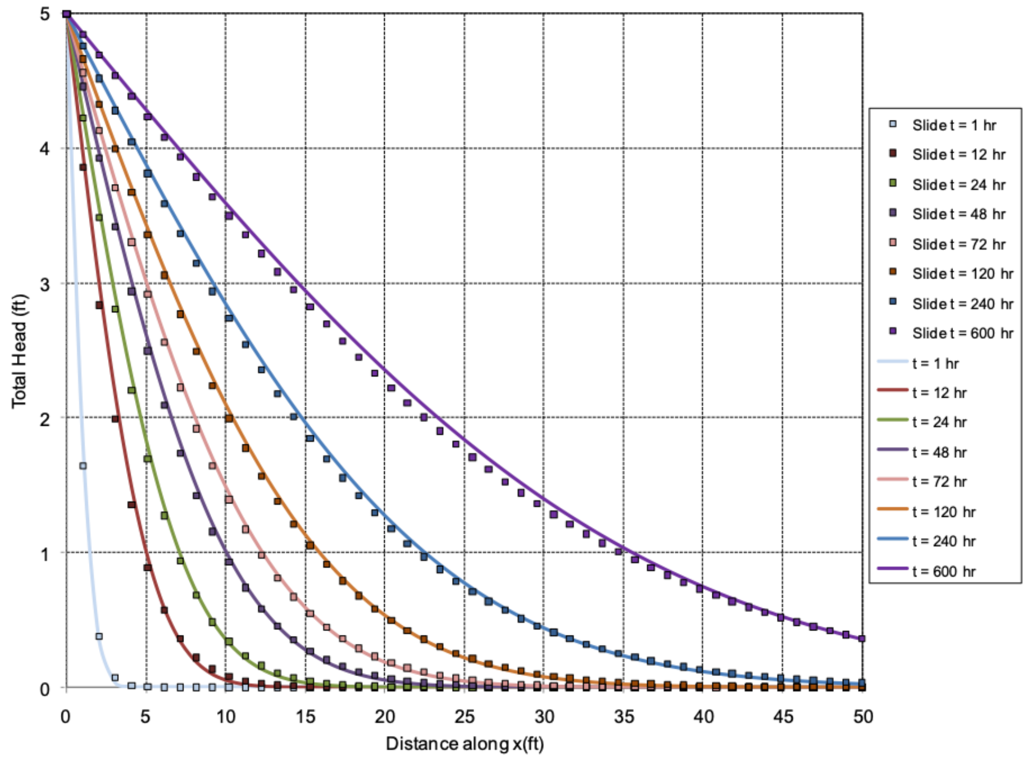


Figure 21.6: Comparison of Slide results and Analytical Solution – Case 1

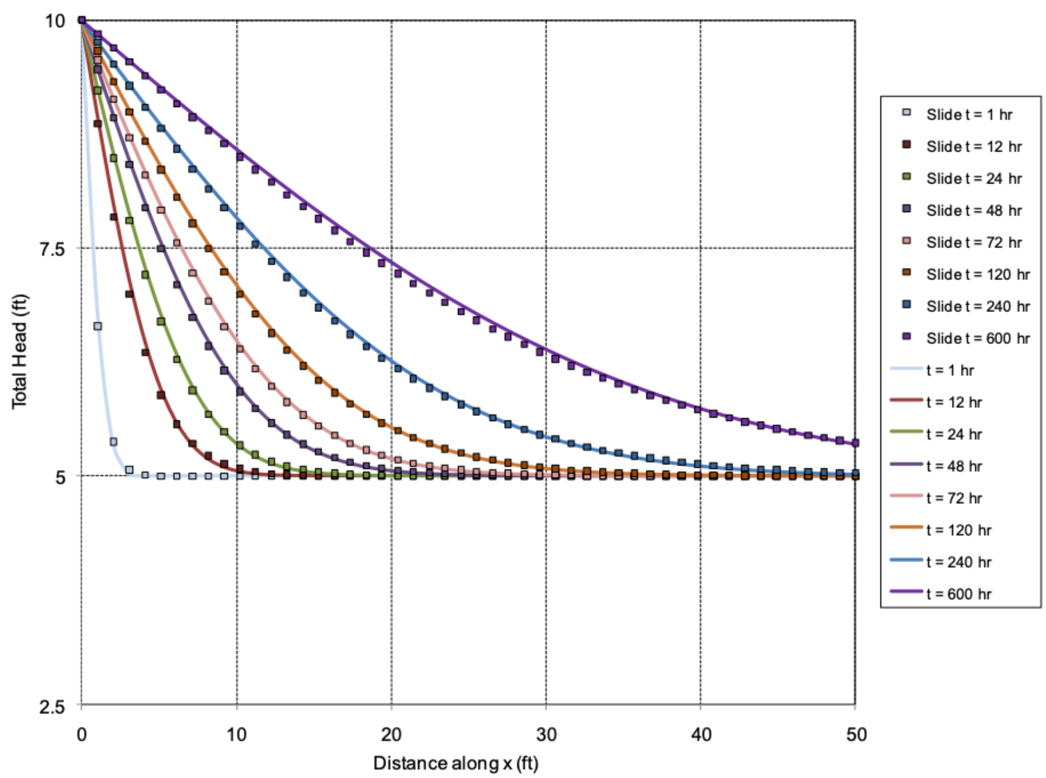


Figure 21.6: Comparison of Slide results and Analytical Solution – Case 2

21.4. References

1. Tao, Y. and Xi, D. (2006), “Rule of Transient Phreatic Flow Subjected to Vertical and Horizontal Seepage:” Applied Mathematics and Mechanics. (27), 59-65.

Note: See files *Groundwater#21_1.slim* and *Groundwater#21_2.slim*.