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A STUDY OF  
CONSOLIDATION OF COHESIVE SOILS

by

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## INTRODUCTION

Settlement is caused by two different types of behavior within the soil. The first type includes cases in which shear stresses exceed the shear strength of the soil. When this happens, the soil fails by sliding downward and laterally along a shear plane within the soil. The second type includes cases in which the shear strength of the soil is not exceeded, but the vertical compressive strains cause settlements within the soil mass. Settlement caused by this second type of soil behavior will be dealt with in this report.

Settlement of a structure may be tolerable, as long as the settlement is uniform throughout, and does not reach excessive proportions. On the other hand, nonuniform or differential settlements can be disastrous to a building, causing plaster to crack, door and window frames to bind, and floors to crack and fault. In an extreme case, differential settlement can cause cracking of structural members, which can lead to an eventual collapse of the building.

The settlement of a structure overlying a layer of some cohesive material, may take place slowly, and be of large magnitude. These settlements once puzzled engineers, because of the long period of time that elapsed between completion of construction, and the appearance of cracking and structural damage. It wasn't until 1919, that Karl Terzaghi(12) made a successful attempt at explaining this phenomena.

By application of Terzaghi's theory, a reasonable estimate can be made of the magnitude and time-rate of settlement. Because of the simplifying assumptions that Terzaghi made, and the variability of soil, settlement calculations can be only estimates of those that will actually take place. Even though the Terzaghi theory provides only an estimate, it is a valuable tool in analyzing the amount and rate of settlement in a large and costly structure.

## PURPOSE OF THE STUDY

The purpose of this report, is to review Terzaghi's theory of consolidation, and to present a method of performing a consolidation test. From this, a hypothetical problem will be presented, and the time-rate and the magnitude of settlement will be computed. Limitations of the theory and comparison to actual settlements will be presented. Hopefully, this report will be of value to practicing foundation engineers in estimating the amount and time-rate of settlement in a structure.

**THIS BOOK  
CONTAINS  
NUMEROUS PAGES  
WITH DIAGRAMS  
THAT ARE CROOKED  
COMPARED TO THE  
REST OF THE  
INFORMATION ON  
THE PAGE.**

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RECEIVED FROM  
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## LITERATURE REVIEW

According to Terzaghi (12): "Every process involving a decrease in the water content of a saturated soil without replacement of the water by air is called a process of consolidation". In noncohesive soils, this consolidation process takes place immediately, after a load is applied, because the water can travel rapidly through the coarse, granular material. In contrast to this, when a load is applied to a cohesive soil, it takes the water a long time to travel through the fine-grained soil. This consolidation characteristic of cohesive vs. noncohesive soils is shown in Figure 1. In a noncohesive soil, the settlement is almost completed at the end of the time of construction, whereas full settlement on a cohesive soil is reached, a long time after the completion of construction.

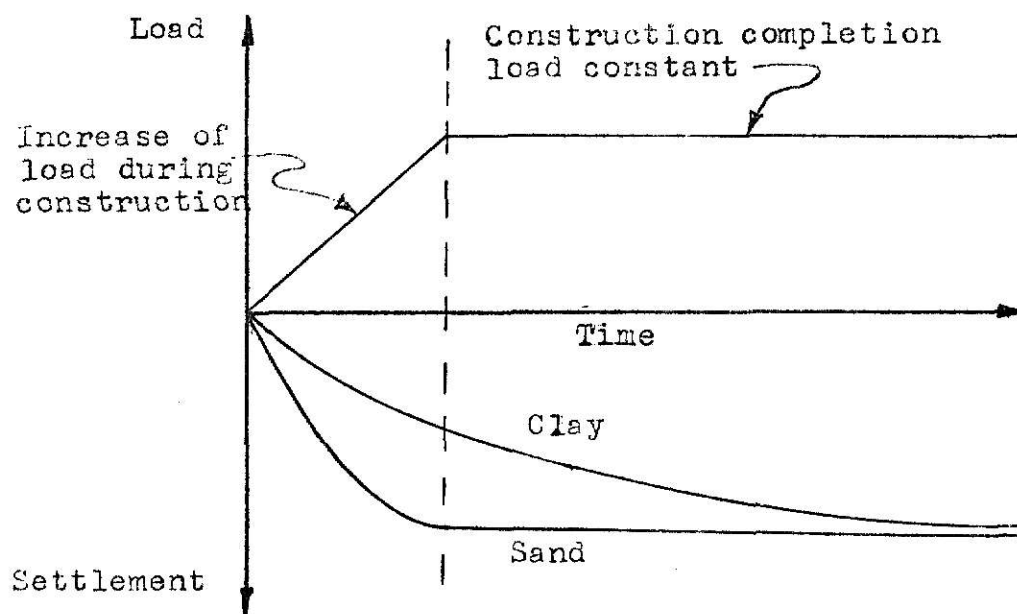


Figure 1

Because noncohesive soils consolidate almost immediately, the discussion of consolidation and compression characteristics of soils, in this paper, will only deal with cohesive soils.

The engineer is primarily concerned with settlement, and is usually interested in one-dimensional, vertical compression of soil under structures that have foundations on or near the surface of the ground. Under normal loading conditions, it is generally assumed that there is very little lateral displacement of the soil, and that the soil undergoes a change in thickness only (3). A device called a consolidometer, is used in the laboratory to simulate a condition of no lateral confinement which is assumed to occur in the field.

During a consolidation test, the soil specimen is confined by a metal ring. Porous stones are placed on the top and the bottom of the specimen, so that air and water can flow out of the soil, without loss of the soil particles. The load is applied on a yoke across the top of the device. The change in thickness is measured by a centrally located dial gage. Figure 2 shows a typical consolidometer.

The load during a consolidation test, is applied in increments. After each increment of load is applied, the load is held constant, until compression has stopped. In some clays, it takes several hours for the compression to



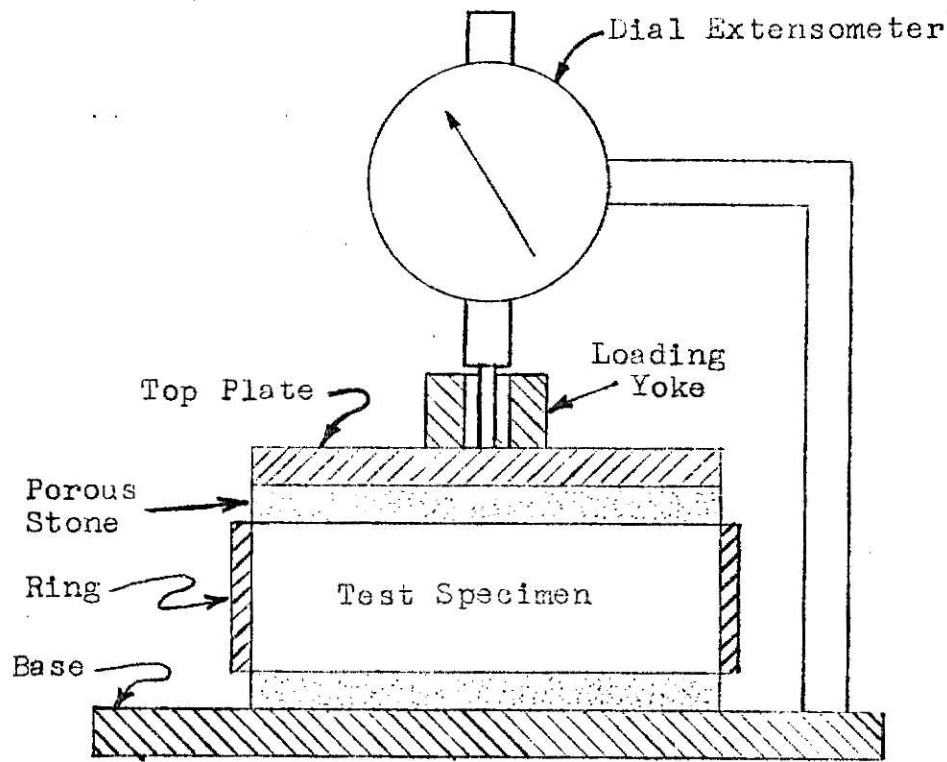


Figure 2

stop, because the compression can only occur as fast as the water is forced from the voids. When the compression has stopped, or is only occurring at a very small rate, the next load increment is applied, and the procedure is repeated.

