ROCSCIENCE

Settle3

Consolidation Analysis

Verification Manual

Table of Contents

1. Consolidation Analysis of Simple Layered Materials

1.1. Problem description

One-dimensional consolidation of stratified soils is verified in this example. Two soils were being considered in this verification. The properties of Soil A and Soil B are listed in Table 1.1.

Both the pore fluid (γ_w) and the thickness of the whole soil profile are assumed to be one unit magnitude. A uniform loading (q) of one unit magnitude is applied on top of the soil. Three different soil configurations were considered.

- Case 1: Uniform Soil
- Case 2: Soil A / Soil B (Soil A is on top of Soil B)
- Case 3: Soil B / Soil A

The model geometries and the boundary conditions of all three different soil profiles are shown in Figure 1.1.

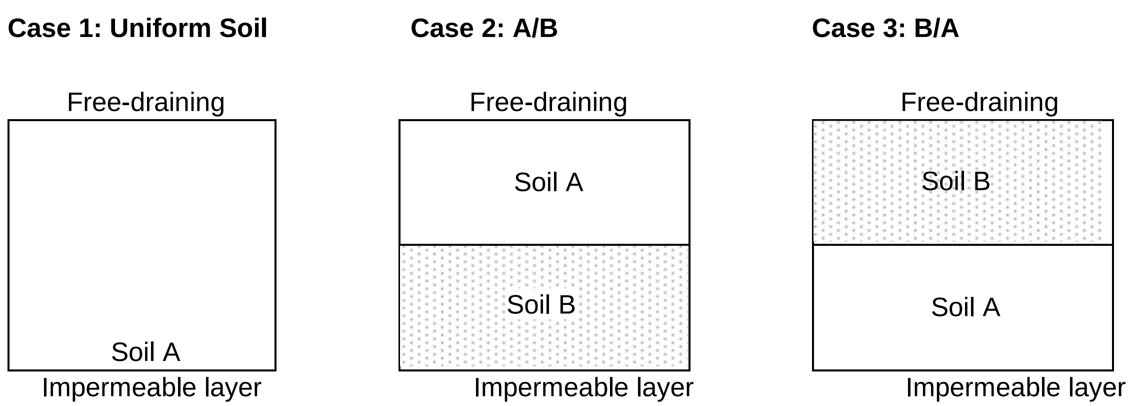


Figure 1.1 – Model Geometry

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

Table 1.1 – Soil Properties

1.2. Results and Discussion

Figures 1.2 to 1.4 show the distribution of excess pore pressures over time in the three cases. Figure 1.5 shows the consolidation profiles of the three different soil configurations. The results are compared to analytical solution presented by Pyrah [1] and are in good agreement.

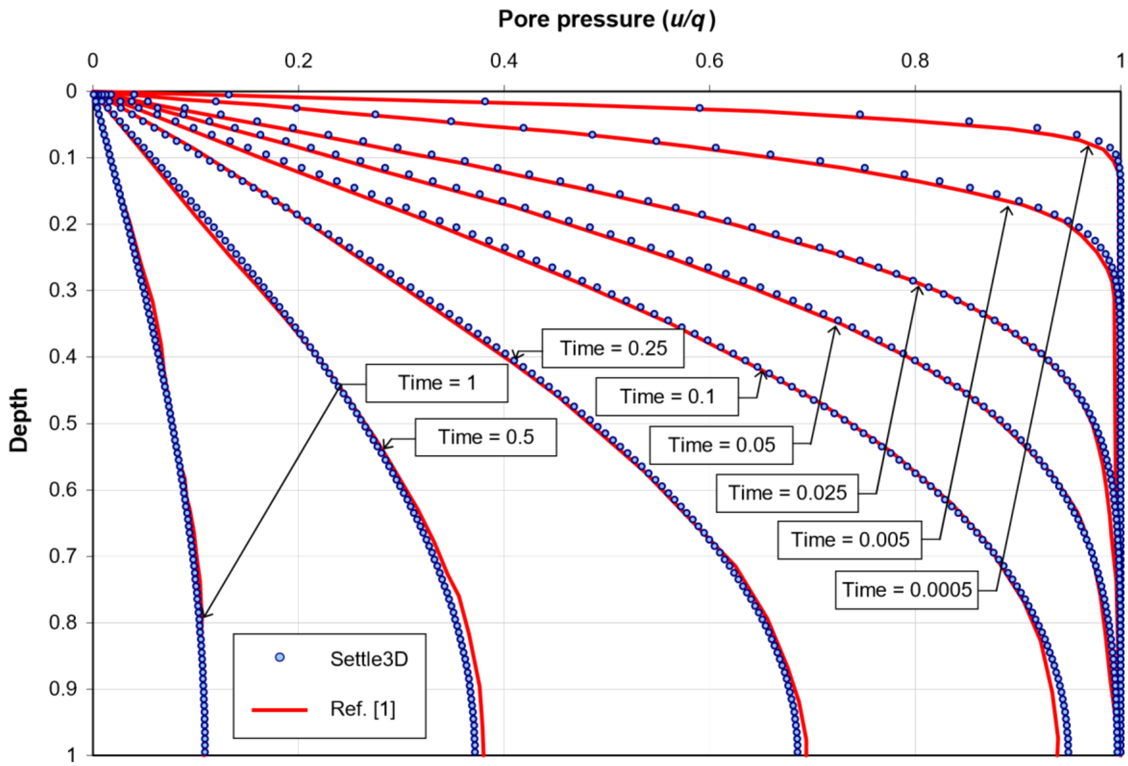


Fig. 1.2 Excess Pore Pressures Distribution with Depth over Time for Case 1

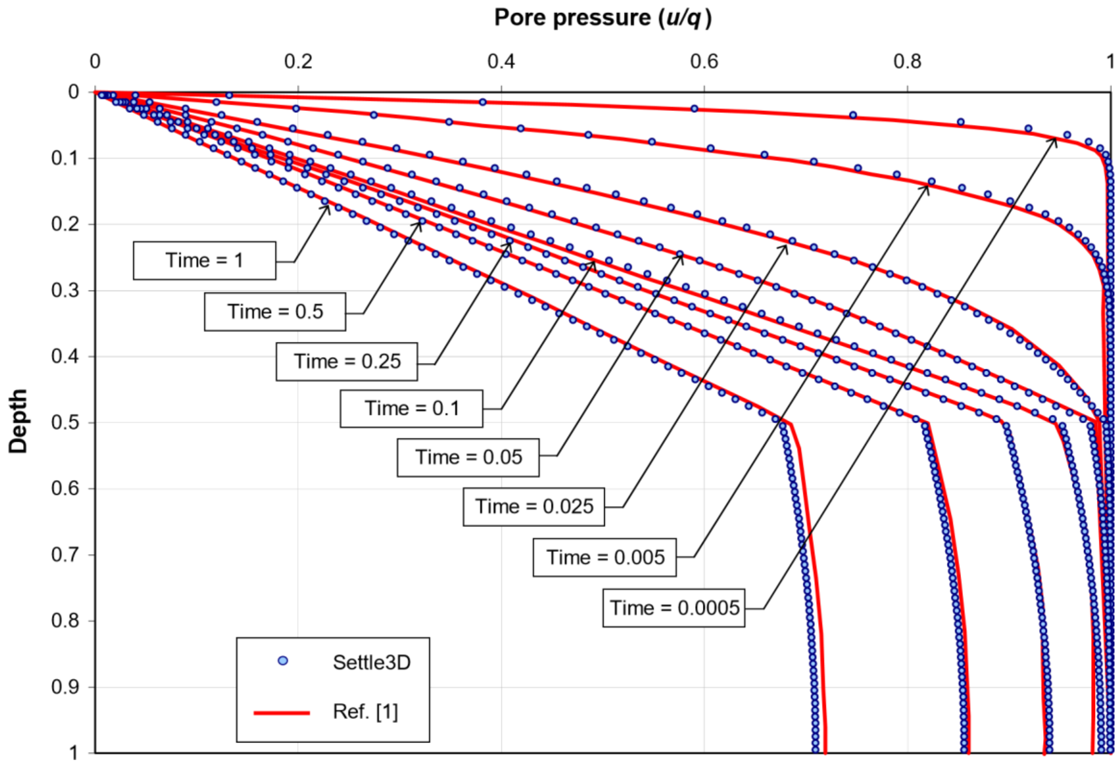


Fig. 1.3 Excess Pore Pressures Distribution with Depth over Time for Case 2

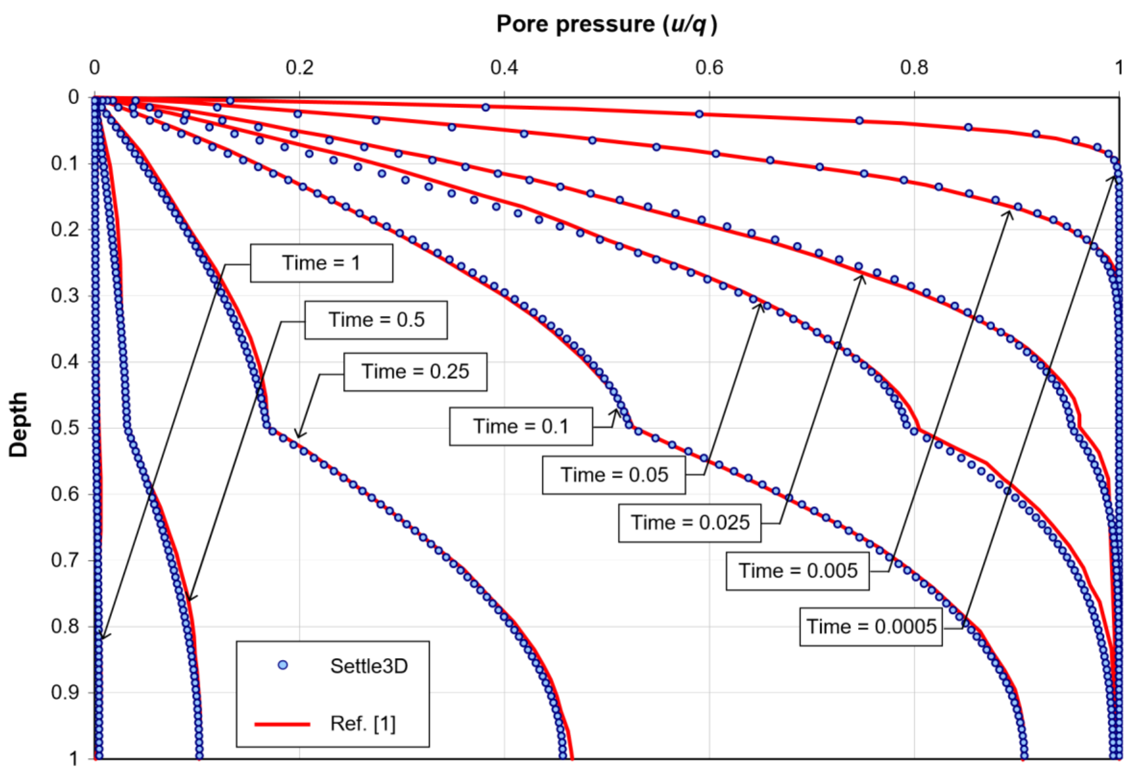


Fig. 1.4 Excess Pore Pressures Distribution with Depth over Time for Case 3

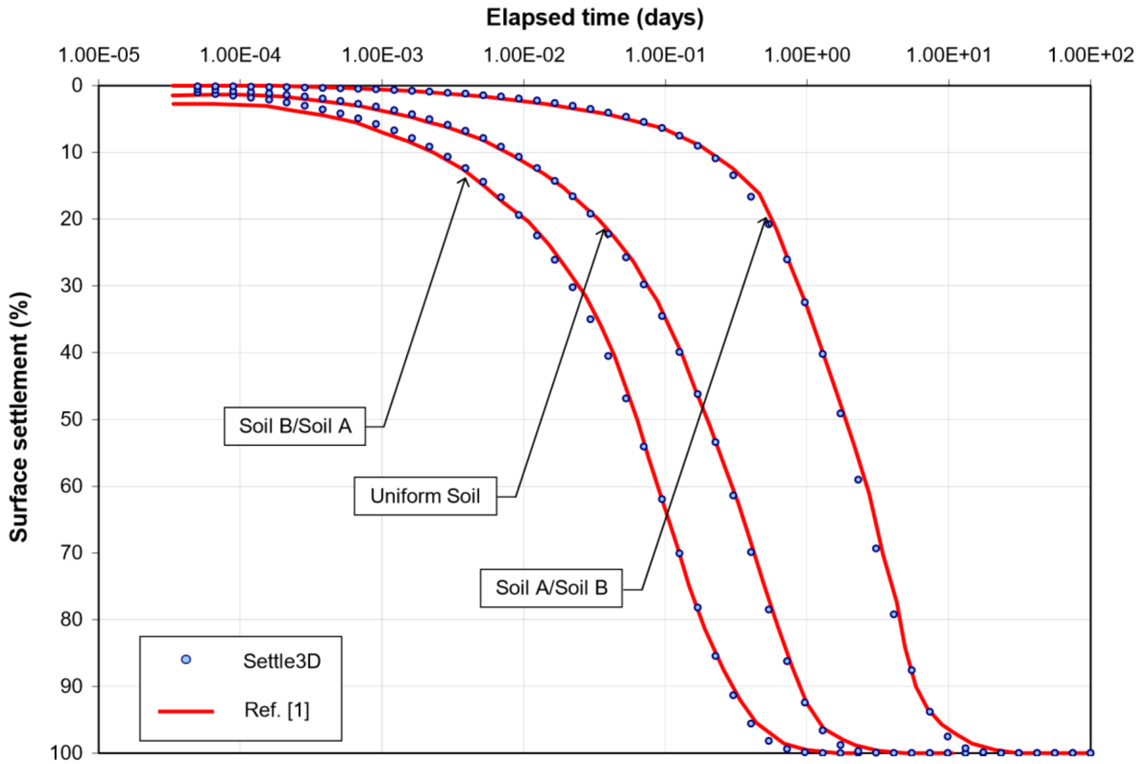


Fig. 1.5 Consolidation Profiles

1.3. References

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

2. Consolidation Settlement Predictions

2.1. Problem description

Two cases were considered in this example. They are taken from Example 11.5 and Example 12.3 of *Geotechnical engineering: principles and practices* by Coduto (1999). For both cases, the soil profile consists of two different materials with a layer of fill placed on top of them. Ultimate settlement due to consolidation is verified in this example. The model geometry of the problems is shown in Figure 2.1 and 2.2.

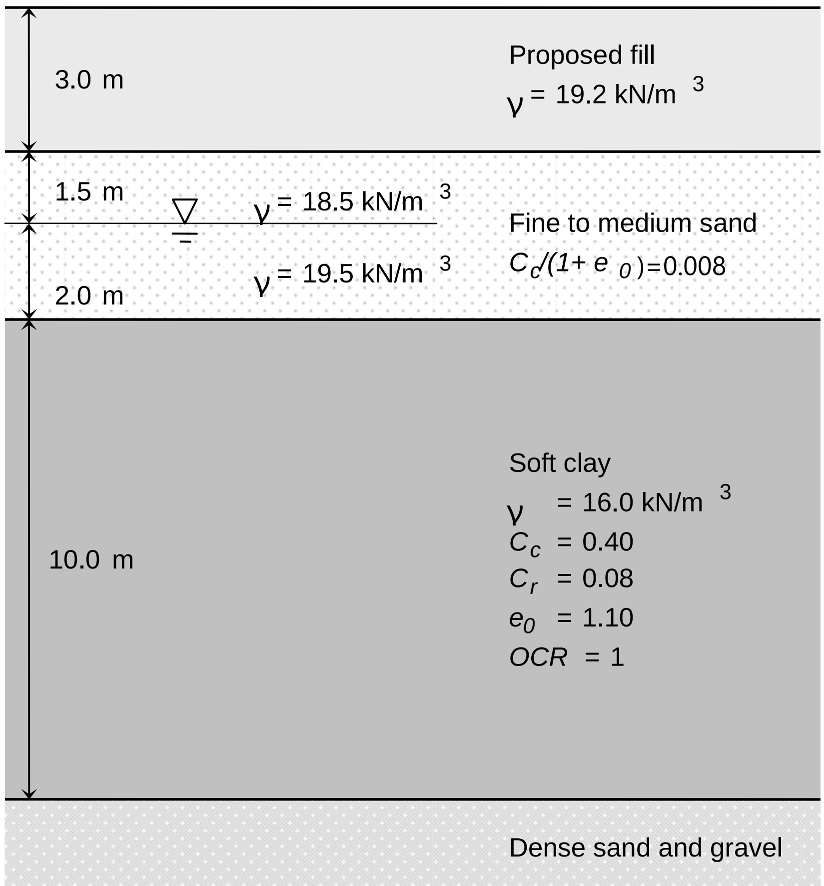


Figure 2.1 – Model Geometry of Case 1

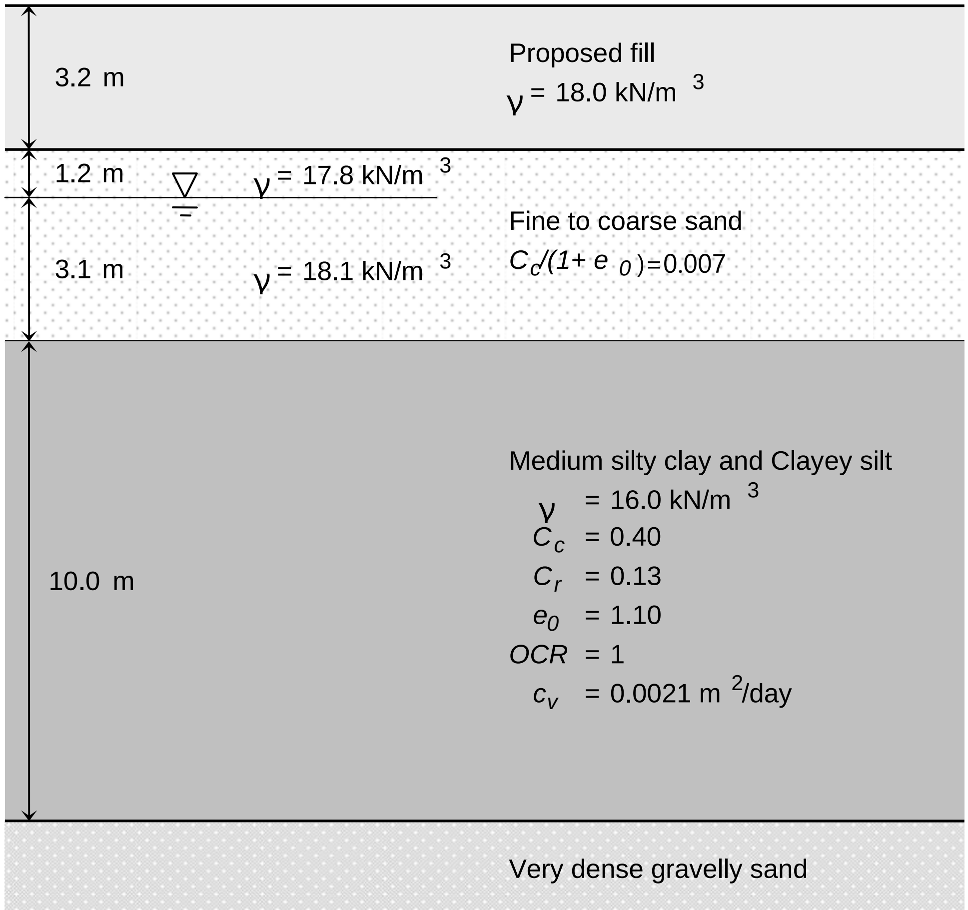


Figure 2.2 – Model Geometry of Case 2

2.2. Results and Discussion

Figure 2.3 and 2.4 show the consolidation profile given by ***Settle3***. The ultimate settlements of both cases are compared to those from [1], i.e.:

- Case 1: 492 mm (***Settle3***) compared to 483 mm [1]
- Case 2: 490 mm (***Settle3***) compared to 485 mm [1] The results are in good agreement.