The results of a consolidation test, are plotted on a graph, with the final void ratio corresponding to each load increment on the ordinate, and accumulated pressure to a logarithmic scale on the abscissa. The void ratio,  $e$ , is equal to the volume of the voids, divided by the volume of the solids,

$$e = \frac{V_v}{V_s} \text{ --- (1)}$$

The resulting diagram is known as an e-Log P curve. A typical e-Log P curve is shown in Figure 3.

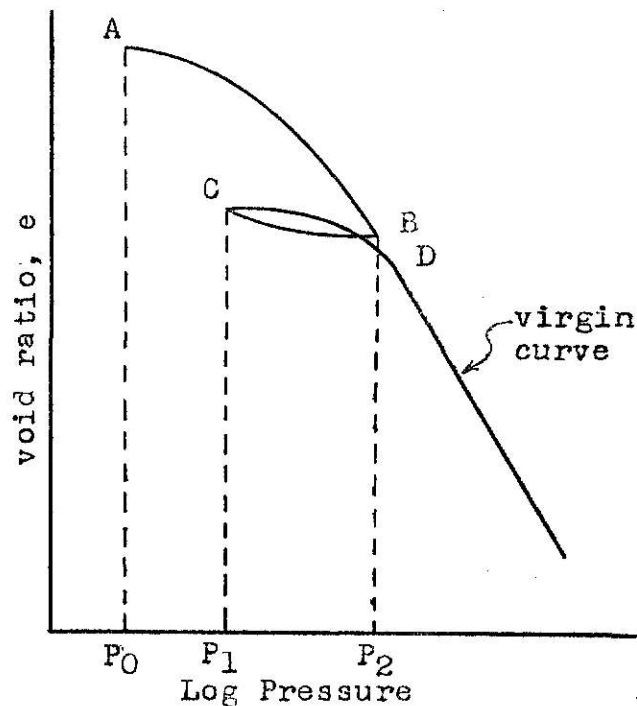


Figure 3

By looking at Figure 3, the curve starts at point A, and is concave downward, but it almost approaches a straight line as it approaches point B. The nearly straight part of the curve is known as the virgin curve. If the loading is stopped at point B, and unloaded in increments, the soil swells, along the unloading curve BC. When loading is resumed, the reloading curve CD approaches the virgin curve at point D.

When the sample is at a state represented by point C, it is preconsolidated, and the pressure,  $P_2$ , is known as the preconsolidation pressure. If the overburden pressure of a soil sample in nature is the same as the preconsolidation load as determined in the laboratory, the sample is normally consolidated. If the preconsolidation load is greater than the in-place overburden pressure, the sample is said to be preconsolidated. When the preconsolidation load exceeds the weight of the present overburden, it has been shown(14), that the excess weight corresponds to the weight of soil layers removed by erosion, a building being removed, or the weight of glaciers which loaded the soil during past glaciations.

In 1932, Arthur Casagrande (1) proposed an empirical method for the graphical determination of the preconsolidation load. According to this method, a tangent KH is drawn at the point of greatest curvature, A, of the e-Log P curve. A line AC is drawn, so that it bisects the angle formed by the tangent AH, and the horizontal line AG. The straight section of DE, of the e-Log P curve is extended until it intersects point B on line AC. The abscissa of point B is the preconsolidation pressure. Casagrande's method is illustrated in Figure 4.

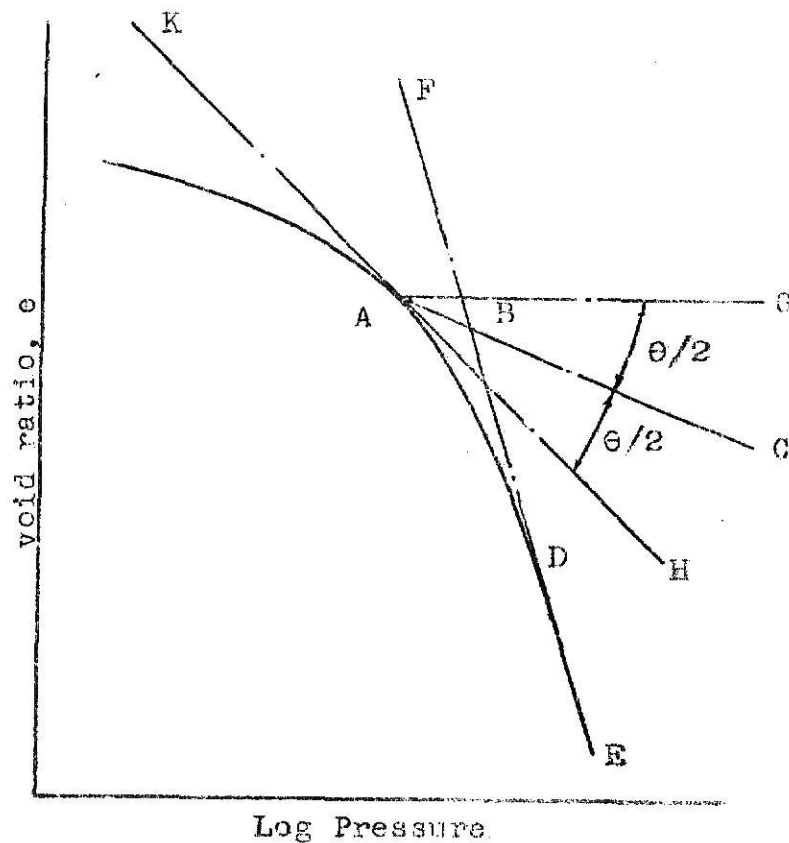


Figure 4

The slope of the straight line portion, or virgin curve portion of an  $e$ -Log  $P$  curve, is known as the Compression Index,  $C_c$ , and is defined by the equation

$$C_c = \frac{e_0 - e_1}{\text{Log}_{10} P_1 - \text{Log}_{10} P_0} = \frac{\Delta e}{\text{Log}_{10} (P_1/P_0)} \quad (2)$$

in which

$e_0$  = known void ratio at  $P_0$

$e_1$  = known void ratio at  $P_1$

$P_0$  = pressure corresponding to  $e_0$

$P_1$  = pressure corresponding to  $e_1$

An approximate relationship between Liquid Limit (LL) of a clay soil, and the Compression Index ( $C_c$ ), was suggested by Skempton (8) in the following formula for normally consolidated clays.

$$C_c = 0.009(LL-10) \text{ - - - - - (3)}$$

Another approximate expression for  $C_c$  was suggested by Hough (3), which uses the in-place void ratio

$$C_c = 0.30(e_0 - 0.27) \text{ - - - - - (4)}$$

In which  $e_0$  is the in place void ratio. Formula (4), can be used for inorganic silty clay or clay. Both of these empirical formulas can prove to be very useful, because they permit the engineer to estimate settlement, if no consolidation tests can be run.

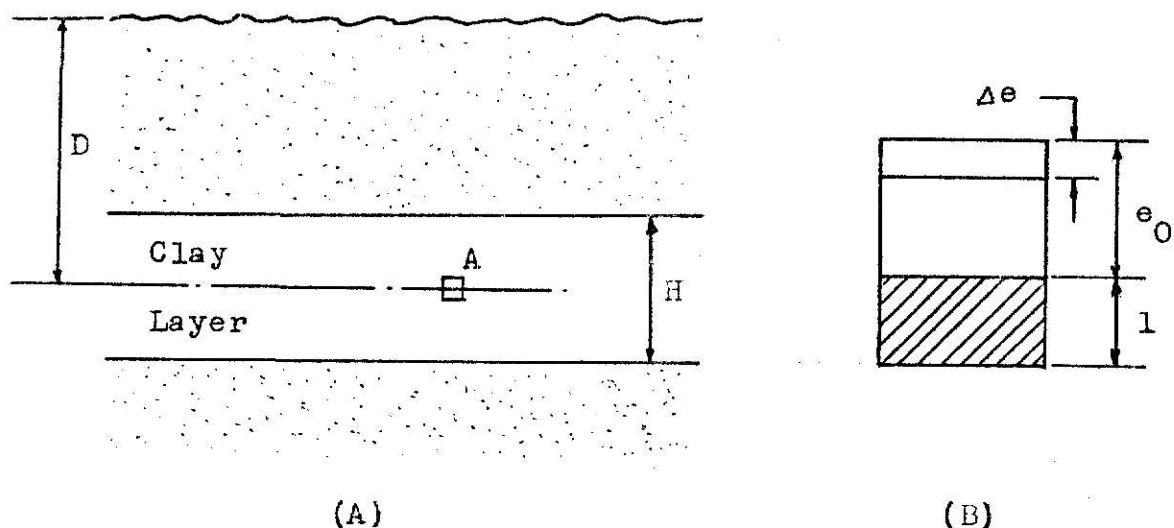


Figure 5

In Figure 5A, a cross section of a clay layer with thickness  $H$ , depth  $D$ ,  $D$  is the distance from the ground surface to the middle of the clay layer. The original pressure at point A is  $P_0$ , and the increase in pressure after loading is  $\Delta P$ . The original void ratio is  $e_0$ , and the change in void ratio due to consolidation is  $\Delta e$ . Figure 5B, shows a prismatic element containing point A.