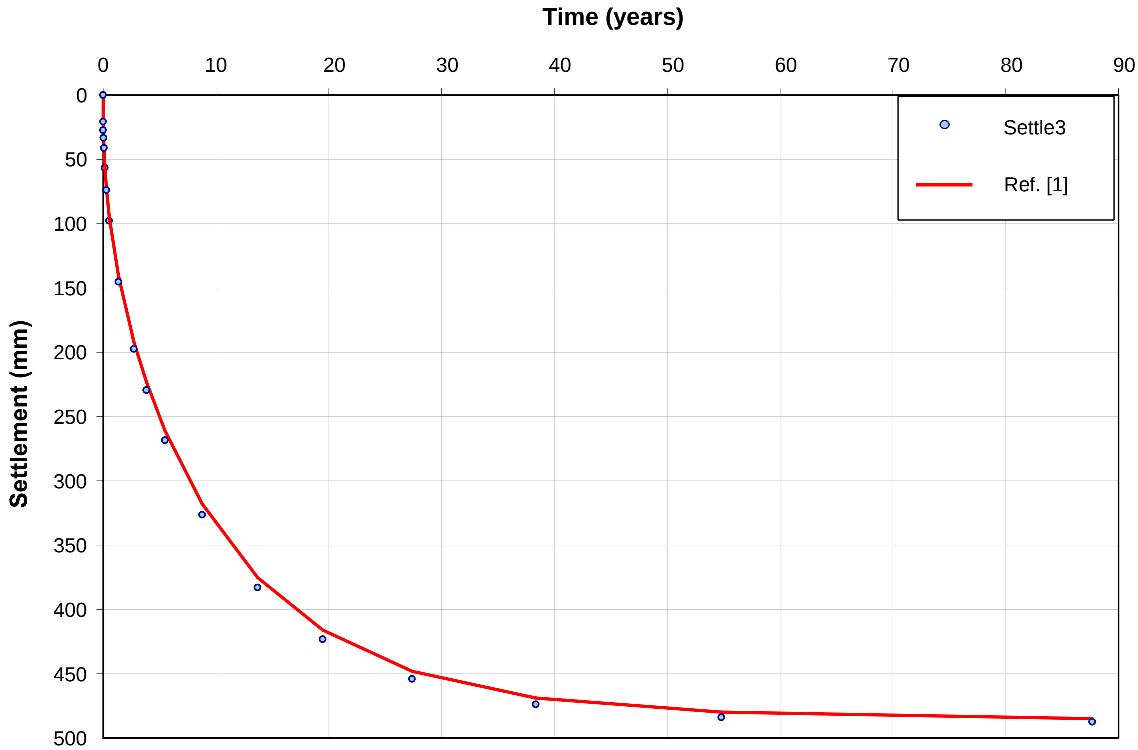


Fig. 2.3 Settlement Prediction of Case 2

2.3. References

1. D. P. Coduto (1999), *Geotechcnial engineering: principles and practices*, New Jersey: Prentice-Hall.

3. Effects of Fill to Consolidation in Lagunillas, Venezuela Case

3.1. Problem description

This verification example is taken from Example 27.2 of *Soil Mechanics SI Versioin* by Lambe and Whitman (1969). The problem is derived from a real case [1]. A two-layered material, a silt layer on top of a clay layer, is the object of this example. The ground surface is positioned at an elevation of -2.0 m. A 4.5 m fill is placed on top of the strata. The model geometry of the problem and its properties are shown in Figure 3.1.

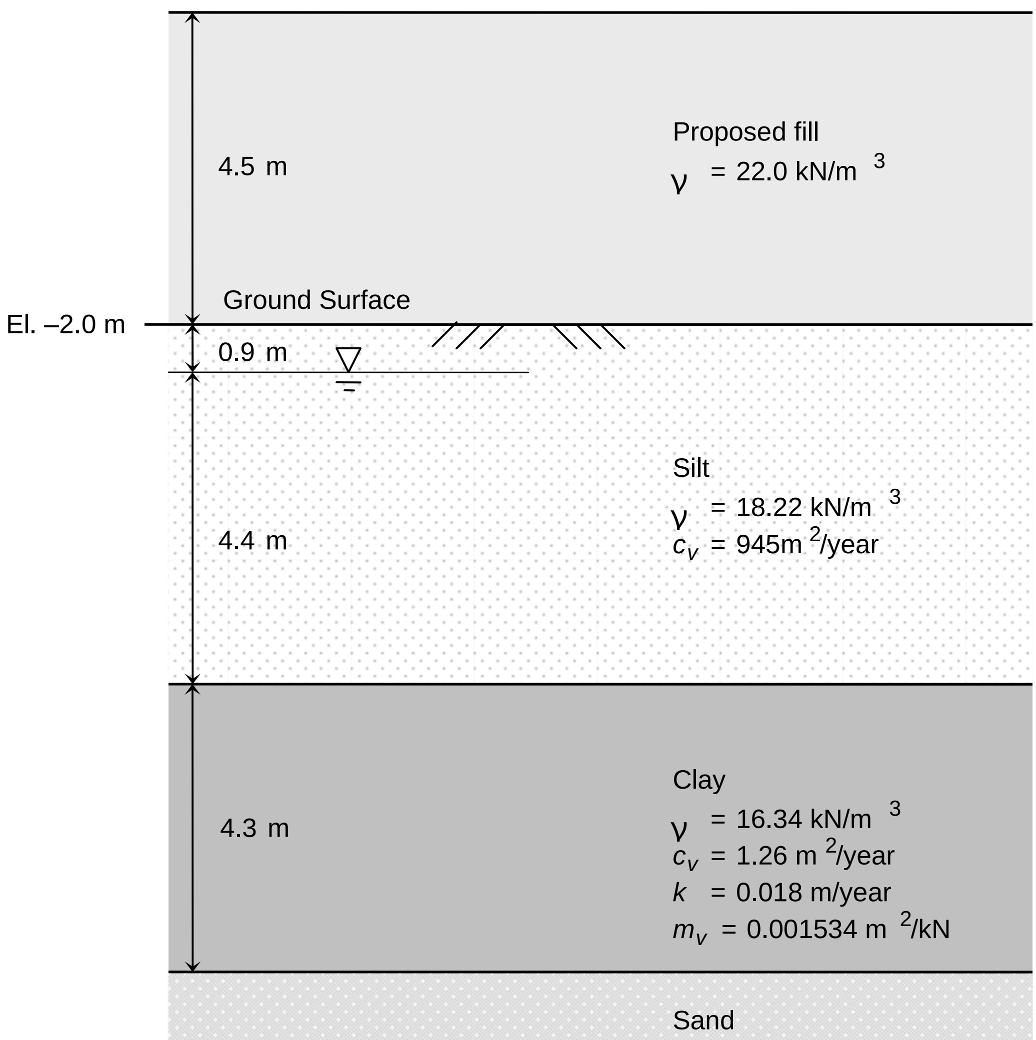


Figure 3.1 – Model Geometry

3.2. Results and Discussion

Figure 3.2 shows the consolidation profile estimated by ***Settle3*** compared to that from [1].

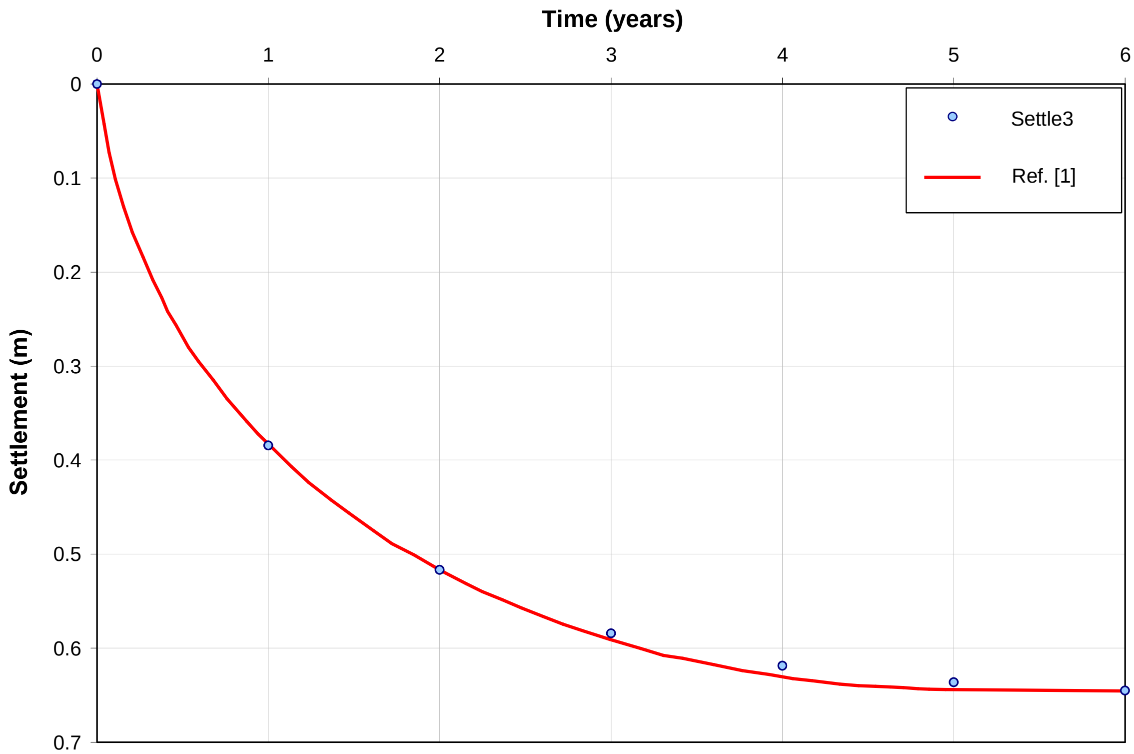


Fig. 3.2 Consolidation Profile

3.3. References

1. T. W. Lambe and R. V. Whitman (1969), *Soil Mechanics SI Version*, New York: John Wiley & Sons.

4. Linear Consolidation of Multi-Layered Soils

4.1. Problem description

A four-layered soil is considered in this example. The model is based on papers by Lee et al. (1992) and Chen et al. (2005). A uniform loading (*q*) of one unit magnitude is applied on top of the soil strata. Single drainage and double drainage were considered. For the single drainage case, drainage is only allowed to occur at the top end. As for the double drainage, drainage is allowed to occur at both top and bottom ends. The model geometry of the problem and its properties are shown in Figure 4.1.

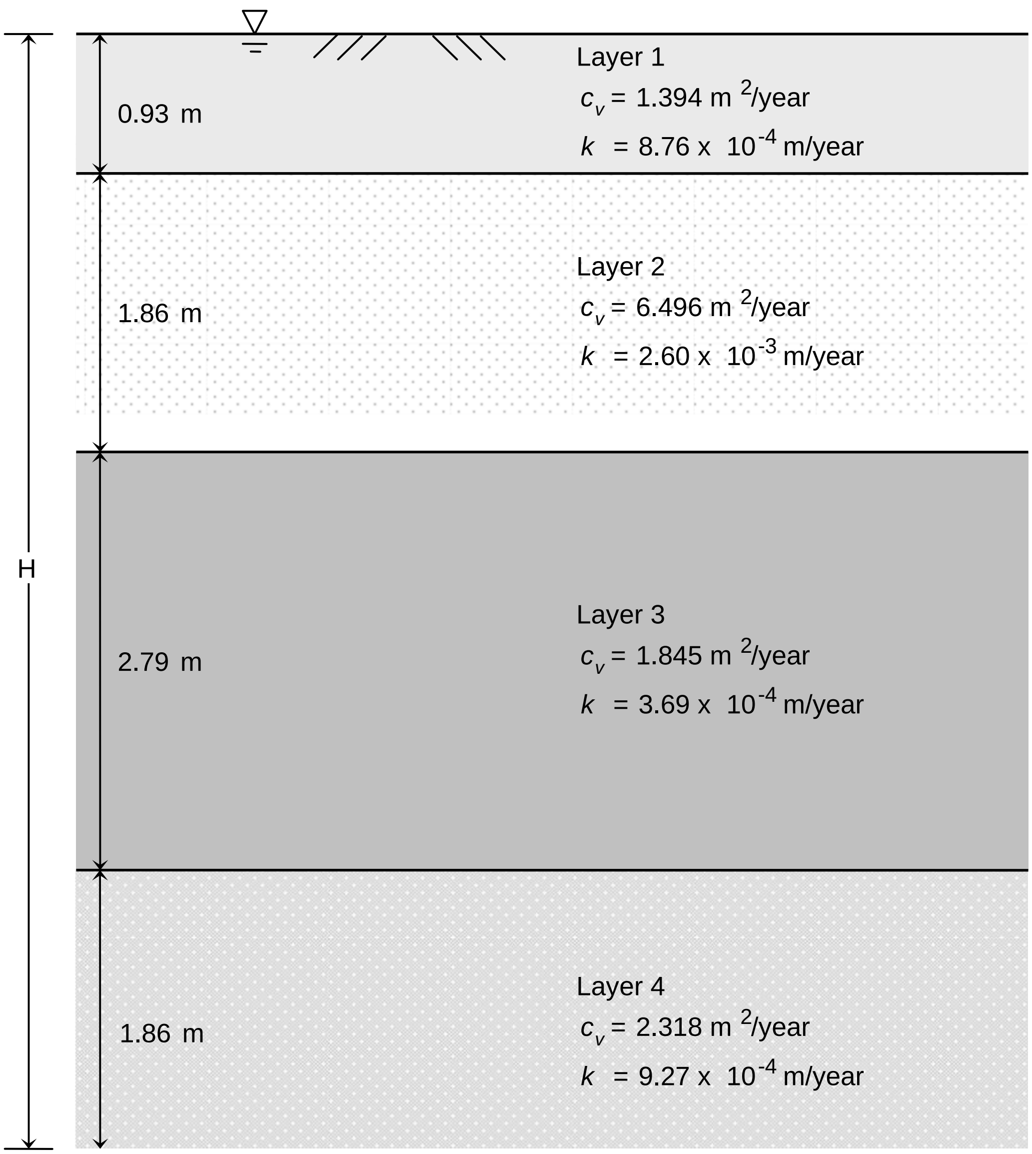


Figure 4.1 – Model Geometry

4.2. Results and Discussion

m_v Calculation

$$m_v = \frac{k}{c_v \gamma_w}$$

4.2.1. Single Drainage case:

Figure 4.2 shows the excess pore pressure (*u*) distribution with depth (*z*) for different time factors (T_v) calculated by ***Settle3*** compared to those presented in Lee et al. [2]. Time factor (T_v) is given by:

$$T_v = \frac{c_v t}{H^2}$$

Figure 4.3 shows the settlement prediction due to consolidation given by ***Settle3*** compared to that in Lee et al. [2].

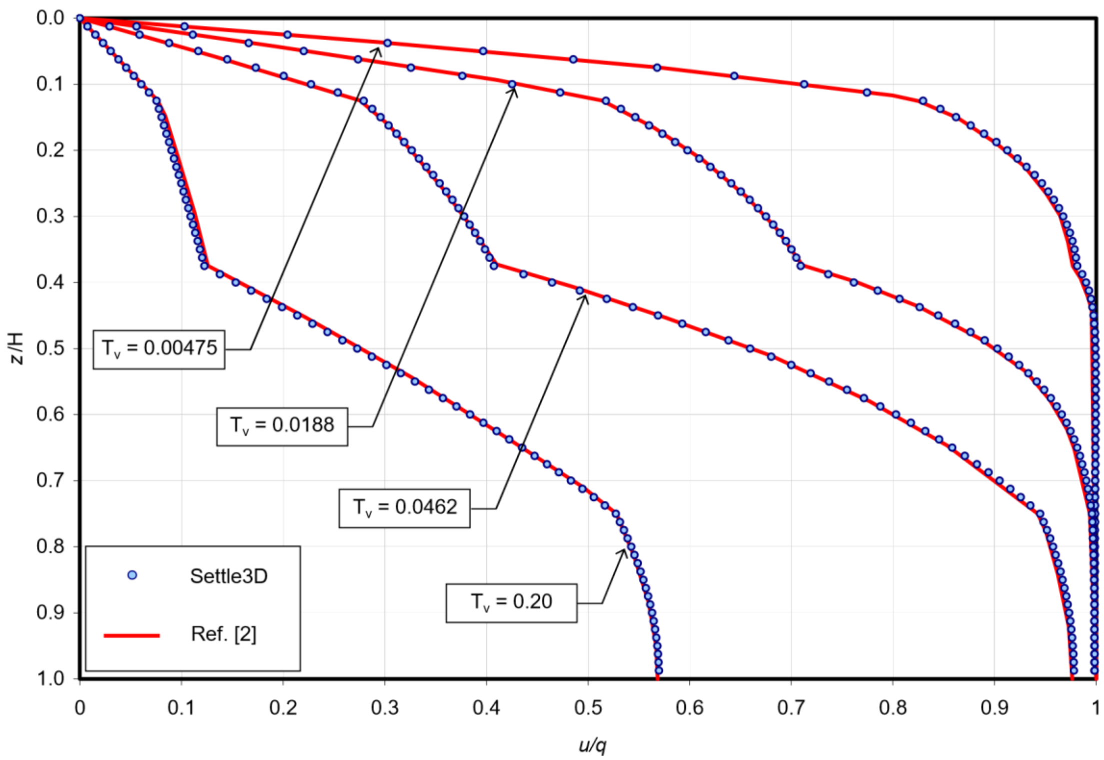


Fig. 4.2 Excess Pore Pressures Distribution with Depth (Single Drainage case)

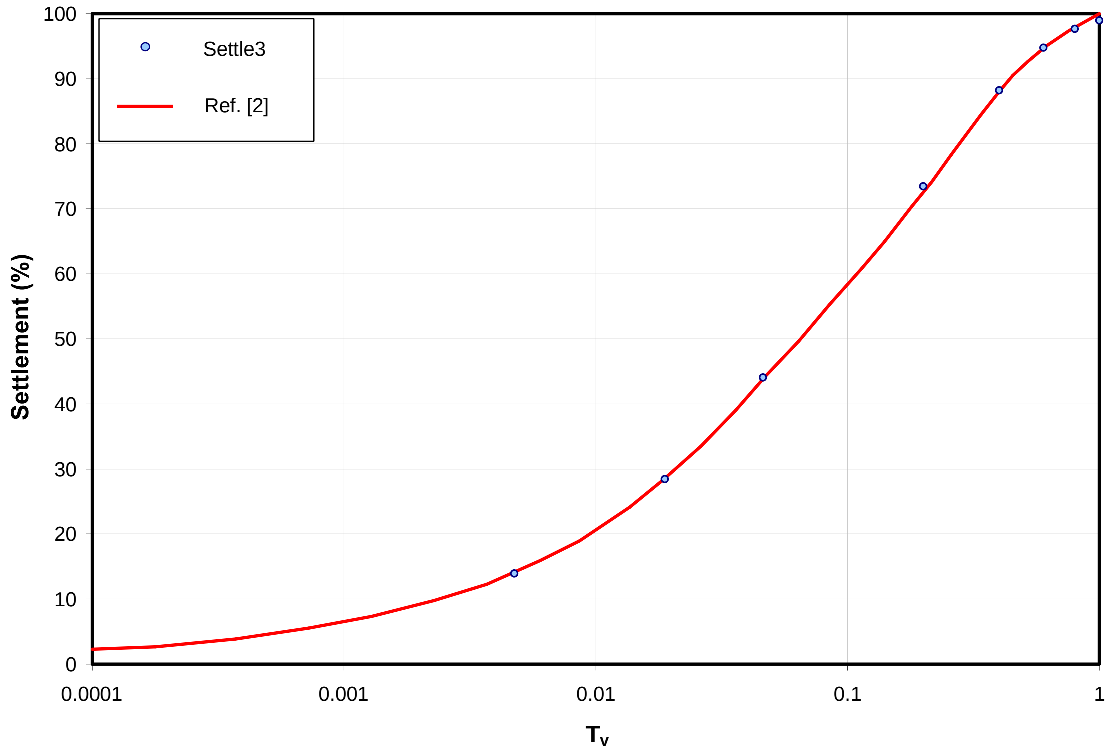


Fig. 4.3 Consolidation Profile (Single Drainage case)

4.2.2. Double Drainage case:

Figure 4.4 shows the excess pore pressure distribution with depth for different time factors (T_v) calculated by ***Settle3*** compared to those presented in Chen et al. [1]. Figure 4.5 shows the settlement prediction due to consolidation given by ***Settle3*** compared to that in Chen et al. [1].

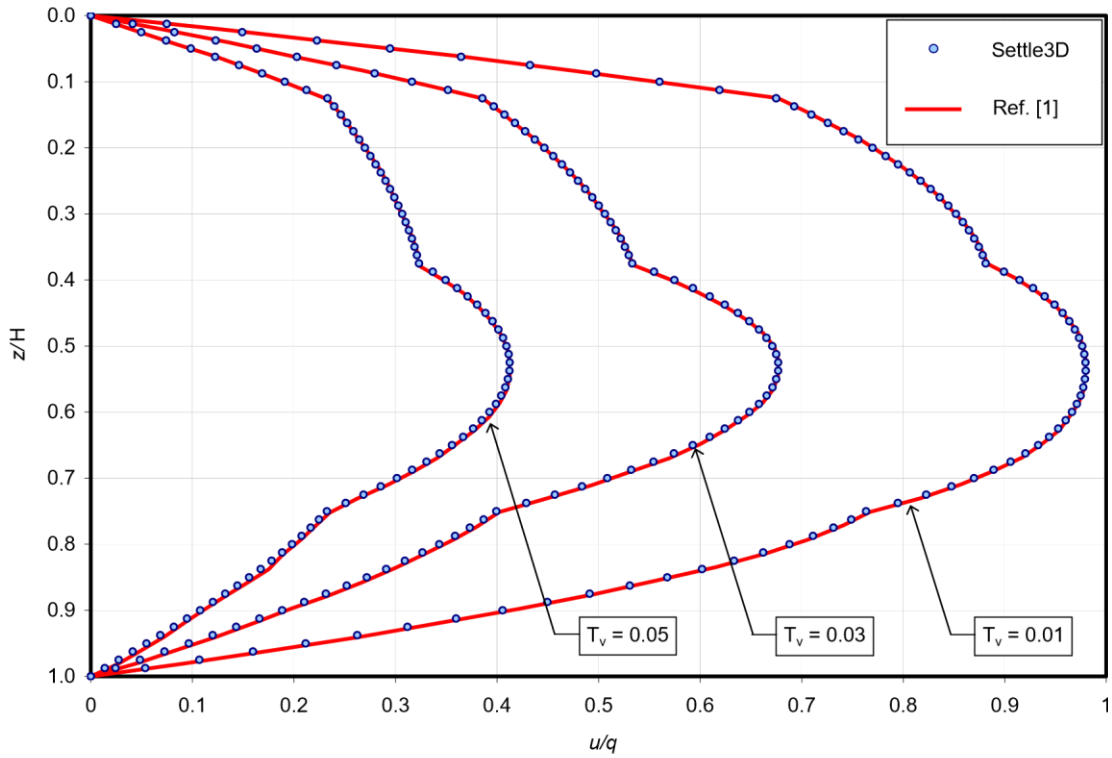


Fig. 4.4 Excess Pore Pressures Distribution with Depth (Double Drainage case)

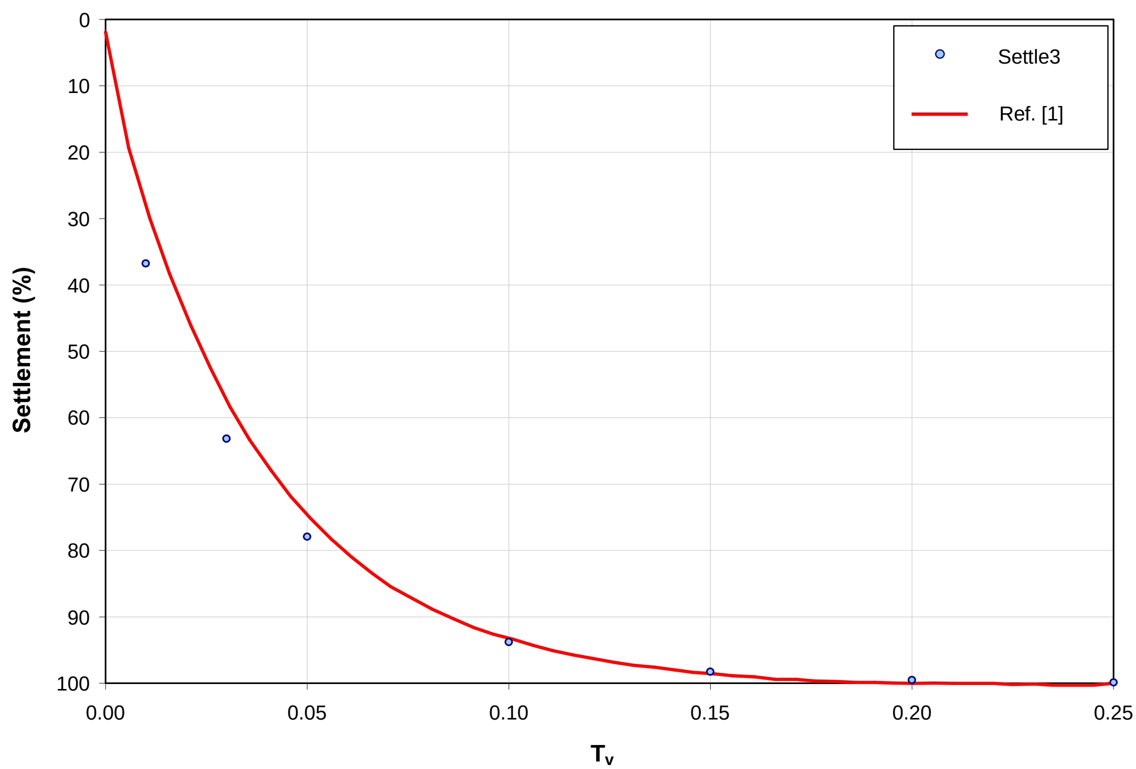


Fig. 4.5 Consolidation Profile (Double Drainage case)

4.3. References

1. R. P. Chen, W. H. Zhou, H. Z. Wang, and Y. M. Chen (2005), “One-dimensional nonlinear consolidation of multi-layered soil by differential quadrature method”, Computers and Geotechnics, Vol. 32, pp. 358-369.

2. P. K. K. Lee, K. H. Xie and Y. K. Cheung (1992), “A study on one-dimensional consolidation of layered systems”, International Journal for Numerical Methods in Geomechanics, Vol. 16, pp. 815-831.

5. Janbu settlement

5.1. Problem Description

A three-layered soil is considered in this example. The model is based on a paper by Janbu (1963). A uniform loading (*q*) of 15 t/m^2 is applied on top of the soil strata. The model geometry of the problem and its properties are shown in Figure 5.1.

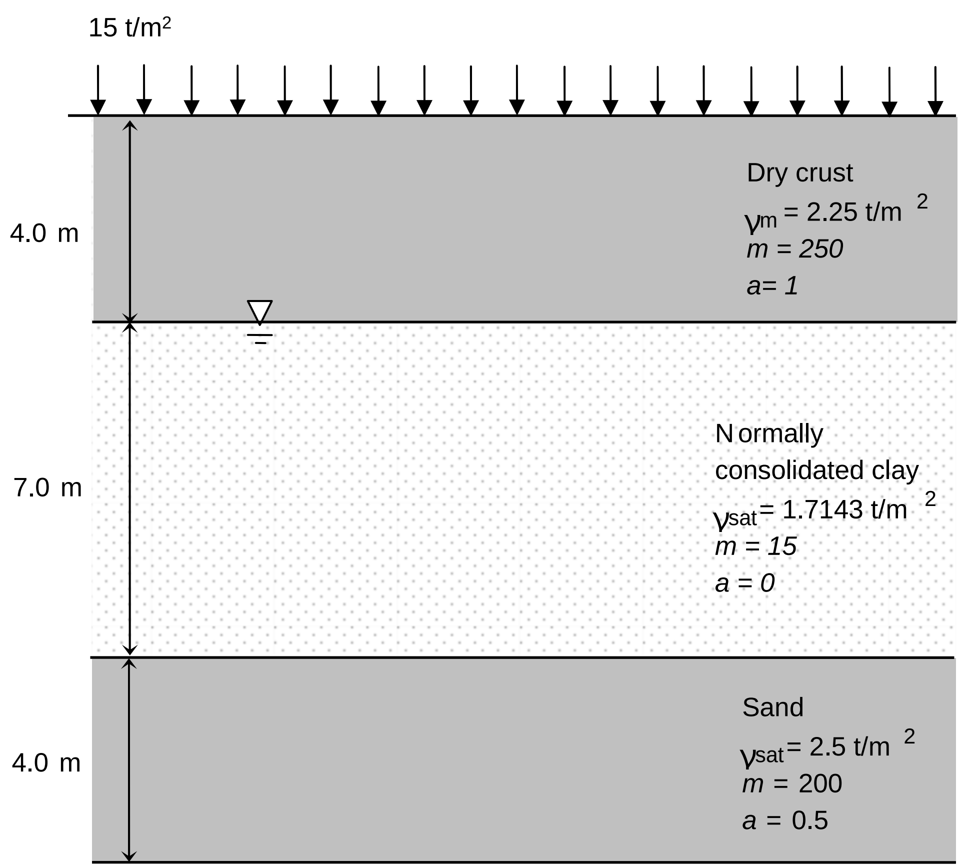


Figure 5.1 – Model Geometry

5.2. Results and Discussion

The result for consolidation obtained using ***Settle3*** was 43.5 cm, which is in very good agreement with the result obtained by Janbu (1963) of 43.6 cm.

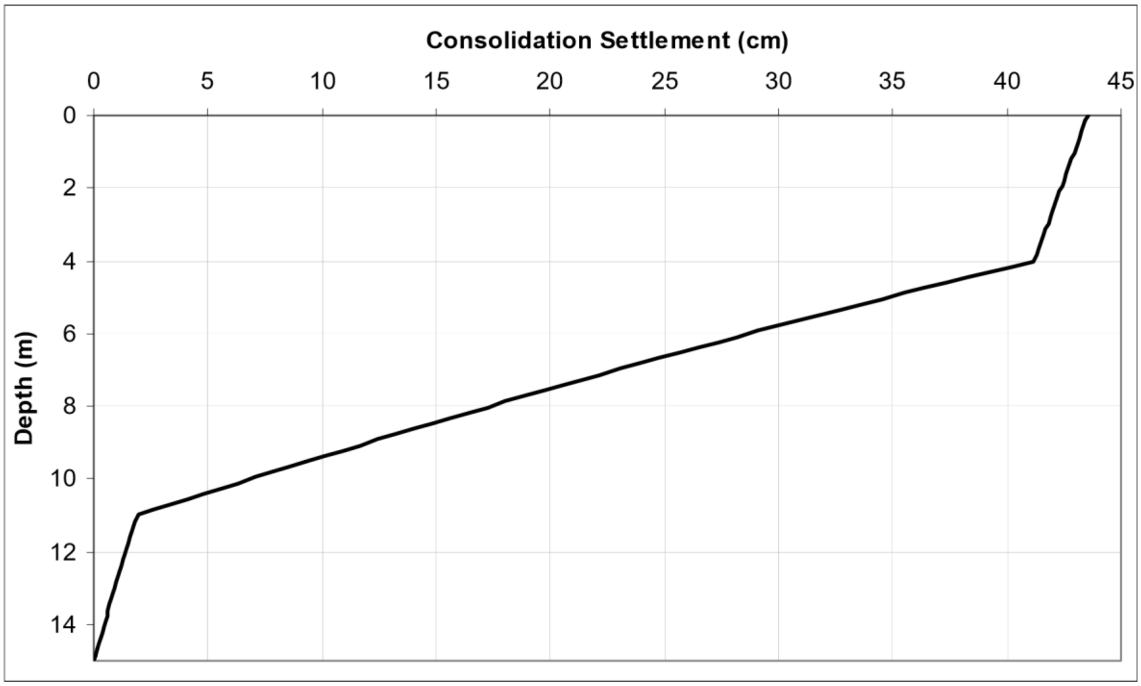


Figure 5.2 – Consolidation Settlement vs. Depth (Settle3)

Layer	Settlement in Layer (cm)	
	Reference [1]	Settle3
Dry Crust	2.4	2.35
Normally Consolidated Clay	39.2	39.26
Sand	2.0	1.93

Table 5.1 Settlement in Each Layer

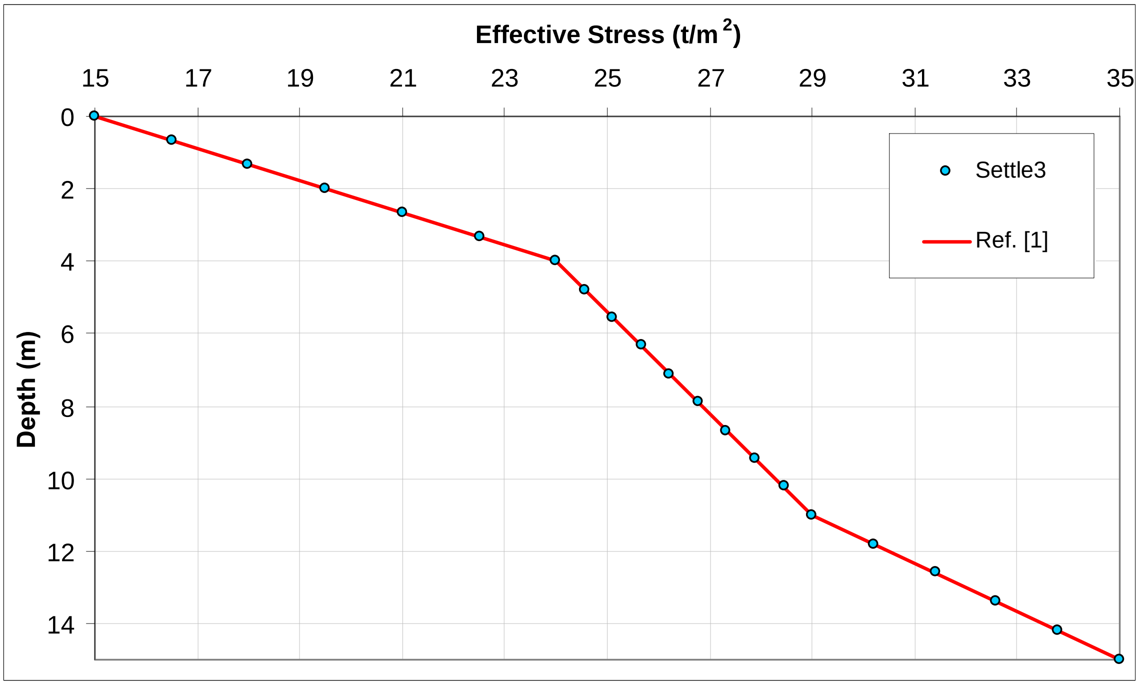


Figure 5.3 –Effective Stress vs. Depth

5.3. References

1. Janbu, N., 1963. “Soil Compressibility as Determined by Oedometer and Triaxial Tests”, European Conference on Soil Mechanics and Foundation Engineering, Wiesbaden, Vol. 1, pp 19-25 and Vol 2. pp 17-21.

6. Consolidation analysis of multi-layered soil

6.1. Problem Description

A five-layered soil is considered in this example. The model is based on technical support question sent by Dott. Gianni Togliani. A large uniform loading of 26 kPa is applied on top of the soil strata. The model geometry of the problem and its properties are shown in Figure 6.1.