In Figure 5B, the height of the solid material is  $1$ , and the height of the voids initially is  $e_0$ . The total height of the element is  $1+e_0$ . By proportion,

$$\frac{S}{H} = \frac{\Delta e}{1 + e_0} \quad \text{where } S = \text{settlement, or change in thickness of the clay layer.}$$

Solving for  $S$ ,

$$S = H \frac{\Delta e}{1 + e_0} \quad \text{--- (5)}$$

This equation can be used to calculate the amount of settlement when the original void ratio, and the change in void ratio are known. Referring to Equation (2),

$$C_c = \frac{\Delta e}{\log P_1/P_0},$$

solving for  $\Delta e$ ,

$$\Delta e = C_c \log P_1/P_0 = C_c \log \frac{P_0 + \Delta P}{P_0}.$$

Substituting for  $\Delta e$  in Equation (5),

$$S = \frac{C_c}{1 + e_0} H \text{ Log } \left( \frac{P_0 + \Delta P}{P_0} \right) \text{ - - - - - (6)}$$

Equation (6), can be used when the Compression Index,  $C_c$ , is known, the overburden pressure,  $P_0$ , is known, and the increase in pressure  $\Delta P$  is known.

When using Equation (6), care must be taken when selecting where along the e-Log P curve the Compression Index should be taken, when dealing with a preconsolidated soil. If  $C_c$  is taken on the steeper portion of the e-Log P curve, and the actual load applied to the soil is lower than the preconsolidation load, settlement calculations using Equation (6), will be greater than those that will actually occur (2).

The total stress applied to a soil mass, is  $P$ . During the consolidation process, some of this total pressure is taken up by the pore water, and is called the neutral stress,  $u$ . The rest of the total stress is transferred intergranularly between soil particles, and is termed effective stress,  $P_e$ . In equation form,

$$P = P_e + u \text{ - - - - - (7)}$$

When a static load is applied to a soil mass, the pore water is stressed, because the load application occurs faster than the pore water can escape the voids. When this occurs, the water in the voids is under excess hydrostatic pressure.

The progress of the consolidation process, can be expressed by the amount of hydrostatic excess after a period of time, relative to the amount of hydrostatic excess at the instant the load is applied. This leads to the following equation:

$$u\% = \frac{u_1 - u}{u_1} \text{ - - - - - (8)}$$

Where

$u\%$  = Percent consolidation

$u_1$  = Initial hydrostatic excess

$u$  = Hydrostatic excess at time,  $t$ , after application of load.

For a particular application of load increment,  $u_1$  is constant, while  $u$  is variable with time. At the instant the load increment is applied,  $u = u_1$ , therefore the percent consolidation is zero. After a period of time,  $u$  gradually decreases until finally it reaches zero. At this point, consolidation is 100%. Terzaghi's theory of consolidation was developed to obtain equations for  $u$  as functions of time and space.

The percent consolidation varies in relation to the distance from a drainage face. Thus a point in a consolidating layer, near a drainage face consolidates more rapidly than a point located farther from the drainage face. In settlement calculations, the engineer need only be con-



cerned with the average percent consolidation of a layer, which will be designated as U.

The time-rate of consolidation depends on these four factors (10):

- 1) thickness of the consolidating layer;
- 2) number of drainage faces;
- 3) permeability of the soil;
- 4) magnitude of the consolidating pressure acting on the compressible layer.

The following general equation, which was developed by Terzaghi(13) applies for all clays, even though there are many variables effecting the rate of consolidation.

The value of C, the coefficient of consolidation is,

$$C = \frac{K(1 + e)}{a \gamma_w} \text{ - - - - - (9)}$$

Where

K = Coefficient of permeability

e = Initial void ratio under an increment of loading

a = Coefficient of compressibility

$\gamma_w$  = Unit weight of water

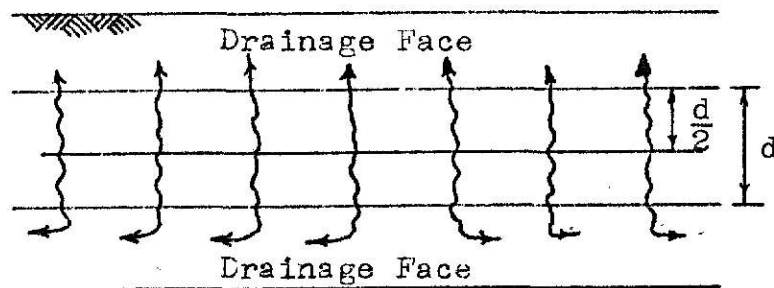
The coefficient of compressibility, a, is the slope of the void ratio vs. pressure curve at the value  $e_{ave.}$ , which is,

$$e_{ave.} = \frac{e_1 + e_2}{2} \text{ - - - - - (10)}$$

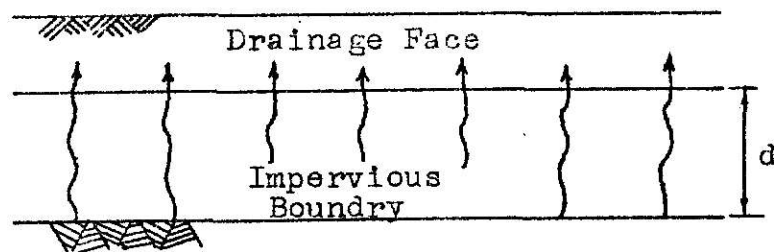
The coefficient of compressibility is defined by the following equation:

$$a = \frac{\Delta e}{\Delta P} \text{ --- (11)}$$

The farthest distance that the pore water has to travel to reach a drainage face, is known as the longest drainage path,  $h$ . When a clay layer has drainage on both the top and bottom, it is said to have double drainage. These two different drainage conditions are shown in Figure 6.



A) Double Drainage,  $h = \frac{d}{2}$



B) Single Drainage,  $h = d$

Figure 6

The Coefficient of Consolidation,  $C$ , and the longest drainage path,  $h$ , were used by Terzaghi, to establish a parameter called a time factor,  $T$ .  $T$  is defined as:

$$T = \frac{Ct}{h^2} \text{ - - - - - (12)}$$

or, solving for  $t$ ,

$$t = \frac{Th^2}{C} \text{ - - - - - (13)}$$

Where

$T$  = Time factor

$C$  = Coefficient of Consolidation

$t$  = Time

$h$  = Longest drainage path

$T$  is a dimensionless number, because  $C$  is expressed in units of  $(\text{Length})^2/(\text{Time})$ ,  $t$  in units of Time, and  $h$  in units of Length. Since  $T$  is dimensionless, a relationship between  $U$  and  $T$  is general, and can be used for any clay layer.