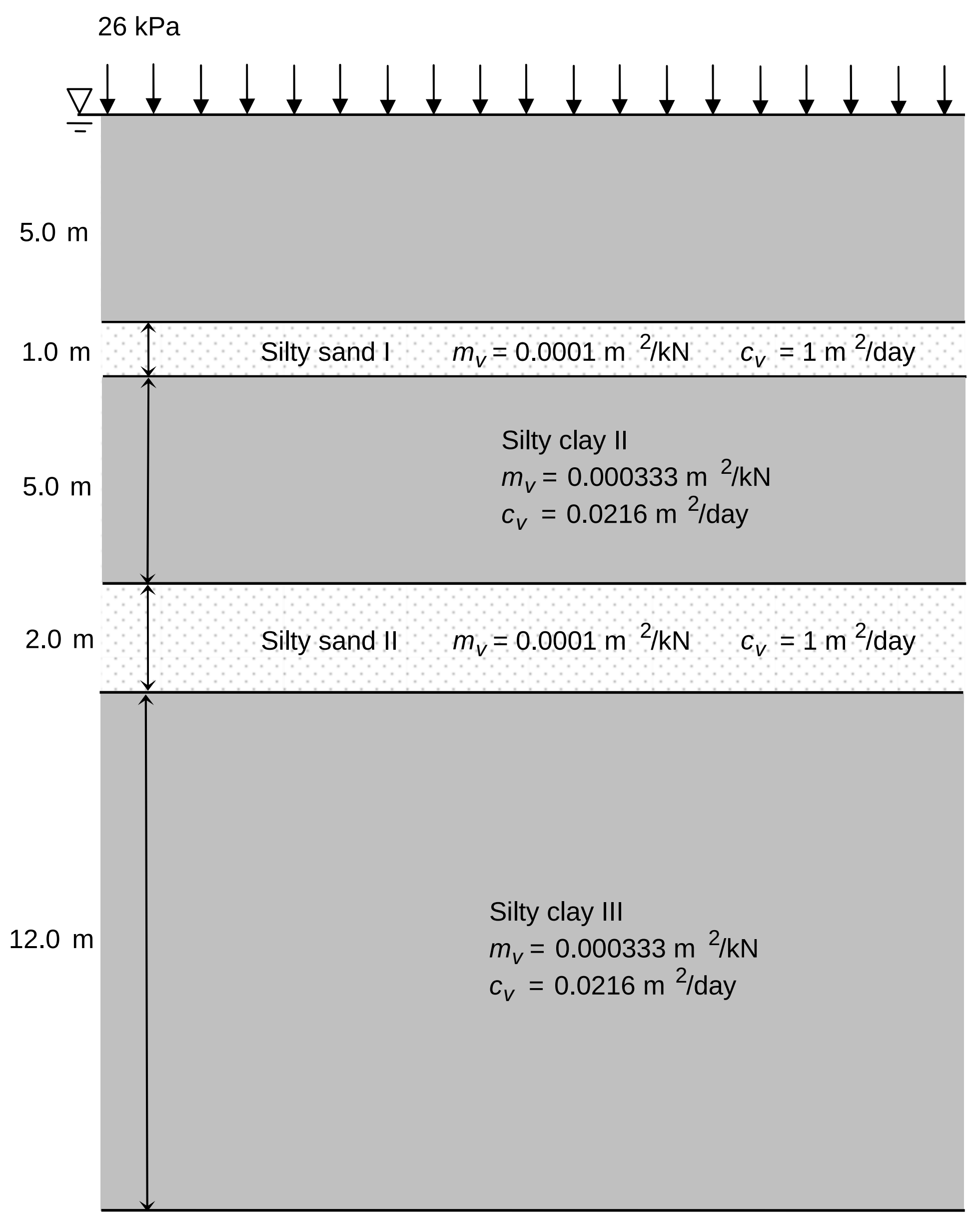


Figure 6.1 – Model Geometry

6.2. Results and Discussion

The result for the final consolidation obtained using ***Settle3*** was 18.9 mm, which is in the same as the result obtained using Terzaghi's one-dimensional consolidation theory.

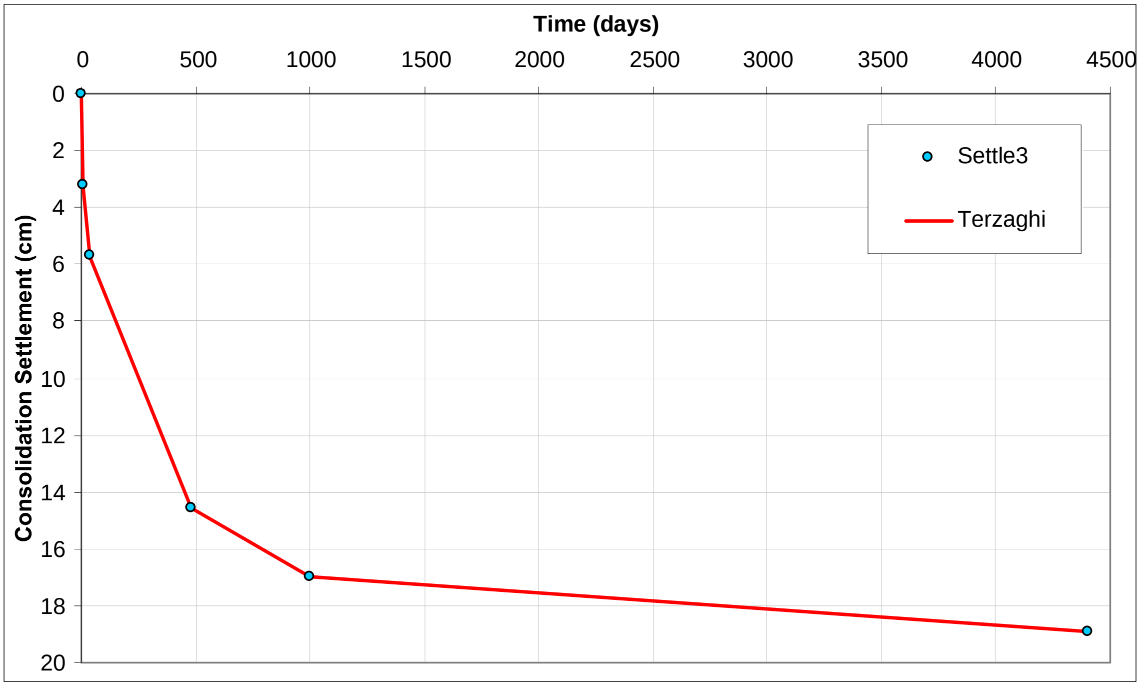


Figure 6.2 – Consolidation Settlement vs. Time

Layer	Consolidation Settlement in Layer (cm)	
	Terzaghi	Settle3
Silty Clay I	3.42	3.42
Silty Sand I	0.26	0.26
Silty Clay II	4.33	4.33
Silty Sand II	0.52	0.52
Silty Clay III	10.39	10.4

Table 6.1 Final Consolidation Settlement in Each Layer

6.3. References

1. Spangler, Merlin G. and Handy, Richard L., "Soil Engineering", Fourth Edition, Harper & Row Publishers, New York, 1982.

7. Consolidation of Bangkok Clay

7.1. Problem Description

A four-layered soil is considered in this example. The model is based a report written by the Department of Highways in Thailand. A 16-m wide embankment is applied on top of the soil strata in two stages, 0.5 m is applied at time = 0 days and the another meter is applied at time = 262 days. The model geometry of the problem and its properties are shown in Figure 7.1.

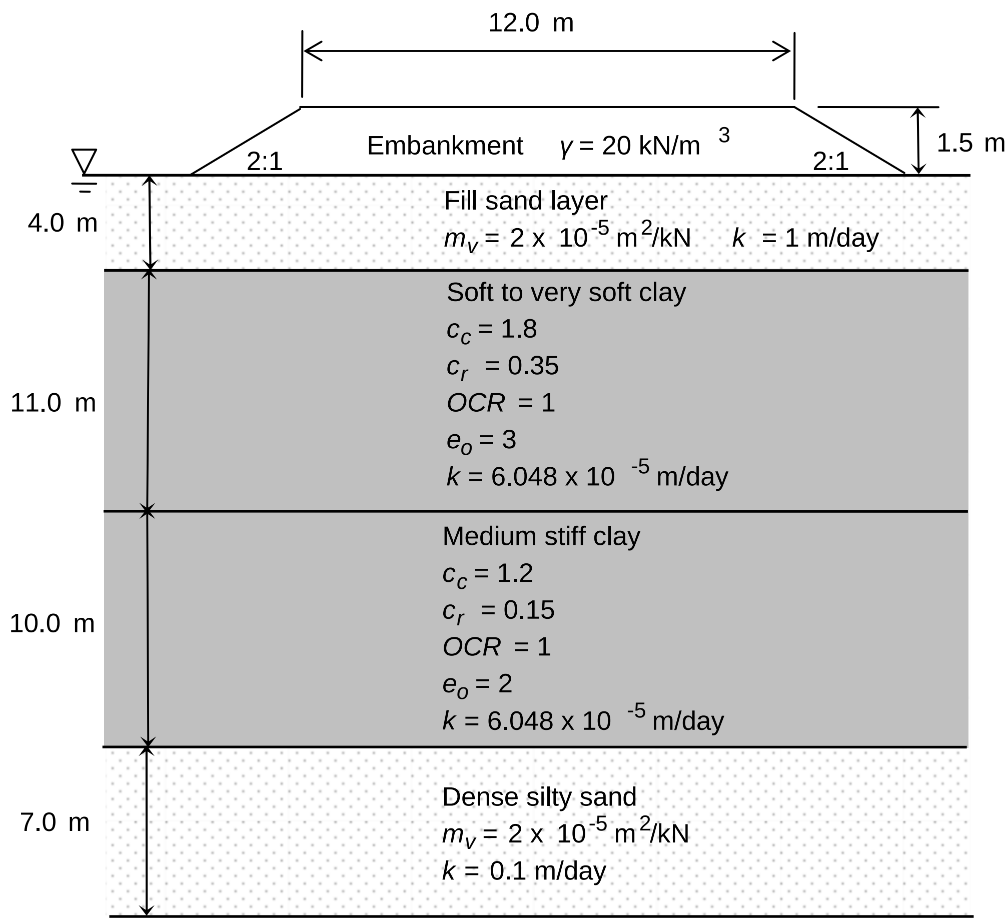


Figure 5.1 – Model Geometry

7.2. Results and Discussion

The results are plotted in Figure 7.2. The reason for the small discrepancy is that in the reference, total settlement is plotted, however for ***Settle3*** only the consolidation settlement is plotted, for lack of information about the stiffness of the soil.

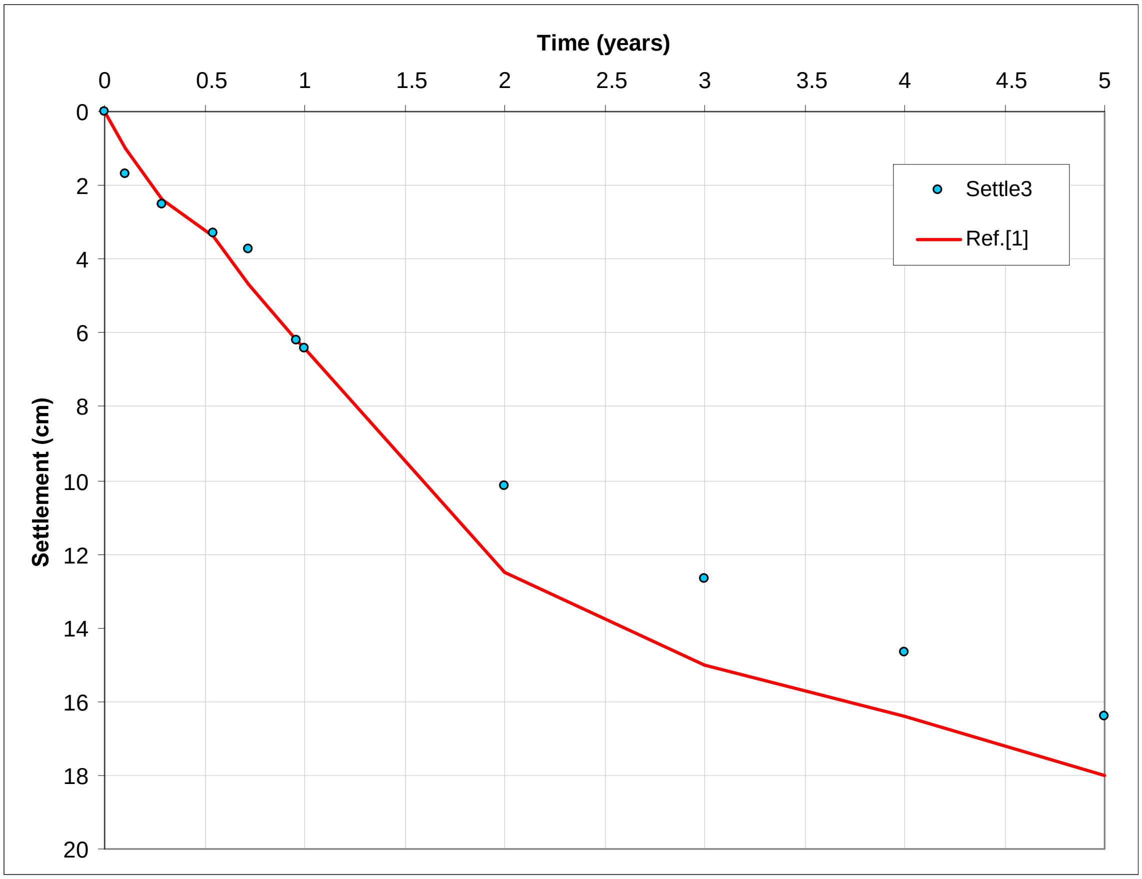


Figure 7.2 –Settlement vs. Time

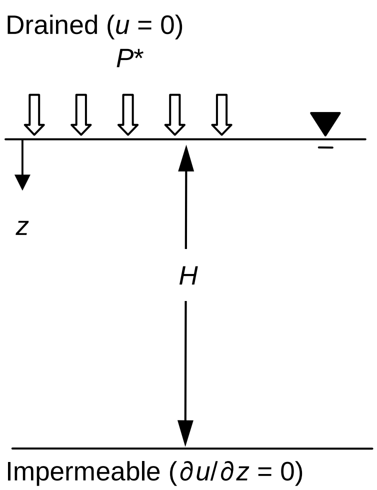
7.3. References

1. Apimeteetamrong, S., Sunitsakul, J., and Sawatparnich, A. 2007 “Performance of Highway Embankments on Bangkok Clay”, Bureau of Road Research and Development, Department of Highways, Thailand.

8. Single Drainage, 1 Layer

8.1. Problem Description

Detournay and Cheng (1993) provide an analytical solution for 1D consolidation of a soil layer of thickness *H* with drainage at the top and an impermeable boundary at the bottom. A load, *P**, of infinite extent is applied at the surface. The geometry of the problem is shown below.



For incompressible fluid and incompressible solid grains, the excess pore pressure for the dimensionless coordinate $\zeta = z/H$ and dimensionless time $\tau = c_v t/H^2$ is given by:

$$u = P^* \sum_{m=1,3,\ldots}^{\infty} \frac{4}{m\pi} \sin\left(\frac{m\pi\zeta}{2}\right) \exp\left(-\frac{m^2\pi^2\tau}{4}\right)$$

The displacement is given by:

$$w = P^* H m_v \sum_{m=1,3,\ldots}^{\infty} \frac{8}{m^2\pi^2} \cos\left(\frac{m\pi\zeta}{2}\right)\left[1-\exp\left(-\frac{m^2\pi^2\tau}{4}\right)\right]$$

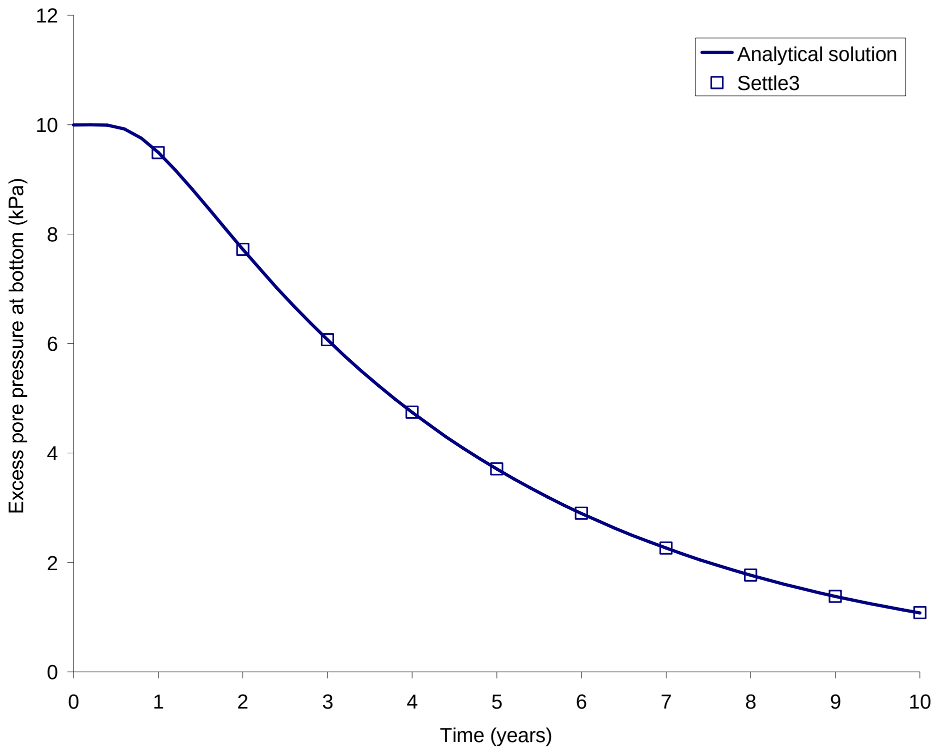
A Settle3 model was constructed to replicate this problem. The following parameters were used:

Applied load, P^*	= 10 kPa
Thickness of layer, H	= 10 m
Coefficient of compressibility, m_v	= 0.001 m^2/kN
Coefficient of consolidation, c_v	= 10 m^2/year

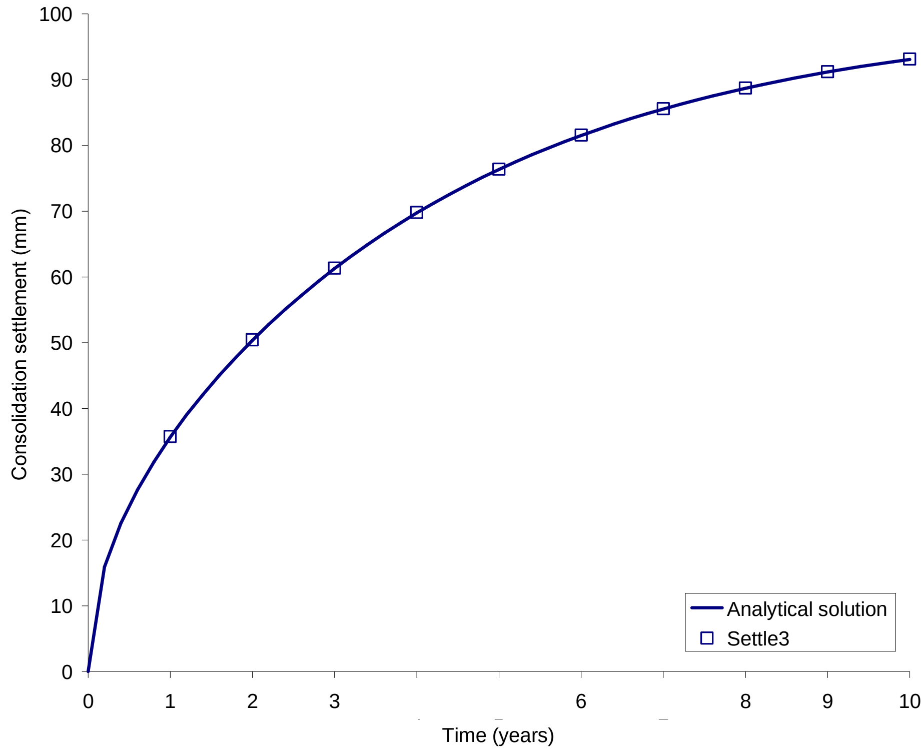
The 1D load was applied by specifying a circular loading area with a radius of 1000 m. Results were obtained at a query point with 20 divisions in the centre of the load.