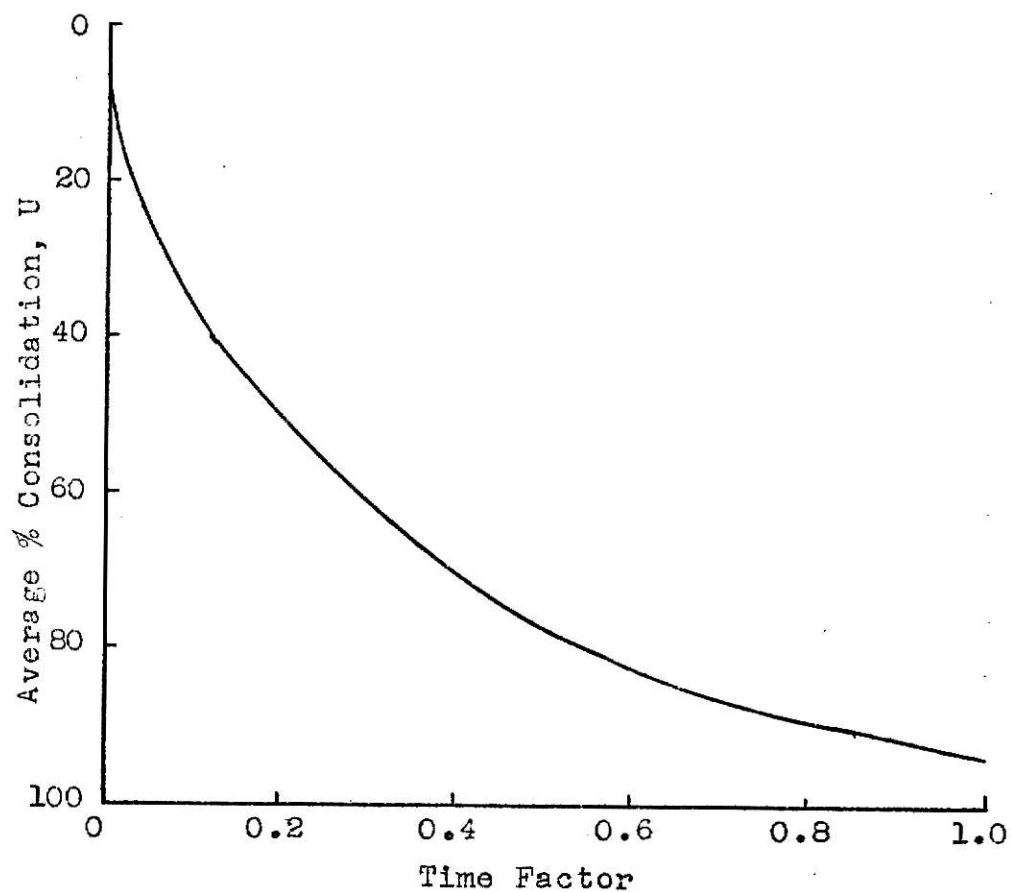
The basic relation for average percent consolidation,  $U$ , and time factor,  $T$ , is

$$U = 1 - \frac{3}{\pi^2} (e^{-n} + \frac{1}{9} e^{-9n} + \frac{1}{25} e^{-25n} + \dots) \text{ - - - - (14)}$$

Where,  $n = \frac{\pi^2}{4T}$

This relation is a mathematical series, and is more convenient to represent in the form of a graph of  $U$  vs.  $T$ .

Instead of solving Equation (14), each time a factor is needed, the value may be read from the graph or table in Figure 7.



U	T
10	0.008
20	0.031
30	0.071
40	0.126
50	0.196
60	0.287
70	0.403
80	0.567
90	0.848
100	$\infty$

Figure 7

The coefficient of permeability,  $K$ , is required when using equation (11) to determine the coefficient of consolidation. It is very difficult and time consuming to measure the coefficient of permeability while conducting the consolidation test. Taylor(11) devised an empirical method of determining the coefficient of consolidation on the basis of information obtained in the consolidation tests. Taylor's method is called the "square root of time fitting method".

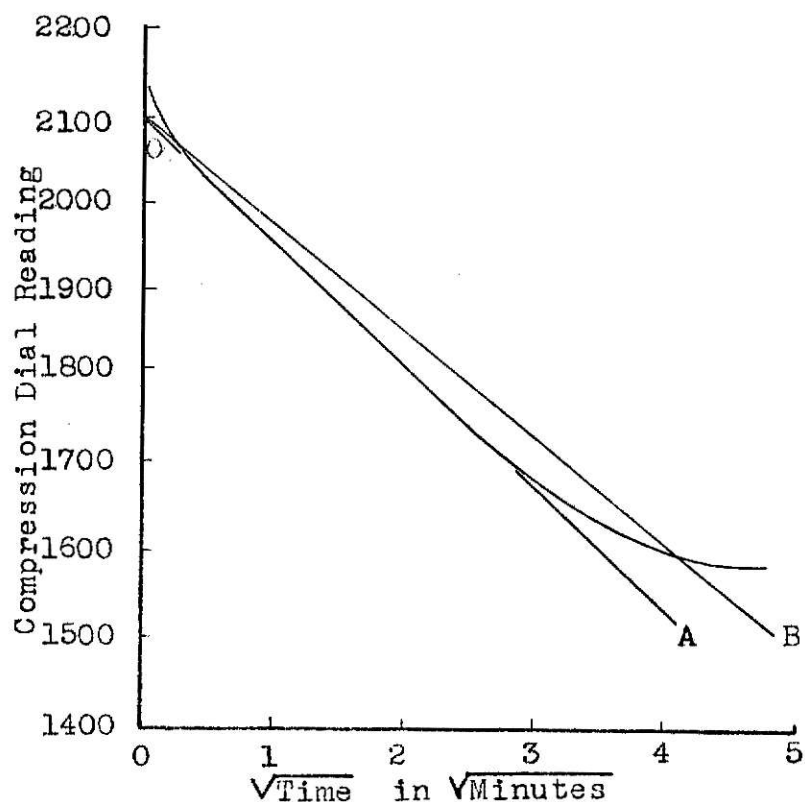


Figure 8

In the square root of time fitting method, all dial readings for each increment of load are plotted of the ordinate, and the corresponding square roots of time on the abscissa, as shown in Figure 8.

First, draw a straight line OA, which best fits the early part of the experimental curve. The intersection of this line with the ordinate is the corrected point of zero consolidation. Next, draw another straight line OB, through the corrected zero point, with each point on the second line having an abscissa 1.15 times those on OA. The intersection of line OB, and the curve, is taken as the point at which 90% of consolidation has occurred. The time value at 90% consolidation is read from the abscissa, and is known as  $t_{90}$ . From Equation (12), and solving for C,

$$C = \frac{Th^2}{t}$$

T for 90% consolidation from Figure 7 is 0.848.

Therefore,

$$C = \frac{0.848 h^2}{t_{90}} \text{ - - - - - (15)}$$

After substituting the experimental value for  $t_{90}$  and the length of the longest drainage path for h, the coefficient of consolidation can be determined for each load increment.

Another empirical method to determine the coefficient of consolidation without knowing the coefficient of permeability, was developed by Casagrande (7), and is shown in Figure 9. When the compression dial readings are plotted against the log of time, the curve usually has a straight line portion in the later loading phase. These two straight lines, when extended, intersect at 100% consolidation. To find the point of 0% consolidation, select a time  $t_1$  where less than half of the consolidation has taken place for that load increment. Locate the point C corresponding to  $t_1$ , and then find a point D corresponding to  $t_1/4$ . Make a horizontal line above D, the same distance as C is below D. This line corresponds to 0% consolidation. The 50% consolidation point is half way between the corrected zero point, and the point of 100% compression. The time value at 50% consolidation is read off of the abscissa, and is known as  $t_{50}$ . Since,

$$C = \frac{Th^2}{t}$$

and T for 50% consolidation from Figure 7 is 0.196,

$$C = \frac{0.196h^2}{t_{50}} \text{ - - - - - (16)}$$

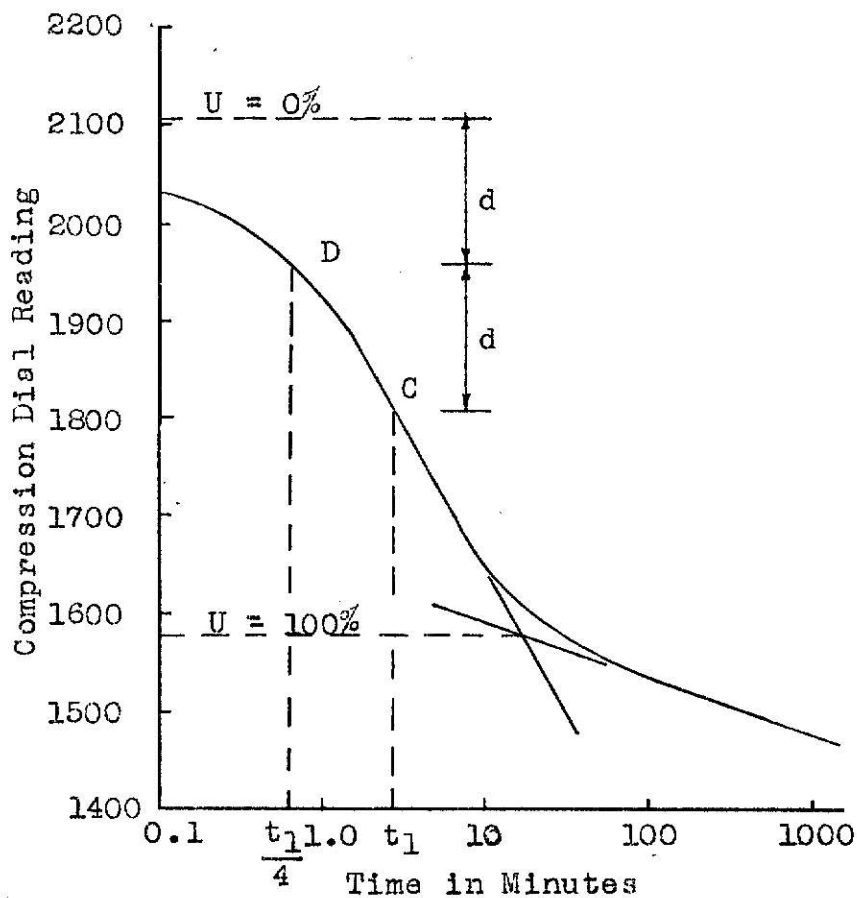


Figure 9

The empirical time-fitting method for determining the coefficient of consolidation, must be found by trial. For some clays, the square root of time fitting method works best, and in other clays the log time method works better. In some clays, both methods can be used(11).

The coefficient of consolidation, should be determined for each load increment, by one of the time fitting methods previously described. Then, an average value of the coefficient of consolidation over the range of loads a particular structure will impose on a clay



layer should be used in making settlement computations.

The reliability of the Terzaghi Consolidation Theory, can be determined by comparing theoretical settlement calculations to actual measurements in the field. Studies by Spangler (9) were made on the Ft. Randall dam embankment in South Dakota. From these studies, the general order of the estimated settlements, compared favorably with the measured settlements. The estimated rates of settlement however, did not compare as well with the measured rates. Observations by MacDonald and Skempton (5) of settlements in 20 buildings showed a variance of -27 to 57 percent, with an average error of 5 percent between actual and computed settlements. The rate of consolidation, was found to be more rapid, than the Terzaghi Theory indicated.

## HYPOTHETICAL PROBLEM

To illustrate the use of the Terzaghi Consolidation Theory, a hypothetical problem will be presented. Calculations will be made on the building that has a foundation plan as shown in Figure 10.

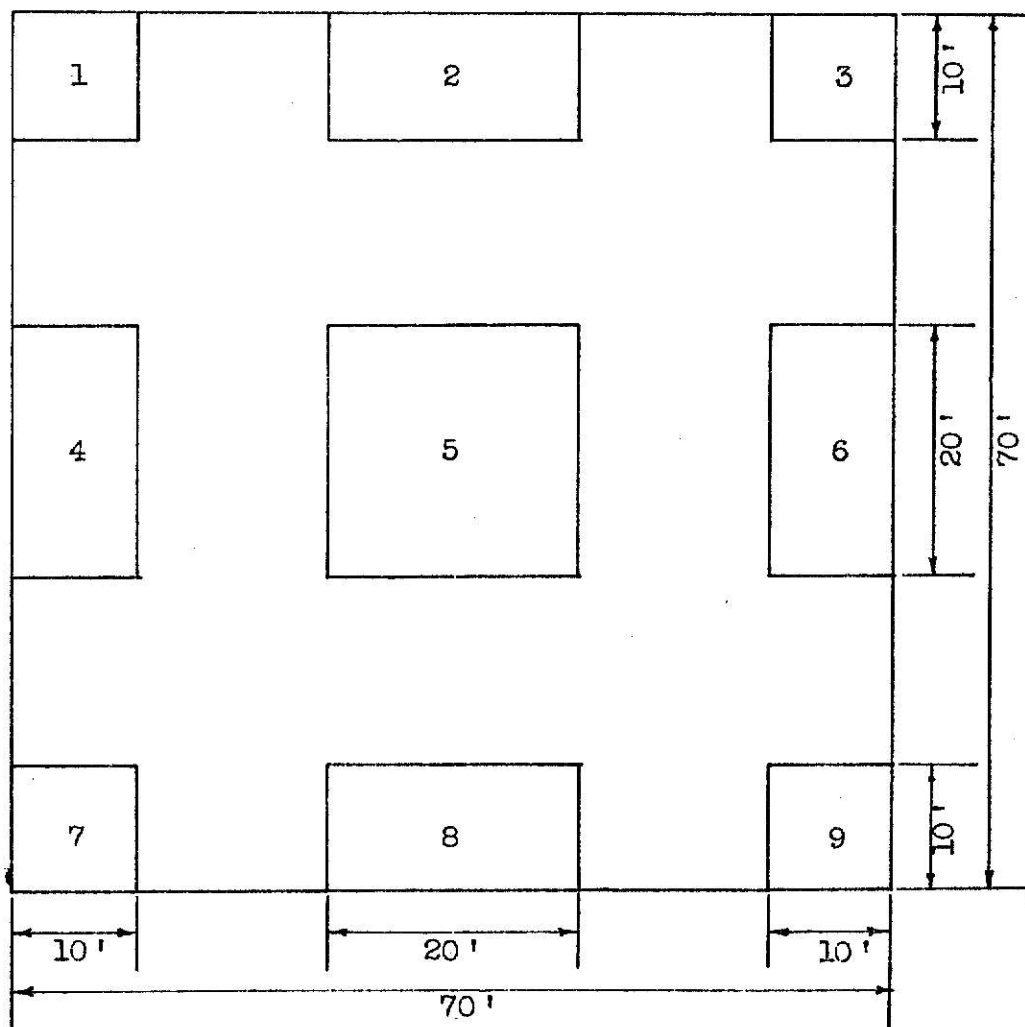


Figure 10

The soil profile the building is founded on is shown in Figure 11.

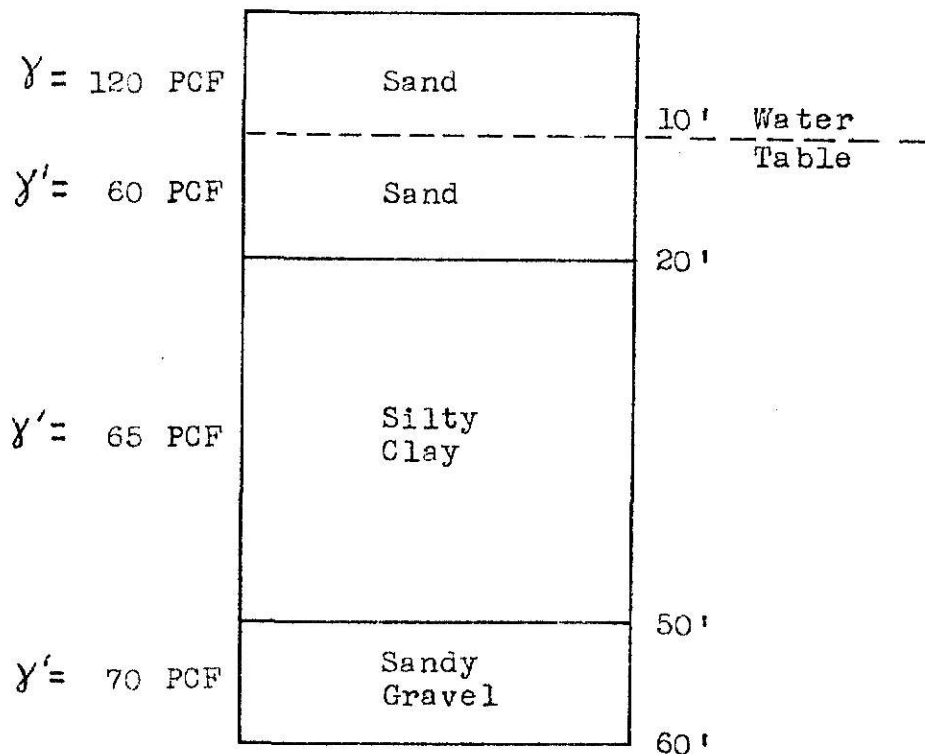


Figure 11

The footings are loaded as follows: Corners 1, 3, 7, and 9, 300 tons; Sides 2, 4, 6, and 8, 600 tons; Center 5, 1200 tons. When these loads are divided by their respective footing areas, it is found, that the pressure beneath each footing is the same, at 3 tons/sq. ft. The bottom of the footings are located at a depth of 10 ft.