8.2. Results and Discussion

The model was run for 10 stages, each of duration equal to 1 year. Results for pore pressure at the bottom of the model and consolidation settlement at the top of the model compared with the analytical solution are shown below.



Excess pressure at the bottom of the soil column.



Settlement at the top of the soil column (displacement is positive down)

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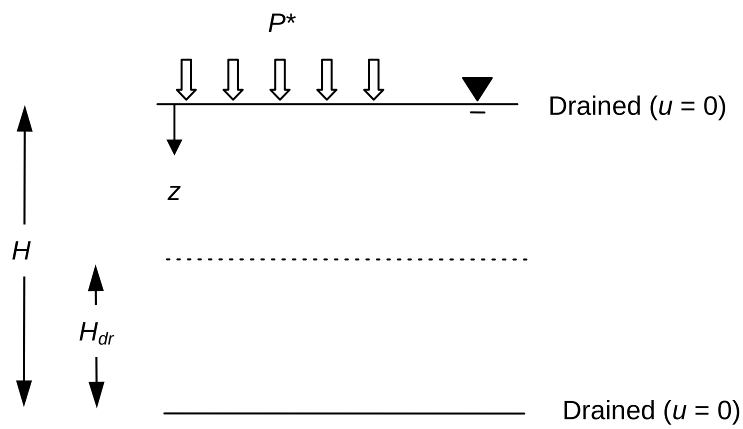
8.3. References

1. Detournay, E. and Cheng, A. H.-D. (1993). Fundamentals of Poroelasticity: Terzaghi's one-dimensional consolidation. *Comprehensive Rock Engineering* (pp.145-148). New York: Pergamon Press Inc.

9. Double Drainage, 1 Layer

9.1. Problem Description

The solution for excess pressure at any point in a doubly drained homogeneous layer subjected to one-dimensional loading conditions is given by Means and Parcher (1963) and referenced in Coduto (1999, p. 429). The geometry of the problem is shown below. A load *P** of infinite extent is applied at the surface of a layer of thickness *H*.



Because the layer is doubly drained, the length of the drainage path, H_{dr} is half the layer thickness (= H/2). The solution for excess pore pressure at dimensionless coordinate $\zeta = z/H_{dr}$ and dimensionless time $\tau = c_v t / H_{dr}^{\ 2}$ is given by:

$$u = P^* \sum_{m=1,3,...}^{\infty} \frac{4}{m\pi} \sin\left(\frac{m\pi\zeta}{2}\right) \exp\left(-\frac{m^2\pi^2\tau}{4}\right)$$

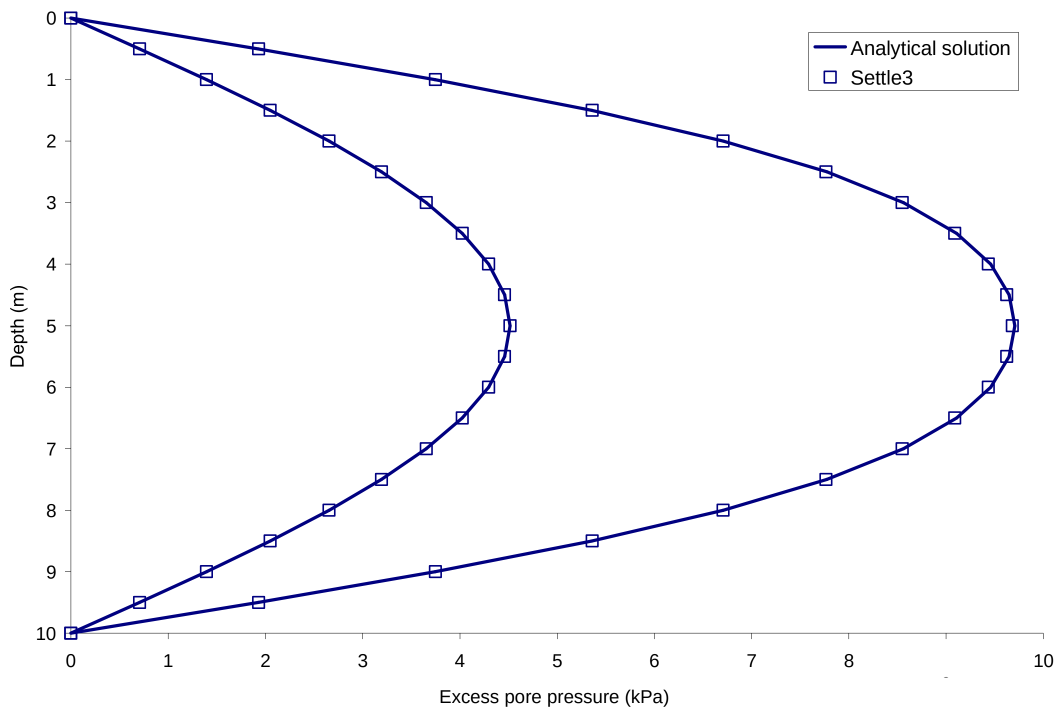
A Settle3 model was constructed to replicate this problem. The following parameters were used:

Applied load, *P**	= 10 kPa
Thickness of layer, *H*	= 10 m
Coefficient of consolidation, c_v	= 0.0021 m^2/day

The 1D load was applied by specifying a circular loading area with a radius of 1000 m. Results were obtained at a query point with 20 divisions in the centre of the load.

9.2. Results and Discussion

The solution was calculated at two times: *t* = 1000 days and *t* = 5000 days. The results are shown below.



Excess pressure versus depth for the doubly drained model.

9.3. References

1. Means, R.E. and Parcher, J.V. (1963). Physical Properties of Soils, Charles E. Merrill Books, Inc.
2. Coduto, D.P. (1999). Geotechnical Engineering: principles and practices, Prentice Hall Inc., 759 p.

10. Horizontal flow due to Wick drains

10.1. Problem Description

Horizontal flow to a single vertical drain can be calculated using the radial flow equation. This equation gives the pore pressure at any given point in at any given time. Performing these calculations for an array of drains quickly becomes unwieldy so some assumptions are made:

- Each drain has a circular (cylindrical in 3D) zone of influence. The edge of this zone is impermeable
- The pore pressure at any point within the zone of influence equals the average pore pressure throughout the entire zone
- The radius of the zone of influence equals the equivalent spacing between drains. Therefore any point within the drain array will experience the same pore pressure drop at the same time (for 1-dimensional loading)

In general, the drain is assumed to have infinite permeability, such that the time required for fluid to flow up the drain is negligible. However, it is possible to consider well resistance in the solution. The solution can be further extended to include the effect of smear, where a zone of specified radius around the drain has a lower permeability.

Several solutions exist to calculate this pore pressure, the most popular being Barron (1948) and Hansbo (1981). Both solutions give the excess pore pressure:

$$u_e = u_{e0} \exp\left(\frac{-8T_r}{\mu}\right)$$

Where the dimensionless time factor, $T_r = \frac{c_h t}{(2r_e)^2}$ when c_h is the horizontal consolidation coefficient and r_e is the equivalent radius of the zone of influence. The parameter μis a function of the drain geometry. Barron and Hansbo give slightly different formulations for μ, but results are generally similar. The formulation of Barron is:

$$\mu = \frac{n^2}{n^2 - S^2}\ln\left(\frac{n}{S}\right) - 0.75 + \frac{S^2}{4n^2} + \frac{k_h}{k_s}\left(\frac{n^2 - S^2}{n^2}\right)\ln S$$

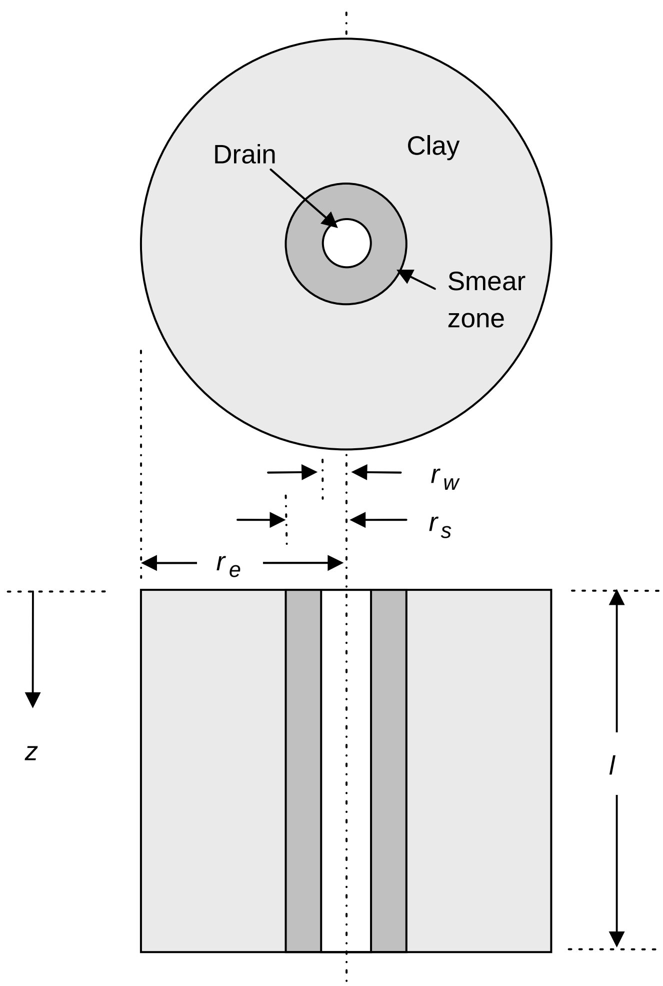
Where n is the ratio of the radius of the zone of influence to the radius of the well,

$n = \frac{r_e}{r_w}$, S is the ratio of smear zone radius to well radius, $S = \frac{r_s}{r_w}$, and $\frac{k_h}{k_s}$ is the ratio of undisturbed soil permeability to smear zone permeability. If S = 1 and $\frac{k_h}{k_s} = 1$, then there is no smear zone.

The above equations assume that the drain itself has an infinite permeability. To account for well resistance, μ is should be replaced by μ_r (Hansbo, 1981)

$$\mu_r = \mu + \pi z(2l - z) k_h / q_w$$

Where l is the length of the drain, z is the distance from the top of the well, k_h is the horizontal permeability of the clay and q_w is the discharge capacity of the well (volume / time). If the well drains at both the top and the bottom (i.e. the bottom intersects a highly permeable layer), the l is set to half the length of the drain.



10.2. Results

10.2.1. Horizontal flow, no smear, no well resistance

A Settle3 model is created with a circular load of radius 1000 m (to simulate a load of infinite extent) and magnitude of 10 kPa. Therefore the initial excess pore water pressure, u_{e0} = 10 kPa. A single material layer is created with a thickness of 10 m. A query point of 20 divisions is placed at the centre of the load.

In all of the problems, we will assume no vertical flow. This is achieved by turning off drainage at the top and bottom of the soil layer and setting c_v to a very small value. The material type for the soil layer is linear and the flow parameters used are:

c_v	=	0.0001 m^2/year
c_h/c_v	=	79,000
mv	=	0.00025 m^2/kN

This input yields a value for c_h = 7.9 m^2/year. Note that m_v is only used to calculate k_h when there is well resistance (see 0). In this case, $k_h = m_v \times c_h \times \gamma_w$ = 0.0194 m/year.

A wick drain region is constructed around the query point with the following parameters:

Drain diameter	=	0.4 m
Drain spacing	=	3.186 m
Drain length	=	10 m
Drain pattern	=	Square

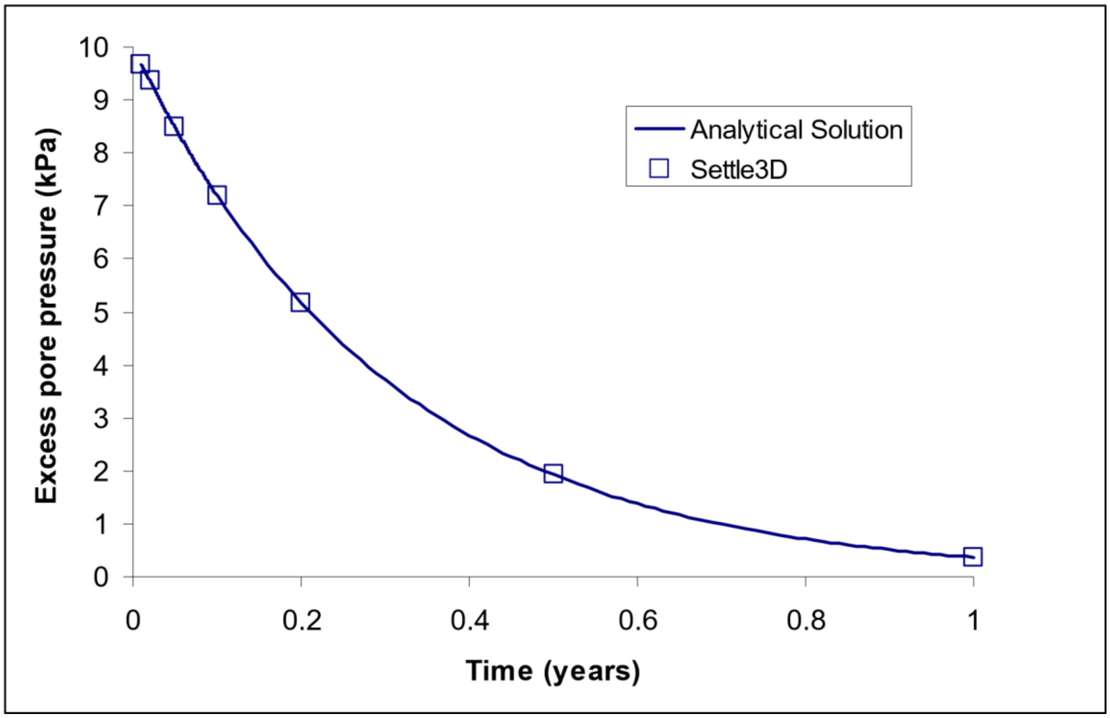
In Settle3 the equivalent radius of the zone of influence is calculated from the drain spacing and pattern:

$r_e = 1.13r$	Square pattern
$r_e = 1.05r$	Triangular pattern

Therefore, for our example, r_e = 1.8 m.

Because there is no vertical flow, the excess pore pressure is the same at all depths. The values for this pore pressure at different times compared to the analytical solution are shown below. For the case with no smear, the solution of Barron (1948) is identical to the solution of Hansbo (1981).

It is clear that Settle3 matches the analytical solution. The maximum error is 0.04%.



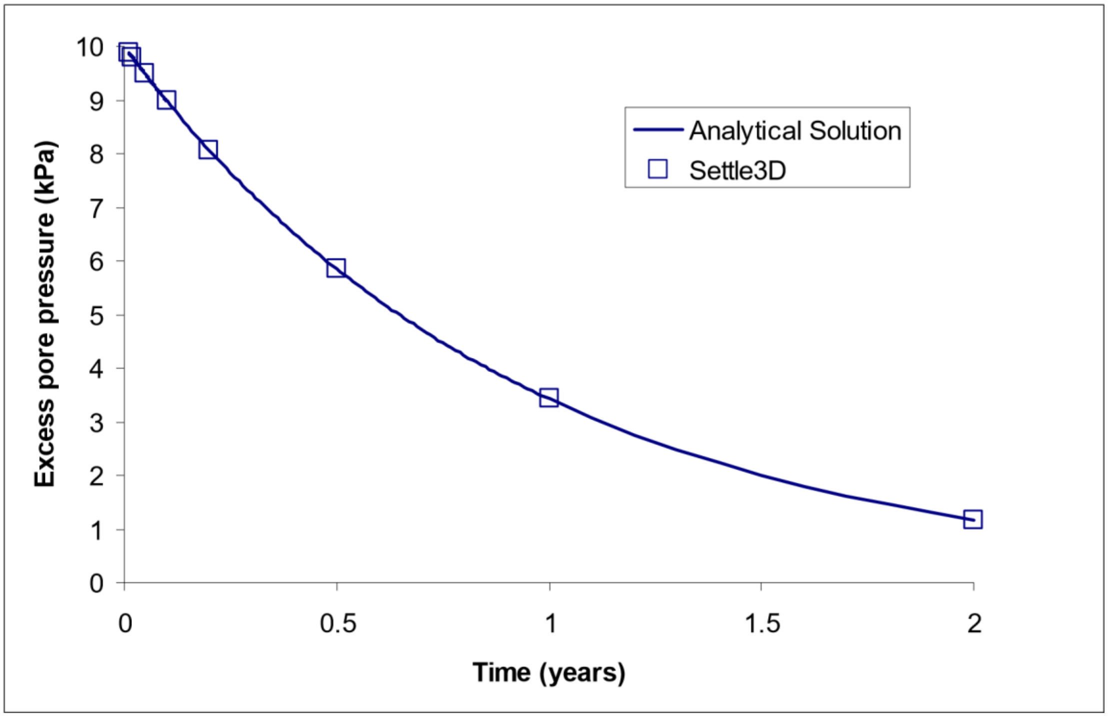
10.2.2. Horizontal flow with smear

The above problem is repeated but this time a smear zone is added. The following parameters are set for the wick drain region:

S = 2.25

Kh/ks = 5

The Settle3 results are shown below compared with the solution of Barron (1948). Again there is a good match with the maximum error = 0.02%. The solution of Hansbo (1981) differs by less than 1% from the Barron solution for this set of parameters.