The results of a consolidation test(10) performed on the silty clay layer of Figure 11, are presented in the table in Figure 12.

Figure 12

		Load Increments (TSF) and Compression Dial Readings (0.0001 in.)									
		0.1-0.2	0.2-0.5	0.5-1.0	1.0-2.0	2.0-4.0	4.0-8.0	8.0-16.0			
t	t	Dial Dif.	Dial Dif.	Dial Dif.	Dial Dif.	Dial Dif.	Dial Dif.	Dial Dif.			
0	0	4896	0	4850	0	4761	0	4478	0	3793	0
0.25	0.5	4892	4	4842	8	4742	19	4400	78	3723	70
1	1	4889	7	4836	14	4728	33	4343	135	3672	121
2.25	1.5	4886	10	4830	20	4714	47	4296	182	3621	172
4	2	4883	13	4825	25	4700	61	4230	248	3570	223
6.25	2.5	4880	16	4819	31	4686	75	4173	305	3518	275
9	3	4877	19	4813	37	4673	88	4116	362	3468	325
12.25	3.5	4874	22	4807	43	4659	102	4059	419	3417	376
16	4	4871	25	4801	49	4645	116	4011	467	3374	419
25	5	4867	29	4794	56	4627	134	3946	532	3308	485
36	6	4864	32	4788	62	4614	147	3900	578	3259	534
49	7	4863	33	4785	65	4607	154	3880	598	3228	565
64	8	4862	34	4783	67	4603	158	3872	606	3207	586
100	10	4861	35	4781	69	4597	164	3860	618	3171	622
225	-	4858	38	4777	73	4587	174	3848	630	3124	669
400	-	4853	43	4767	83	4548	213	3821	657	3034	759
1440	-	4850	46	4761	89	4478	283	3793	685	2954	839
										2332	622
										1762	570

First of all, the void ratio is computed for the initial pressure of each increment of load. From Equation (1),

$$e = \frac{V_s}{V_v} = \frac{A (H - H_s)}{A H_s}$$

And since the area of the consolidation test samples is the same, the area term cancels

$$e = \frac{H - H_s}{H_s} \text{ ----- (17)}$$

where

$e$  = void ratio

$H$  = specimen height

$H_s$  = height of solids

$$H_s = \frac{W_s}{G \gamma_w A} \text{ ----- (18)}$$

where

$W_s$  = weight of the solids

$G$  = Specific Gravity

$\gamma_w$  = unit weight of water

$A$  = area of sample

From Equations (17) and (18), the void ratio can be calculated from the height of the specimen,  $H$ , the Specific Gravity,  $G$ , and the dial readings from the consolidation test. The void ratios for the corresponding pressure increments are tabulated in the table in Figure 13.

Load Increment (tsf)	Initial Void Ratio	Sample Height (in.)	Max. Flow Path	$\sqrt{t_{90}}$	$t_{90}$	Coeff. of Con. $C_c$
0.1 to 0.2	0.915	1.4226	0.7113	5.75	33.1	47.3
0.2 to 0.5	0.909	1.4159	0.7080	5.9	34.8	44.6
0.5 to 1	0.897	1.3972	0.6986	5.9	34.8	43.4
1 to 2	0.859	1.3489	0.6744	5.6	31.4	44.8
2 to 4	0.767	1.2727	0.6364	5.65	31.9	39.4
4 to 8	0.654	1.1996	0.5998	5.75	33.1	33.4
8 to 16	0.570	1.1400	0.5700	5.7	32.5	30.9
16	0.494					

Figure 13

After obtaining the void ratios for the corresponding pressure increments, an e-Log P curve can be drawn, and the Compression Index,  $C_c$ , determined, as shown in Figure 14.

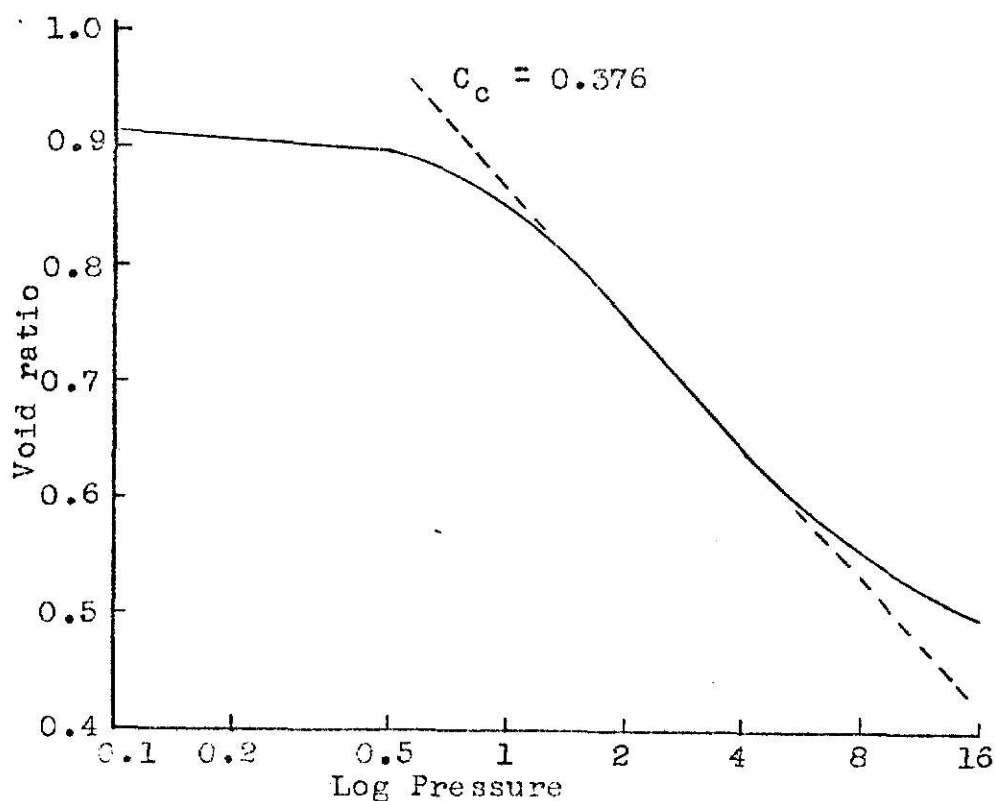


Figure 14

The next step is to determine the Coefficient of Consolidation,  $C$ . The square root of time fitting method works well with this silty clay, and will be used to determine the Coefficient of Consolidation. Figure 15, shows the square root of time fitting method used to determine  $C$ , for the load increments from 1 to 2 and 2 to 4 tons/sq. ft.

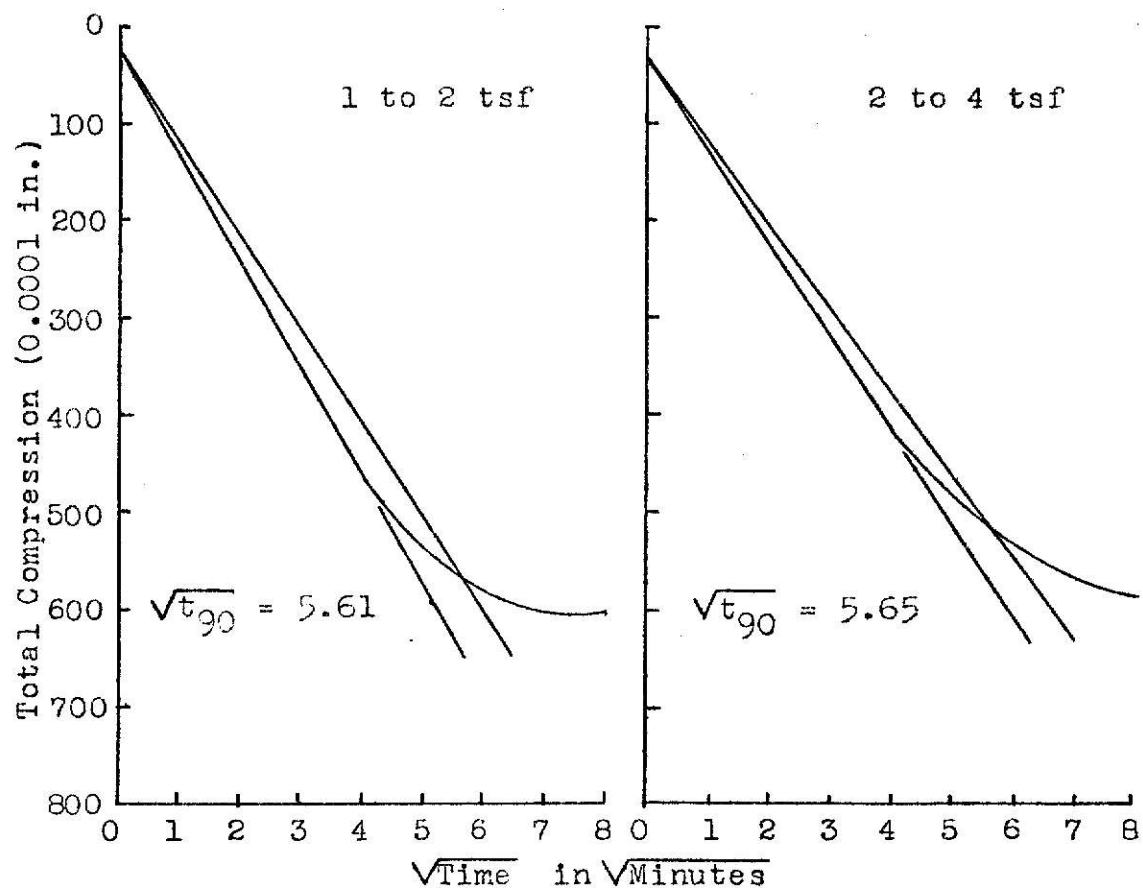


Figure 15

Now, the initial overburden pressure,  $P_0$ , will be calculated at the mid-height of the silty clay layer. Since only intergranular pressures cause compression of a soil mass, and not the hydrostatic pressure, the submerged unit weight ( $\gamma'$ ) is used to calculate the initial overburden pressure below the water table. Therefore, by multiplying the thickness of the layers, times their respective unit weights,  $P_0$  can be obtained.

$$(10) (120) = 1200 \text{ PSF}$$

$$(10) (60) = 600 \text{ PSF}$$

$$(15) (65) = \underline{975 \text{ PSF}}$$

$$P_0 = 2775 \text{ PSF}$$

The next step is to determine  $\Delta P$ , the net change in pressure due to excavation and loading. This change in pressure, will be computed with the aid of a Newmark Influence Chart (6). To use the chart, the footing is drawn to a scale so that the distance XY, equals the distance from the bottom of the footing, to the center of the compressible silty clay layer. The footing is drawn on the chart, so that the center of the chart corresponds to the point where the change in pressure is desired. The squares inside the footing plan are then counted, and multiplied by the influence value for the chart, this value is multiplied by the footing pressure, and the result is the change in pressure at the point in question.



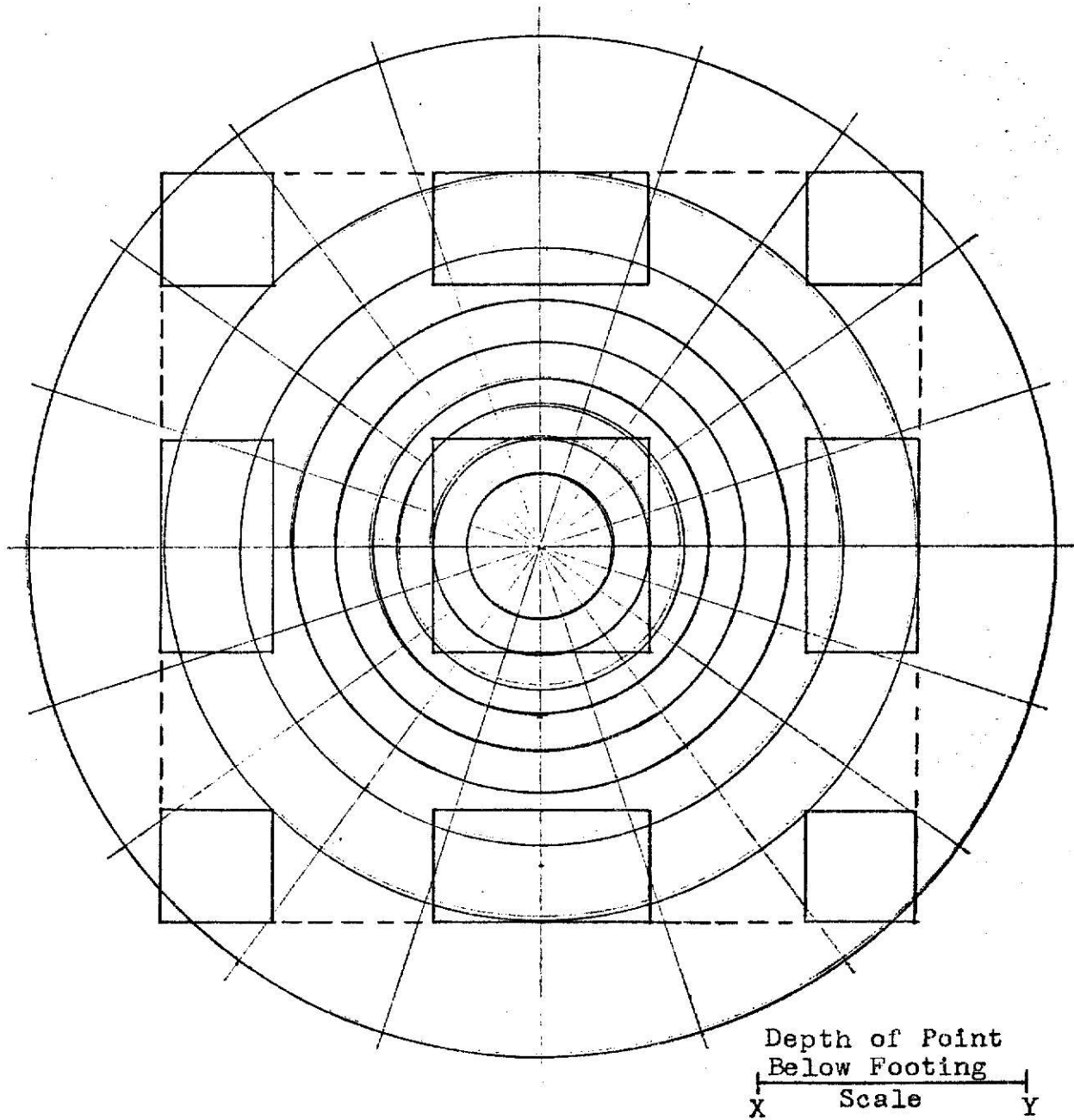


Figure 16

Figure 16, shows the footing plan drawn over the Newmark Chart, to determine the change in pressure below the center footing, Number 5. The number of squares covered by the excavation outline (dashed line), is 127. This number multiplied by the influence value of 0.005, and by the weight of excavation per square foot, gives the change in pressure due to excavation,

$$(127) (0.005) (10) (120) = 762 \text{ PSF.}$$

The number of squares covered by the footing outline is 49, therefore the change in pressure due to the footing load is,

$$(49) (0.005) (6000) = 1470 \text{ PSF.}$$

The net total change in pressure due to footing pressure minus the change in pressure due to the excavation.

$$\Delta P = 1470 - 762 = 708 \text{ PSF.}$$

From Figure 14, at  $P_0 = 2775 \text{ PSF}$ ,  $e_0 = 0.68$ ,  $C_c = 0.376$ . The thickness of the silty clay layer,  $H$ , is 30 ft. The settlement for center footing number 5 can now be computed from Equation (6).

$$S = \frac{C_c}{1 + e_0} H \text{ Log } \frac{P_0 + \Delta P}{P_0}$$

$$S = \frac{0.376}{1 + 0.68} (30) \text{ Log } \frac{2775 + 708}{2775} = 0.66 \text{ ft.} = 7.9 \text{ in.}$$