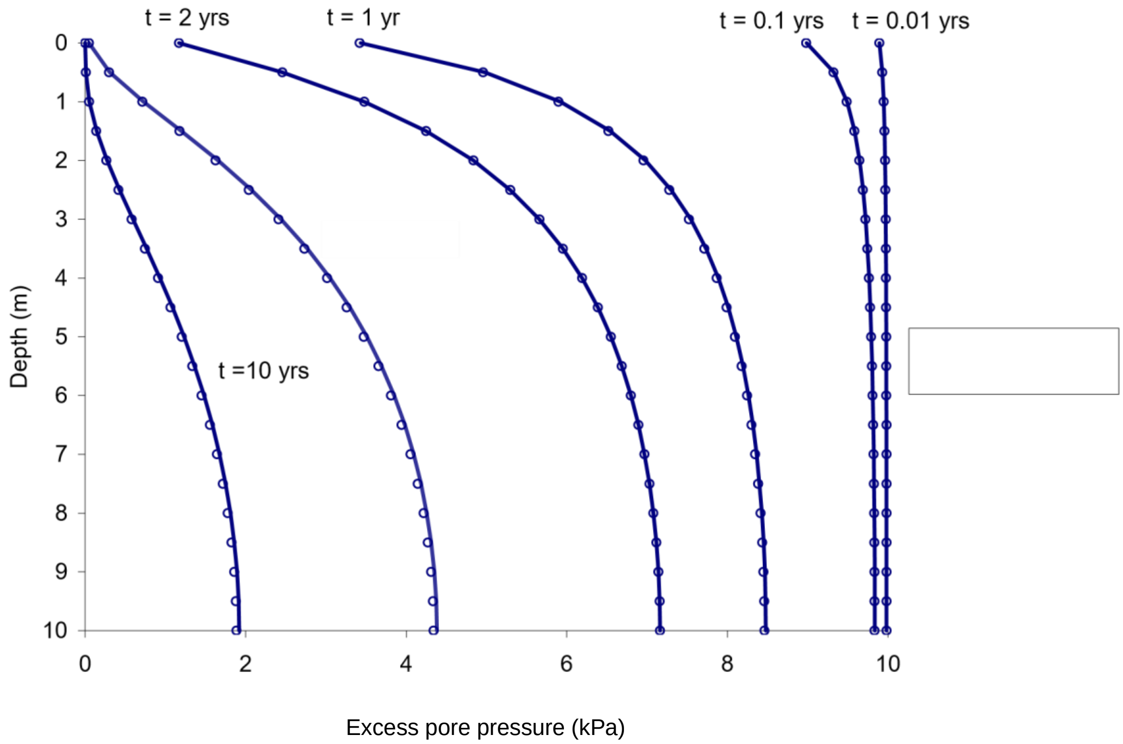


10.2.3. Horizontal flow with smear and well resistance

In this example, it is assumed that the drain has some finite permeability that impedes flow. For the wick drain region, the well resistance is turned on and the following parameters are set:

Double drainage	=	off
q_w	=	0.244 m^3/year

The excess pressure versus depth is shown at different times below. It is clear that unlike the examples with no well resistance, the pore pressure varies with depth. The Settle3 results match well with the solution of Hansbo (1981). A maximum error of 4.5% is observed but this occurs at the top of the soil column at late times when the pore pressures are very small and the errors are therefore magnified. The small deviations from the analytical solution are likely due to the fact that the vertical permeability is not exactly 0.



10.3. References

1. Barron, R.A. (1948). Consolidation of fine-grained soils by drain wells, *Trans. Am. Soc. Civ. Eng.*, **113**, 718-742.
2. Hansbo, S. (1981). Consolidation of fine-grained soils by prefabricated drains, *Proc. 10th Int. Conf. on Soil Mechanics and Foundation Engineering*, Balkema, Rotterdam, **3**, 677-682.

11. Horizontal and Vertical Flow with Wick drains

11.1. Problem Description

Leo (2004) presents a closed-form solution for coupled horizontal and vertical drainage in a soil perforated by Wick drains. A Settle3 model is created to simulate this problem. A circular load is applied with a radius 1000 m (to simulate a load of infinite extent) and magnitude of 10 kPa. A single material layer is created with a thickness of 10 m. The top of the layer is drained and the bottom is undrained. A query point of 20 divisions is placed at the centre of the load.

The material type for the soil layer is linear and the flow parameters used are:

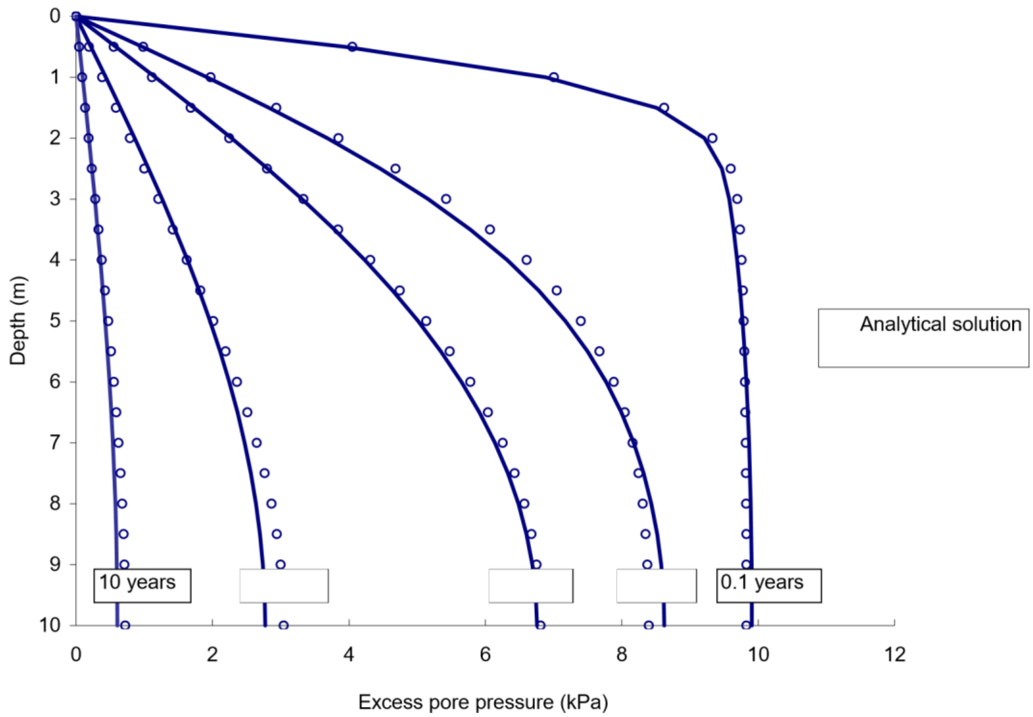
c_v	=	3.95 m^2/year
c_h / c_v	=	2
m_v	=	0.00025 m^2/kN

A wick drain region is constructed around the query point with the following parameters:

Drain diameter	=	0.4 m
Drain spacing	=	3.186 m
Drain length	=	10 m
Drain pattern	=	Square
S	=	2.25
k_h/k_s	=	5
Well resistance	=	on
Double drainage	=	off
q_w	=	0.244 m^3/year

11.2. Results and Discussion

The graph below shows the excess pore pressures at different times calculated by Settle3 compared with the solution of Leo (2004). Settle3 matches quite well the analytical solution. The maximum difference in pressure for the data shown is 0.3 kPa. Leo (2004) notes similar differences between his solution and the solution of Hansbo (1981) if c_v = 0.



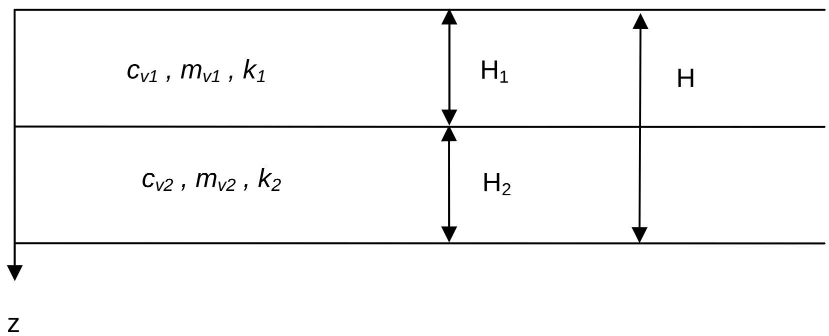
11.3. References

1. Leo, C.J. (2004). Equal strain consolidation by vertical drains, *Journal of Geotechnical and Geoenvironmental Engineering*, **130** (3), 316-327.
2. Hansbo, S. (1981). Consolidation of fine-grained soils by prefabricated drains, *Proc. 10th Int. Conf. on Soil Mechanics and Foundation Engineering*, Balkema, Rotterdam, **3**, 677-682.

12. Two-Layer Consolidation

12.1. Problem Description

Analytical solutions for time-dependent consolidation of double soil layers are presented in Zhu and Yin (2005). Solutions are given for 2-layer soil profiles as shown:



Where:

> k_1 , k_2 are the permeabilities of the two layers c_{v1} , c_{v2} are the coefficients of consolidation of the two layers m_{v1} , m_{v2} are the one-dimensional compressibilities of the two layers

For a linear material we assume:

$$c_v = \frac{k}{m_v \gamma_w}$$

Where γ_w is the unit weight of water.

Solutions are expressed in terms of dimensionless parameters p and q where:

$$p = \frac{\sqrt{k_2 m_{v2}} - \sqrt{k_1 m_{v1}}}{\sqrt{k_2 m_{v2}} + \sqrt{k_1 m_{v1}}}$$

$$q = \frac{H_1\sqrt{c_{v2}} - H_2\sqrt{c_{v1}}}{H_1\sqrt{c_{v2}} + H_2\sqrt{c_{v1}}}$$

Three different models are constructed for comparison with the analytical solutions. The analytical solutions are given in terms of p and q as shown above. The p and q parameters only give the contrast between layers so for the Settle3 models the following procedures were used:

- One layer is always assumed to have c_v = 1 m^2/year and the other layer is given a value of c_v relative to this according to *p* and *q*.
- The compressibility m_v for all layers is assumed to be 0.001
- The thicknesses of one layer is always set to 10 m and the other layer thickness is set relative to this according to *p* and *q*

The three models tested used the following parameters:

Model	*p*	*q*	H_1 (m)	H_2 *(m)*	c_{v1} *(m^2/year)*	c_{v2} *(m^2/year)*	*Drainage*
1	0.9	0.8	4.737	10	1	361	Double
2	-0.82	-0.5	10	2.967	102.23	1	Single
3	0.82	0.5	0.3297	10	1	102.23	Single

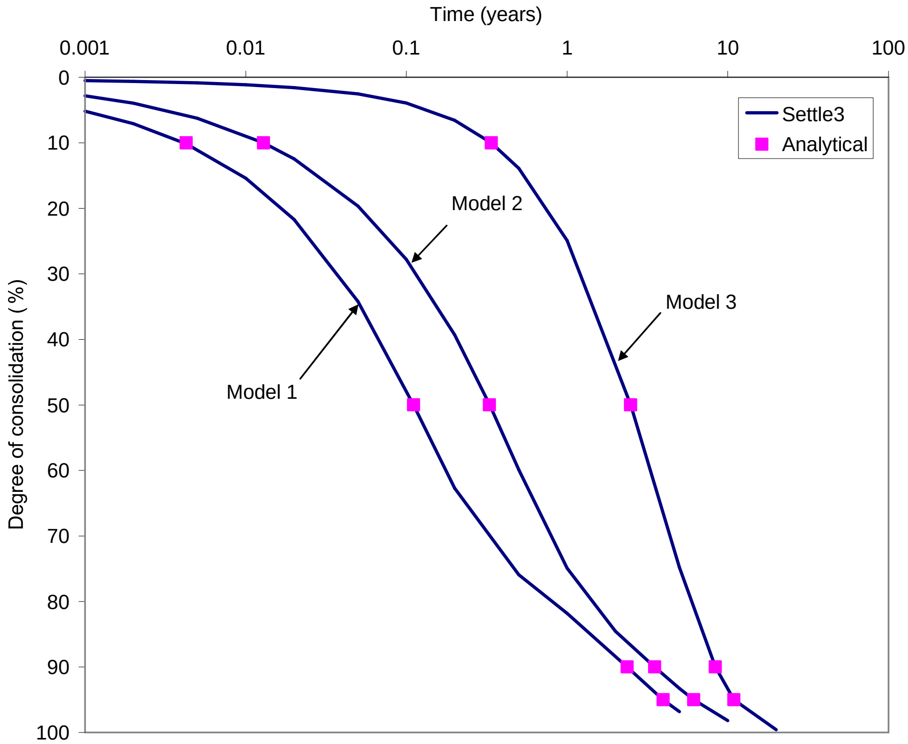
12.2. Results and Discussion

For each model, degree of consolidation is plotted versus time. Degree of consolidation is calculated by first running each model without time-dependent consolidation to get the *final* consolidation displacement (this could also be calculated by simply multiplying m_v × *loading stress* × *H*). The models were then run over many stages with time-dependent consolidation turned on. Degree of consolidation was calculated at each stage by dividing the consolidation settlement at that stage by the final consolidation settlement.

A load of infinite extent is simulated by applying a circular load of radius 1000 m. The magnitude of the load was set to 10 kPa for all models. A query string made up of 100 points was placed at the centre of the load.

The results are shown in the graph below. The solutions of Zhu and Yin (2005) give the time required to reach 10%, 50%, 90% and 95% consolidation. These are plotted along with the Settle3 results. It is clear that Settle3 accurately reproduces the analytical results. Errors are shown numerically in the table below.

	Average error (%)	*Maximum error (%)*
Model 1	0.45	1.7
Model 2	0.050	0.13
Model 3	0.045	0.098



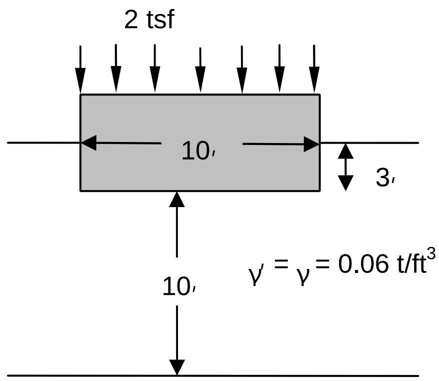
12.3. References

1. Zhu, G. and Yin, J-H, 2005. Solution charts for the consolidation of double soil layers. *Canadian Geotechnical Journal*, **42**, 949-956.

13. Settlement of square load using empirical methods

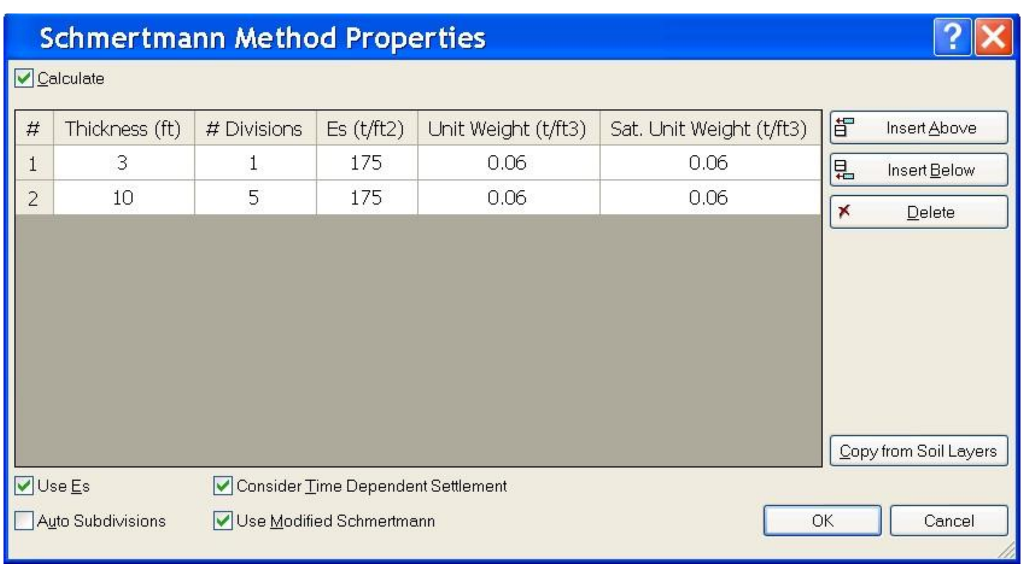
13.1. Problem Description

The U.S. Army Corps of Engineers (1990) provide an example settlement problem with solutions calculated using many different methods. The problem geometry is shown below. A 10 foot square footing is constructed 3 feet below the surface. The magnitude of the load is 2 tsf. The compressible layer is 13 feet thick and the water table is somewhere below this. The material is sand with a unit weight of 0.06 ton/ft^3. The average blow count in the sand corrected to 60% efficiency is N_{60} = 20 blows/ft. The average elastic modulus determined from dilatometer and pressuremeter tests is E_s = 175 tsf. We wish to determine the settlement directly after construction and 10 years after construction.



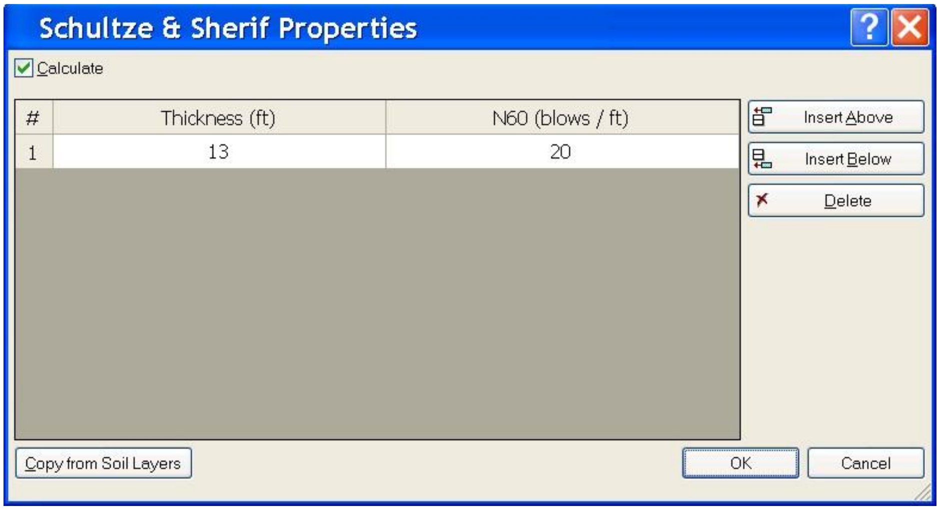
13.1.1. Schmertmann

The input for the Schmertmann analysis in *Settle3* is shown below.