Now, the time-rate of settlement will be calculated, and plotted on a time-settlement curve. In the problem, the compressible silty clay layer lies between two permeable, sandy layers. The silty clay layer can drain to the upper and lower layers, and the longest distance the pore water has to travel to escape, is only one-half the thickness of the layer. The pressure that the layer is subjected to is approximately 2 tons/sq. ft., so the Coefficient of Consolidation used, will be the average of the Coefficients obtained from the increments from 1 to 2 TSF, and 2 to 4 TSF. From the table in Figure 13,

$$C = \frac{(44.8 + 39.4)}{2} = 42.1 \frac{\text{sq. ft.}}{\text{year}}.$$

From Equation (13),

$$t = \frac{T_h^2}{C},$$

the time it takes for a certain percentage of consolidation to occur can be computed. For  $U = 20\%$  (20% consolidation),

$$S_{20\%} = 0.20 (7.9) = 1.58 \text{ in.}$$

$$h = 15 \text{ ft.}, T = 0.031, \text{ and } C = 42.1 \text{ sq. ft./year.}$$

$$t_{20\%} = \frac{(0.031) (15)^2}{42.1} = 0.166 \text{ year.}$$

The results of computations for other values of percent consolidation,  $U$ , are tabulated in the table in Figure 17, and are also shown on a graph of time,  $t$ , vs. settlement,  $S$ , in Figure 18.

Values of $U$ (%)	Time Factor ( $T$ )	Settlement (in)	Time (yr)
0	0	0	0
10	0.008	0.8	0.043
20	0.031	1.6	0.166
30	0.071	2.4	0.379
40	0.126	3.2	0.673
50	0.197	4.0	1.053
60	0.287	4.7	1.534
70	0.403	5.5	2.154
80	0.567	6.3	3.030
90	0.848	7.1	4.532
100		7.9	

Figure 17

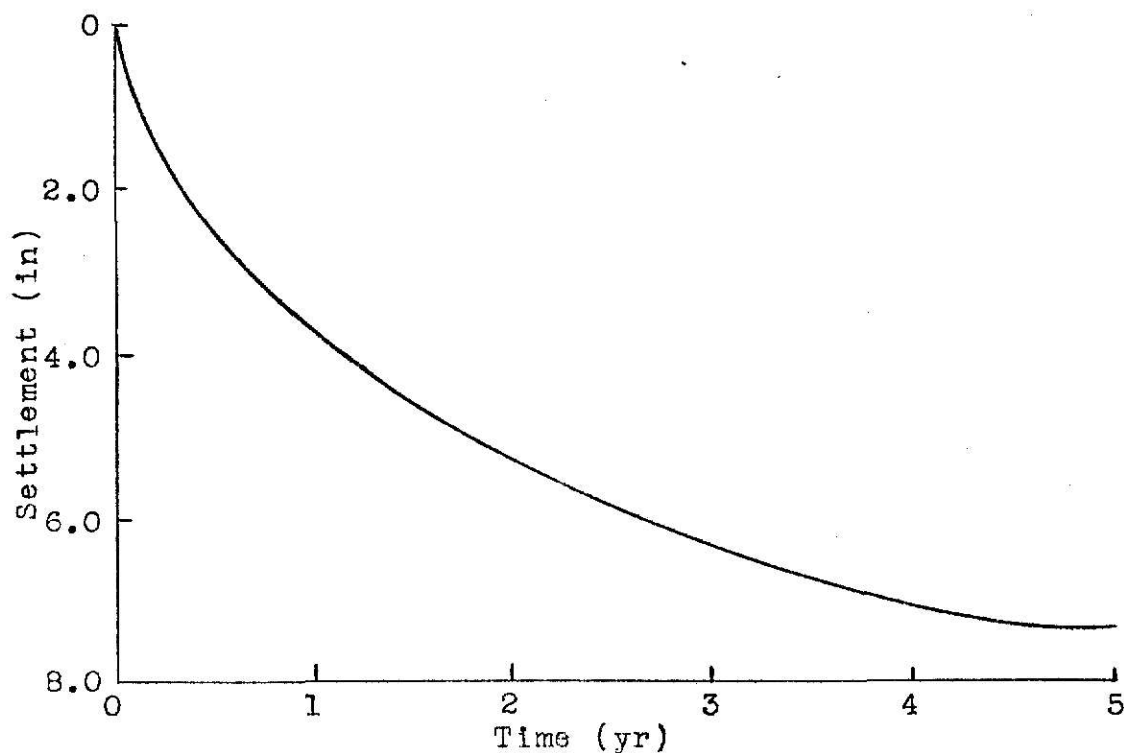


Figure 18

## CONCLUSION

Terzaghi's Consolidation Theory, can be used to estimate the magnitude, and the time rate of settlement of a structure. It is necessary to know, the Coefficient of Consolidation, the Compression Index, the thickness of the compressible clay layer, the number of drainage faces, and the pressure the structure imposes on the clay layer. The Coefficient of Consolidation, and the Compression Index, can be determined in the laboratory from the results of a consolidation test. The thickness of the clay layer, and the number of drainage faces, can be determined from a thorough soil investigation. The load the structure imposes on the clay layer can be calculated with the aid of a Newmark Influence Chart.

Terzaghi's Consolidation Theory, is fairly accurate in estimating the magnitude of settlement, but, the actual rate of settlement, is more rapid than the theory indicates. Even though the Terzaghi Consolidation Theory provides only an estimate, it is a valuable tool in analyzing the amount and rate of settlement in a structure.

## REFERENCES

1. Casagrande, A., "The Structure of Clay and Its Importance in Foundation Engineering," Journal Boston Society of Civil Engineers, April 1932.
2. Casagrande, A., "The Determination of the Preconsolidation Load and Its Practical Significance," Proceedings of the International Conference on Soil Mechanics and Foundation Engineering, Vol. III, 1936.
3. Hough, B. K., Basic Soils Engineering, Ronald Press, New York, 1957.
4. Lambe, T. W., Soil Testing for Engineers, John Wiley and Sons, New York, 1951.
5. MacDonald, D. H. and, Skempton, A. W., "A Survey of Comparisons between Calculated and Observed Settlements of Structures on Clay," Conference on Calculated and Observed Stresses and Displacements, the Institute of Civil Engineers, London, 1955.
6. Newmark, N. M., "Simplified Computation of Vertical Pressures in Elastic Foundations," Circular No. 24, Engineering Experiment Station, University of Illinois, 1935.
7. Peck, R. B., Hanson, W. E., and Thornburn, T. H., Foundation Engineering, John Wiley and Sons, New York, 1974.
8. Skempton, A. W., "Notes on the Compressibility of Clays," Quarterly Journal Geological Society of London 100, 1944.
9. Spangler, M. G., and Griebeling, A. L., "Foundation Settlement in the Ft. Randall Dam Embankment," Highway Research Board, Bulletin 173, 1958.
10. Spangler, M. G., and Handy, R. L., Soil Engineering, 3rd Edition, Intext Educational Publishers, New York, 1973.
11. Taylor, D. W., Fundamentals of Soil Mechanics, John Wiley and Sons, New York, 1943.
12. Terzaghi, K., Theoretical Soil Mechanics, John Wiley and Sons, New York, 1943.
13. Terzaghi, K., and Peck, R. B., Soil Mechanics in Engineering Practice, John Wiley and Sons, New York, 1948.
14. Tschebotarioff, G., Soil Mechanics, Foundations, and Earth Structures, McGraw-Hill, New York, 1951.

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A STUDY OF  
CONSOLIDATION OF COHESIVE SOILS

by

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AN ABSTRACT OF A MASTER'S REPORT  
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## ABSTRACT

Consolidation is defined as a process involving a decrease in the water content of a saturated soil, without replacement of the water by air. In noncohesive soils, this process takes place almost immediately. On the other hand, cohesive soils consolidate over a long period of time. Terzaghi's Consolidation Theory, which is outlined in this report, relates the amount of consolidation of a compressible soil as a function of time.

Comparisons of actual settlements in structures to settlements calculated from the application of Terzaghi's Theory are presented. These comparisons show that the theory fairly accurately predicts the magnitude of settlement, but the rate of consolidation is more rapid than the theory predicts. A method of performing a consolidation test and a hypothetical problem applying the Terzaghi Consolidation Theory are presented.