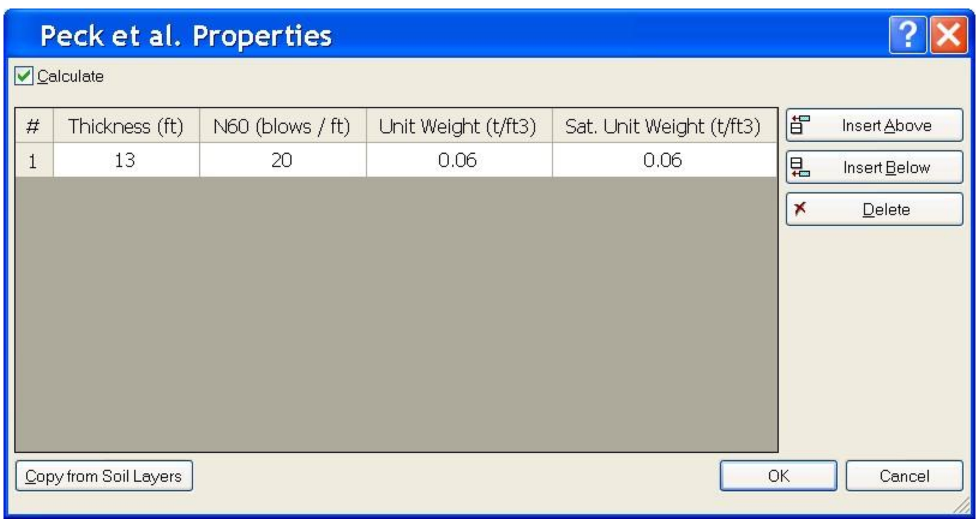
13.1.2. Schultze and Sherif

The input for the Schultze and Sherif method is shown below.



13.1.3. Peck, Hanson and Thornubrn

The input for the Peck, Hanson and Thornburn method is shown below.



13.2. Results

The results for each method calculated by *Settle3* are shown in the table below compared with the results from the Army Corps (1990). You can see that the results are generally the same except for the Peck, Hanson and Thornburn method. The Peck et al.

results are different because the Army Corps incorrectly calculates the overburden correction for blow counts at the bottom of the footing instead of in the middle of the compressible layer. *Settle3* shows the correct value.

The Schmertmann result is slightly different because the Army Corps calculates the influence factor at the boundaries between layers, whereas *Settle3* computes the influence factors and the centre of each layer.

Method	Settlement (Settle3)	Settlement (Army Corps)
Schmertmann (modified) 0 years	0.59 inches	0.57 inches
Schmertmann (modified) 10 years	0.82 inches	0.80 inches
Schultz and Sherif	0.26 inches	0.26 inches
Peck et al.	0.73 inches	0.57 inches

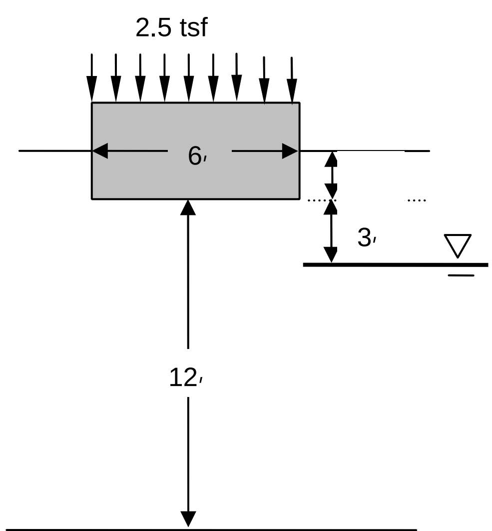
13.3. References

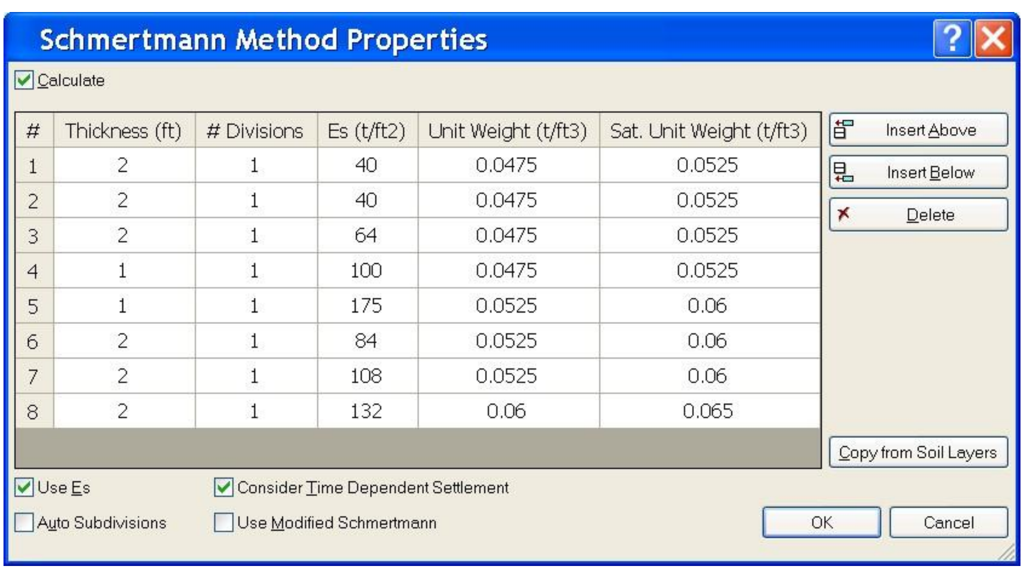
1. U.S. Army Corps of Engineers (1990). Settlement Analysis: Engineer Manual 1110-1-1904. Washington DC.

14. Schmertmann settlement of rectangular load

14.1. Problem Description

Day (2005) presents an example settlement calculation using the classic Schmertmann approach. The footing is 6 × 8 feet and is embedded 2 feet into the soil. The load is 2.5 tsf. The water table is at a depth of 5 feet. The problem geometry and soil layer data used in the analysis are shown below. The settlement is desired 1 year after construction.





#	Thickness (ft)	# Divisions	Es (t/ft2)	Unit Weight (t/ft3)	Sat. Unit Weight (t/ft3)
1	2	1	40	0.0475	0.0525
2	2	1	40	0.0475	0.0525
3	2	1	64	0.0475	0.0525
4	1	1	100	0.0475	0.0525
5	1	1	175	0.0525	0.06
6	2	1	84	0.0525	0.06
7	2	1	108	0.0525	0.06
8	2	1	132	0.06	0.065

14.2. Results

The settlement calculated by *Settle3* is **1.66 inches**. This is essentially the same as the value of 1.67 inches calculated in Day (2005). The difference is likely due to the number of significant digits carried through the calculation.

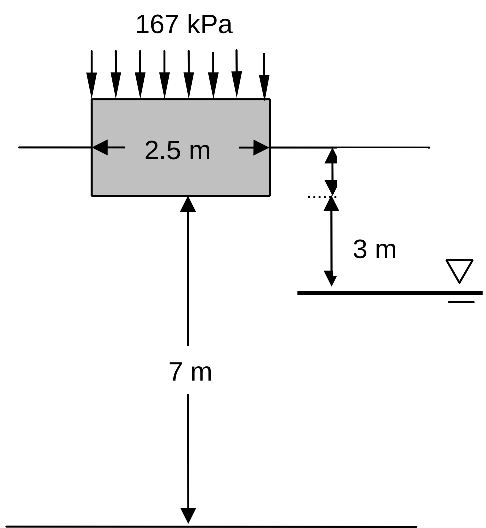
14.3. References

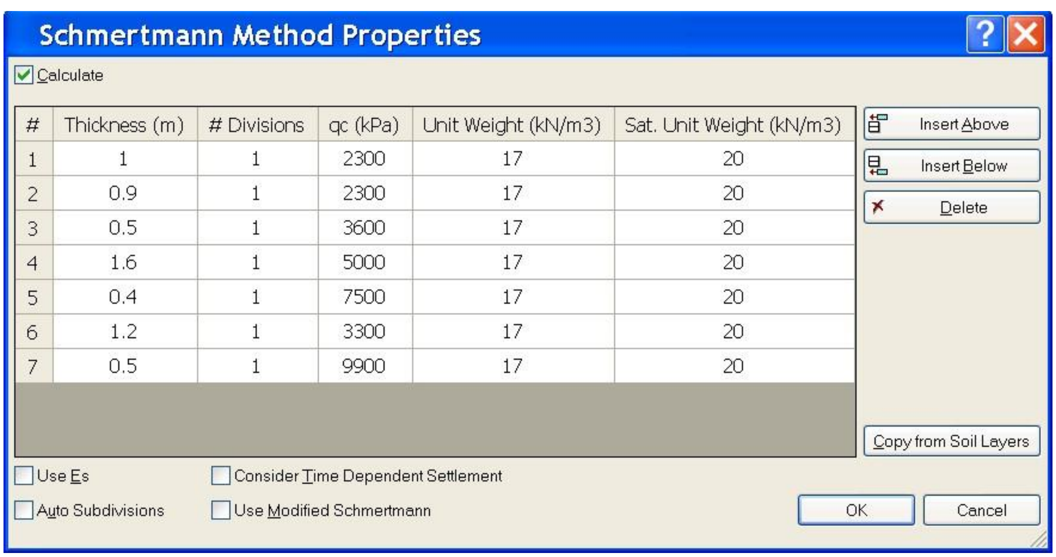
1. Day, R.W. (2005). Foundation Engineering Handbook, McGraw-Hill.

15. Schmertmann settlement of square load

15.1. Problem Description

Craig (1997) presents an example settlement calculation using the classic Schmertmann approach. The footing is 2.5 m × 2.5 m and is embedded 1 m into the soil. The net load is 150 kPa. This represents the load minus overburden at the bottom of the load. The soil unit weight is 17 kN/m^3 therefore the load is 150 kPa + 17kN/m^3 × 1 m = 167 kPa. The water table is at a depth of 4 m. The problem geometry and soil layer data used in the analysis are shown below. The settlement is desired at the time of construction.





#	Thickness (m)	# Divisions	qc (kPa)	Unit Weight (kN/m3)	Sat. Unit Weight (kN/m3)
1	1	1	2300	17	20
2	0.9	1	2300	17	20
3	0.5	1	3600	17	20
4	1.6	1	5000	17	20
5	0.4	1	7500	17	20
6	1.2	1	3300	17	20
7	0.5	1	9900	17	20

15.2. Results

The settlement calculated by *Settle3* is **26.8 mm**. This is essentially the same as the value of 26.9 mm calculated in Craig (2005). The difference is likely due to the number of significant digits carried through the calculation.

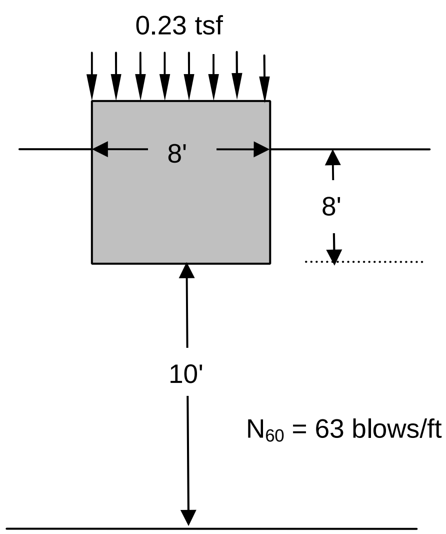
15.3. References

1. Craig, R.F. 1997. Soil Mechanics, 6th edition, Spon Press.

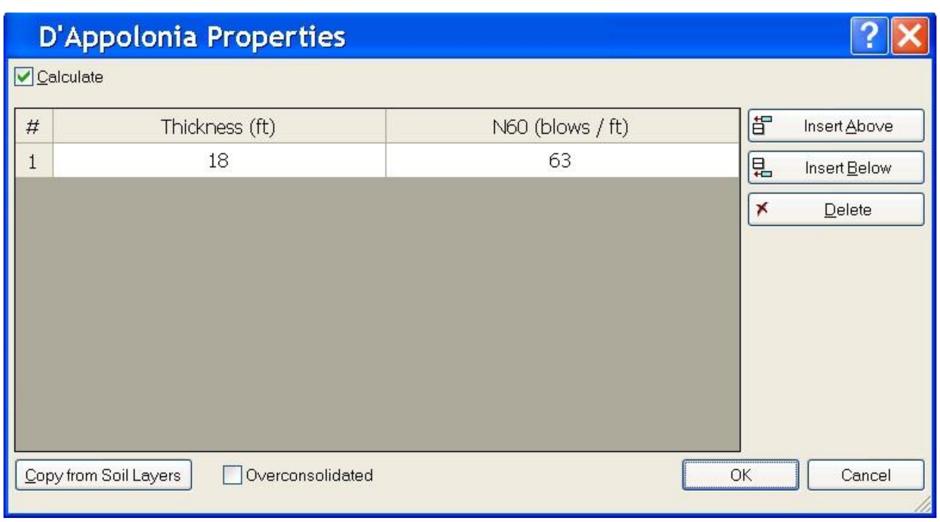
16. D'Appolonia settlement

16.1. Problem Description

Meranda (2005) presents settlement calculations for different footings using different empirical methods. An example using the D'Applonia method is given in section 7.6.8. The problem geometry is shown below. The length of the footing is 40.25 feet and the soil is assumed to be normally consolidated sand/gravel.



The input for *Settle3* is shown below



16.2. Results

Settle3 calculates a settlement of **0.013** inches; very close to the value of 0.014 calculated by Meranda (2005).

16.3. References

1. Meranda, J. (2005). Analysis of spread footing foundations as a highway bridge alternative, MSc thesis, Russ College of Engineering and Technology of Ohio University,

17. Koppejan consolidation

17.1. Problem Description

This problem will test the accuracy of the Koppejan material model in *Settle3* by comparing with analytical results calculated from a spreadsheet.

Assume a 10 m thick layer of normally consolidated Koppejan material with the following properties:

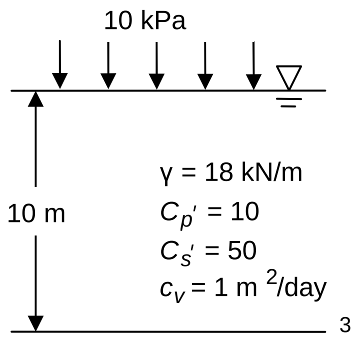
γ = 18 kN/m^3

C_p' = 10 C_s' = 50

c_v = 1 m^2/day

Assume the groundwater table is at the surface and the unit weight of water is 9.81 kN/m^3.

A load of infinite extent and a magnitude of 10 kPa is applied:



17.2. Results

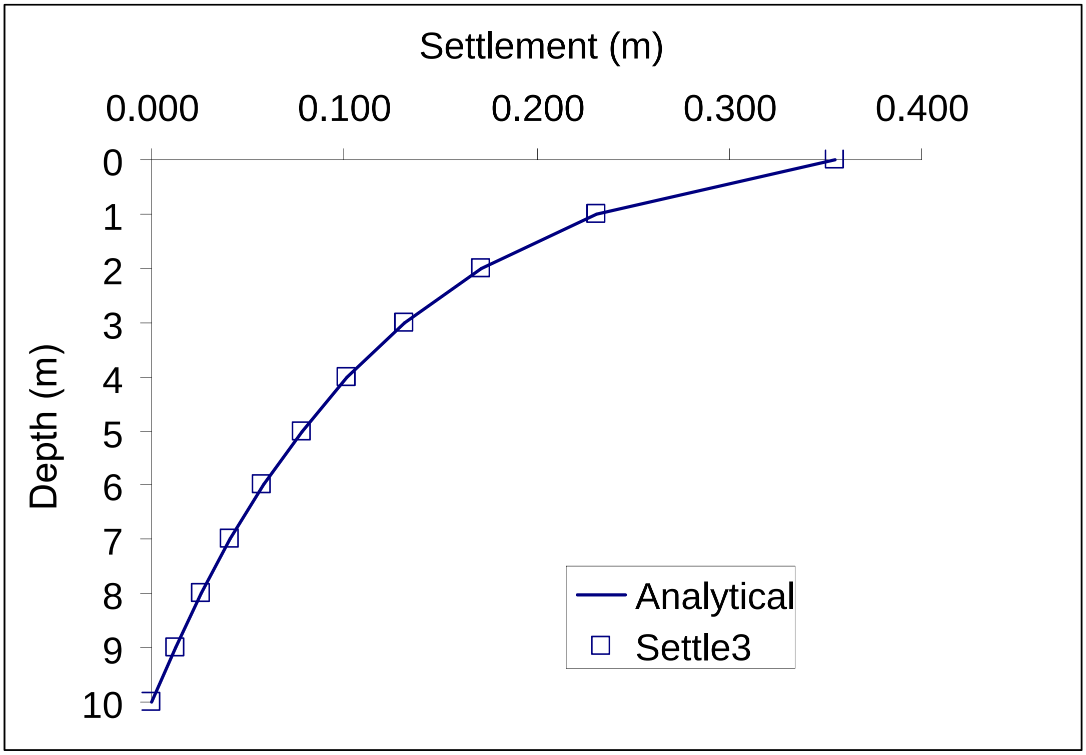
17.2.1. Part 1: No time dependence

To enable hand-calculations, assume the layer is divided up into 10 sublayers. If we neglect the effect of time, we can calculate the ultimate settlement (100% consolidation) using a spreadsheet. The strain in each sublayer is:

$$\varepsilon = \left(\frac{1}{C_p}\right)\ln\left(\frac{\sigma_f'}{\sigma_i'}\right)$$

Where σ_i' is the initial effective stress (due to gravity) and σ_f' is the final effective vertical stress (in this case, $\sigma_f' = \sigma_i' + 10$ kPa). Settlement is then calculated by summing strain×thickness for each sub-layer.

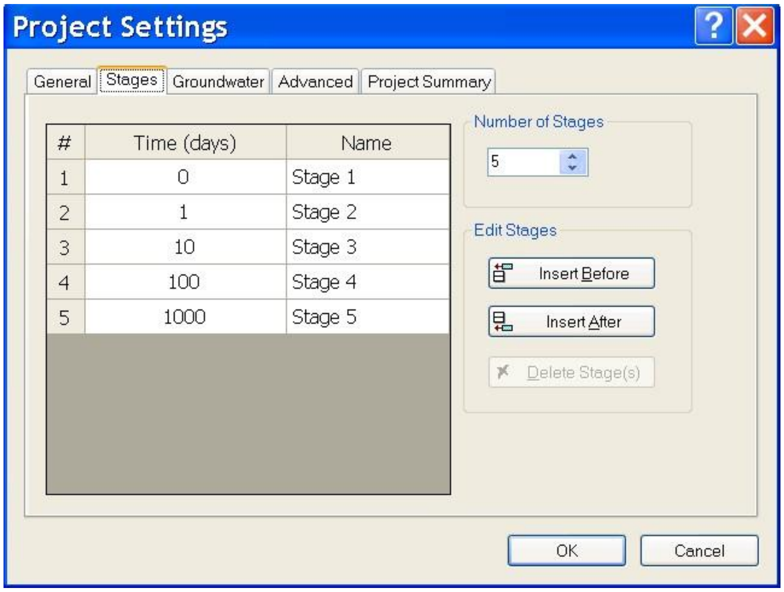
The analytical surface settlement is 0.355 m. The settlement calculated by *Settle3* is exactly 0.355 m. The plot of settlement versus depth is shown below:



17.2.2. Part 2: time dependence and incremental solution

By turning on time-dependence, it is necessary for *Settle3* to perform an incremental solution – i.e. settlement is calculated at many intermediate times prior to the final time at 100% consolidation.

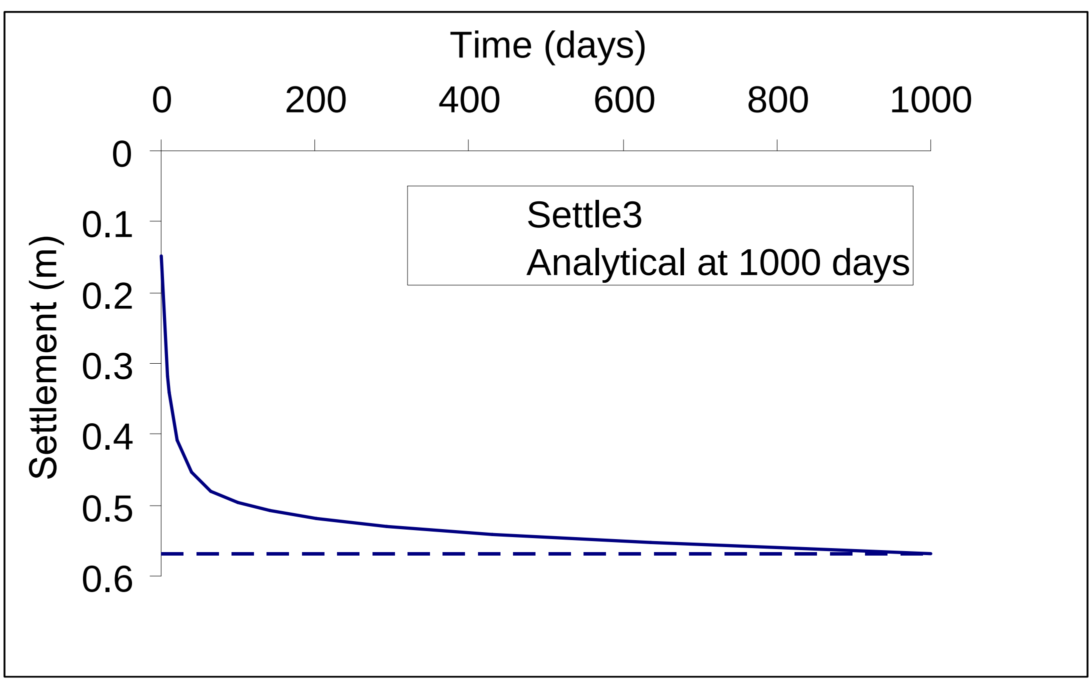
For this test, time dependent consolidation analysis is turned on and the time units are set to days. The coefficient of consolidation for the material is set to $c_v = 1$ m^2/d. Stages are added as shown:



The strain for a sublayer can be calculated at any degree of consolidation, *U*, using the equation:

$$\varepsilon = \left(\frac{U}{C_p} + \frac{1}{C_s}\log(t)\right)\ln\left(\frac{\sigma_f'}{\sigma_i'}\right)$$

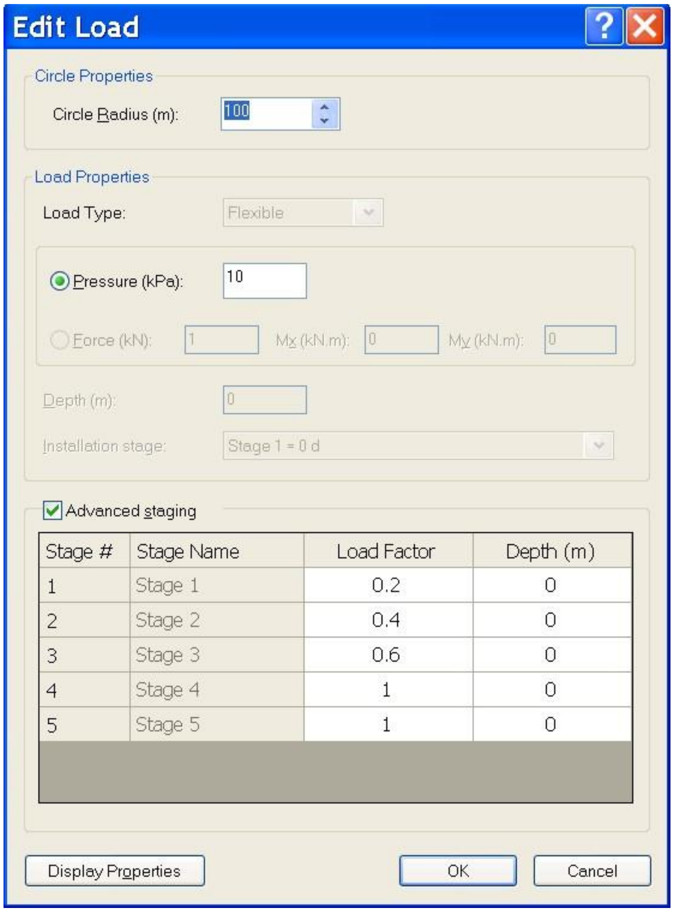
If we assume that all layers have reached 100% consolidation at 1000 days (U = 1), using a spreadsheet we can calculate that the settlement at 1000 days is 0.568 m. The settlement calculated by *Settle3* is also 0.568 m. You can see on the plot below how the settlement evolves with time and arrives at the correct value at 1000 days.



17.2.3. Part 3: Superposition

To test the superposition of loads in *Settle3*, the load is gradually increased through the stages and compared to the settlement that is calculated when the load is applied all during the first stage.

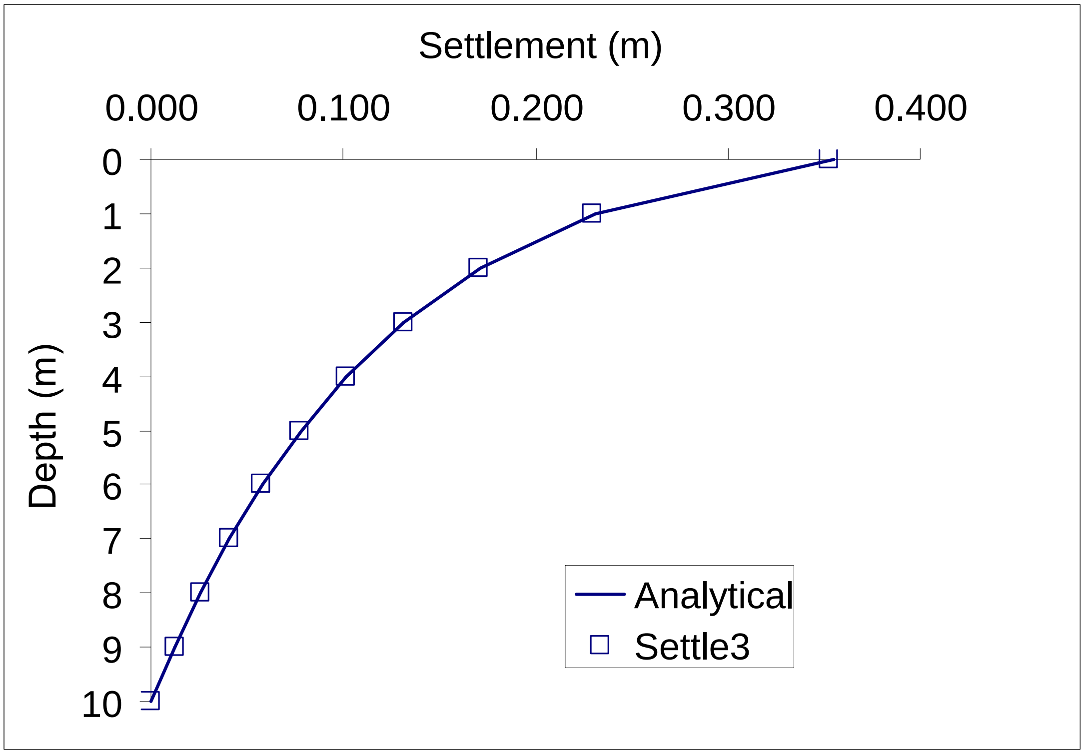
The model from Part 2 was used except the load was subjected to advanced staging as shown:



You cannot compare the secondary consolidation (the time dependent part), however the primary consolidation for this model at 1000 days should be the same as the analytical solution in part 1.

The settlement in part 1 at the surface was calculated to be 0.355 m. The consolidation settlement calculated for this model is 0.352 m – a difference of 0.8%.

The results are shown below:


Settlement (m)
0.000
0.100
0.200
0.300
0.400
Depth (m)
0
1
2
3
4
5
6
7
8
9
10
Analytical
Settle3

18. Secondary settlement with Mesri formulation

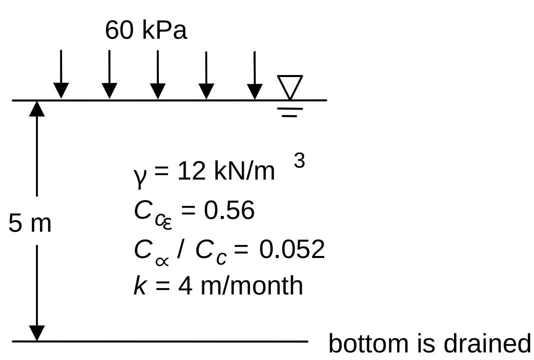
18.1. Problem Description

Mesri at al. (1997) present an example of how surcharging can reduce secondary settlement (creep). The example involves an embankment load on a 5-m thick layer of Middleton Peat. The given properties of the peat are:

$C_{c\varepsilon}$	=	0.56
$C_{\propto} / C_c$	=	0.052
Material	=	Middleton Peat

An embankment load is added that increases the effective vertical stress to 60 kPa. Since the unit weight of peat is generally very low, and the layer is not very thick, we will neglect the effect of gravity load and apply a load of 60 kPa. A load of infinite extent is assumed.

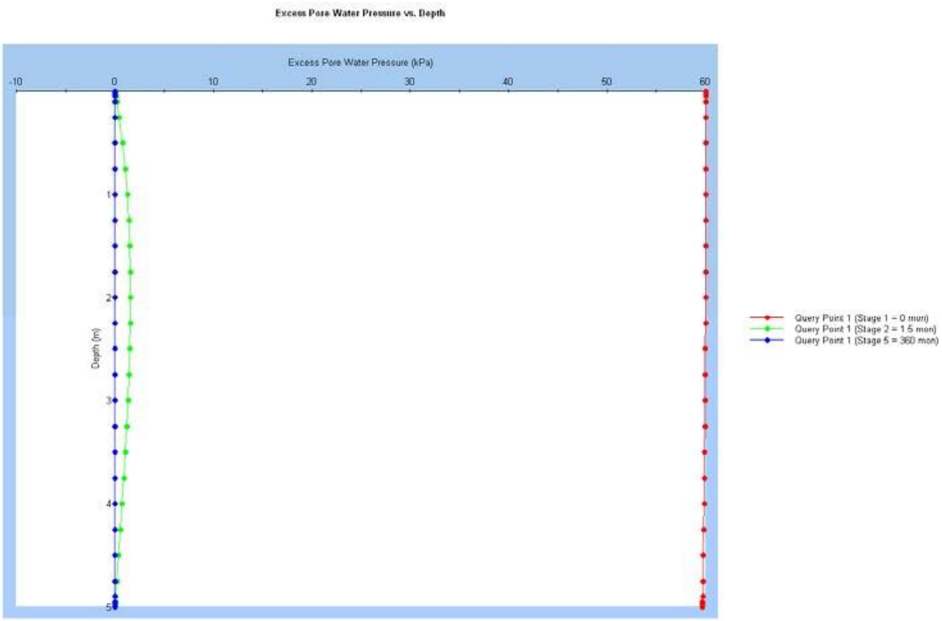
No value is given for permeability, however it is stated that primary consolidation for the embankment load finishes at 6 weeks, therefore a permeability of k = 4 m/month is estimated by trial and error. The problem geometry is shown below.

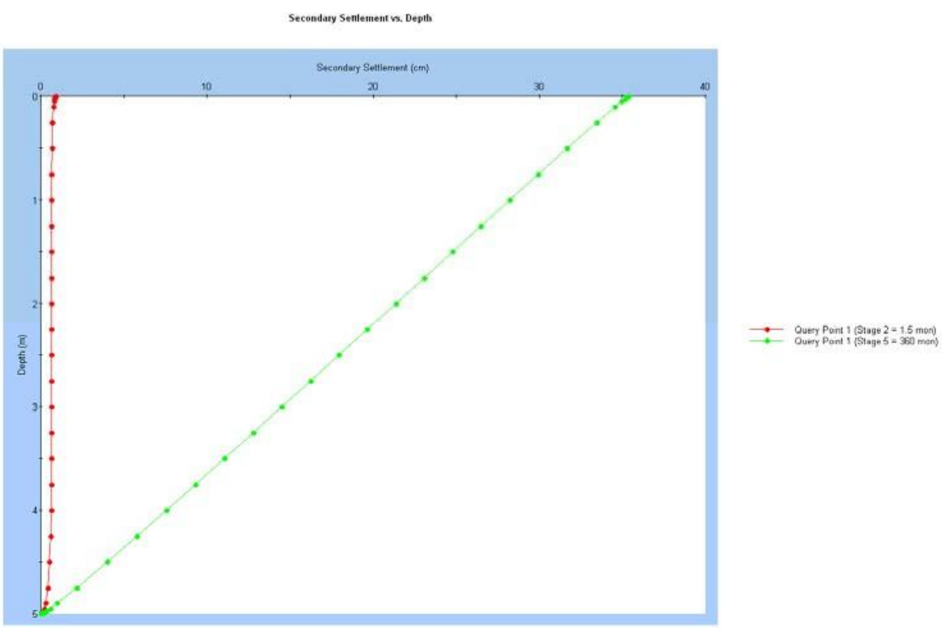


18.2. Results

18.2.1. Part 1: no surcharge

For the case when the load is applied without surcharge, the example problem states that the end of primary consolidation is reached in 6 weeks and the total amount of secondary consolidation after 30 years is 35 cm. As mentioned above, the permeability of the *Settle3* model was adjusted to match this result. The excess pore pressure and secondary consolidation at 6 weeks and 30 years are shown below. Unlike the example in Mesri et al. (1997), we cannot say that all primary consolidation finishes at 6 weeks, since some parts of the soil consolidate faster than others. However, you can see that at 6 weeks, the excess pore pressure has mostly dissipated and that secondary consolidation is starting near the top and bottom of the soil layer where there is drainage. The secondary settlement in the *Settle3* model after 30 years is 35.4 cm.

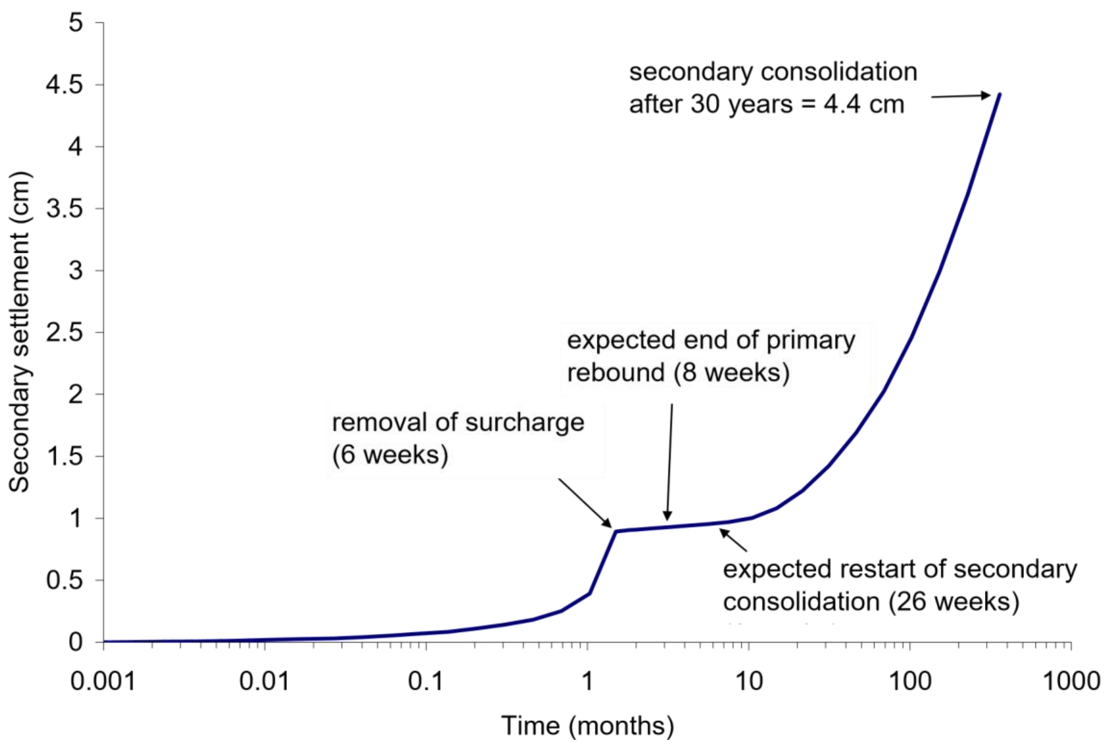




18.2.2. Part 2: Surcharge

Mesri et al. show that when a surcharge of 120 kPa is applied, the post surcharge secondary settlement is reduced to 4 cm. For this example, we will apply a load of 120 kPa and then reduce it to 60 kPa at 6 weeks (assumed to be the end of primary consolidation). Mesri et al. state that primary rebound should finish in two weeks. There is then a delay of 20 weeks (after the removal of surcharge) before the start of secondary settlement. The post-surcharge secondary settlement after 30 years should then be 4 cm. The results from *Settle3* are shown below. You can see that the calculated secondary consolidation of 4.4 cm is close to the value of 4 cm calculated by Mesri et al. If we only consider the post-surcharge secondary consolidation, then *Settle3* yields a value of 3.5 cm.

The discrepancies are due to the fact that the analysis of Mesri et al. assumes that the entire layer finishes consolidating at the same time, whereas in fact, the top and bottom parts of the soil near the drained boundaries finish consolidating first and the centre of the layer finishes consolidating later. This also explains the small amount of secondary consolidation observed in the below graph between the time of surcharge removal and the expected start of secondary consolidation.



18.3. References

1. Mesri, G., Stark, T.D., Ajlouni, M.A. and Chen, C.S., 1997. Secondary compression of peat with or without surcharging, *Journal of Geotechnical and Geoenvironmental Engineering*, **123**, 411-421.