Kochi Chapter

Indian Geotechnical Conference
IGC 2022
15th – 17th December, 2022, Kochi

A New Equation to Predict the Primary Consolidation Settlement of Clays

Dr.Rajasekhar P

[1] Chithira (H), South Chittoor, Ernakulam, 682027
E-mail: raju58vaikom@gmail.com

Abstract. In Geotechnical Engineering, consolidation of clays and its prediction plays an important role, as the calculation of the possible settlement of any structure is crucial in any foundation design. The calculation of the settlement is now carried out based the theory proposed by Karl Terzaghi in 1936. Over a period of time, many limitations of this theory have been identified. In this paper an attempt is made to derive a new equation for accurately predicting the primary consolidation of clays. As predicted settlement values using conventional method vary much from the actual, most of the previous attempts, were to correct or modify the value of the coefficient of consolidation. However, these corrections have not yielded desirable results to achieve accurate predicted values. The derivation of the conventional equation is revisited and a modification is proposed. The procedure is drawn in similar lines as that of the existing method to retain all the relevant terminologies that have been established in the field. This is done by modifying the relevant coefficients and factors. The newly derived expressions are explicit and direct for predicting the settlement at any stage. An example is also illustrated to demonstrate its validity and applicability.

Keywords: Modified Coefficient of Volume Change, Modified Coefficient of Consolidation, Modified Time Factor, Velocity-Consolidation Coefficient, Settlement Prediction.

1 Introduction

Foundations are one of the most important structural element in any building and the foundation that rests on soil layer transfers the entire load of the building to the ground/earth. In order to keep the building intact for the entire servable life of the building the underlying soil layer also should able to support the building without any subsidence. Of the different types of soils, buildings founded on clayey soils undergo settlement due to consolidation and it shall be within permissible limits. Hence the calculation of long term settlement due to consolidation plays a very vital role in the building design.
It is well established that primary consolidation of clays is due to pore pressure dissipation and the one-dimensional consolidation is represented by the following equation proposed by Terzaghi (1936).

$$[1]\quad \frac{\delta \bar{u}}{\delta t} = C_v \frac{\delta^2 \bar{u}}{\delta z^2}$$

where ū is the pore pressure at a depth z in the clay layer having thickness H and t is the time elapsed after the application of load. The constant C_v is defined as the coefficient of consolidation.

The equation Eq. [1] is employed to derive the settlement–time relation since the settlement is dependent on pore-pressure dissipation, which in turn is time dependent. This equation is analogous to well-known Heat Equation and solution of the equation is arrived in the similar lines as that of Heat Equation.
The current methods of evaluation of settlement have many limitations primarily due the fact that C_v does not remain constant as assumed in the derivation of the equation Eq. [1]. Terzhagi et al. (1995) mention these limitations as a part of the derivation of the Eq. [1].
The main reason for the variation of coefficient of consolidation, C_v is due to the fact that it depends on the coefficient of volume change m_v as $C_v = \frac{k}{\gamma_w m_v}$. But the coefficient of volume change mv is not a constant which evident from the relation $m_v = \frac{\Delta H_t}{H.\Delta\sigma}$. Here it may be noted that for clays incremental settlement ΔH_t is not linearly proportional to the incremental load $\Delta\sigma$, which is evident from the relation depicted below

$$\Delta H_f = H.\frac{C_c}{1+e_0}\left\{\log_{10}\left(1+\frac{\Delta\sigma}{\sigma_0}\right)\right\}$$

Another parameter which governs the C_v is the coefficient permeability k, which also does not remain constant throughout the consolidation process. For the above reasons, the analytical solution of the Eq. [1] does not match with the laboratory tests. Several attempts have been made by researchers over the time to overcome this limitation by suggesting various alternate methods to evaluate C_v like Rectangular Hyperbola Method by Sridharan et al. (1987) or by suggesting methods to correlate C_v to other soil parameters like Index properties viz. Carrier (1985), Sridharan and Nagaraj (1999), and Sridharan and Nagaraj (2004). In the above attempts, the corrections are proposed without altering the fundamental equation cited above under Eq. [1]. Though the variation of C_v with applied pressure (load) was identified by researchers like Olsen (1986), the mathematical reason for such a variation is also not seen to have investigated in detail.
In the present paper the fundamental Terzhagi's Equation, Eq. [1] is revisited after deriving a simpler form of the same as shown in Eq. [8]. A new expression is derived to accurately predict the rate of primary consolidation settlement of clays at any stage. A slightly different approach is followed to derive these expressions when compared to the conventional method. However, various terminologies are retained as such, as they have been already established and familiar in the field.

2 Current approach

As stated earlier, by definition the coefficient of volume change, $m_v = \frac{\Delta H_t}{H.\Delta\sigma}$;

Therefore settlement at any time ΔH_t is given by: $\Delta H_t = m_v.H.\Delta\sigma' = m_v H.(\Delta\sigma - \bar{u})$, Where H is the thickness of the clay layer, Δσ is the applied pressure, $\bar{u}$ is the average (mean) pore pressure at any time, and Δσ' is the effective pressure so that $\Delta\sigma = \Delta\sigma' + \bar{u}$. This can be written as follows when $u = \frac{\bar{u}}{\Delta\sigma}$

[2] $\Delta H_t = m_v H \Delta\sigma(1-u)$

Solving partial differential equation (PDE) $\frac{\delta\bar{u}}{\delta t} = C_v \frac{\delta^2\bar{u}}{\delta z^2}$; and after applying the boundary conditions, the Degree of Consolidation (U) is obtained as given below.

[3] $$U = 1 - \frac{8}{\pi^2}\left\{e^{\frac{-\pi^2}{4}T_v} + \frac{1}{9}e^{\frac{-9\pi^2}{4}T_v} + \frac{1}{25}e^{\frac{-25\pi^2}{4}T_v}..\right\}$$

Where, U is the degree of consolidation which is defined as ratio of settlement at any time to final settlement. $U = \frac{\Delta H_t}{\Delta H_f}$

$\Delta H_t =$ Settlement at any time (t)

$\Delta H_f =$ Final settlement

$T_v = \frac{C_v}{d^2}.t$

where C_v is the coefficient of consolidation, d is the length of the drainage path, which is equal to H or $\frac{H}{2}$ depending whether single or double drainage as the case may be and t is time elapsed after loading.

Following empirical relations are well known and given in literature for easy computation. The expressions for double drainage case are given below

$U \leq 0.6$

[4] $T_v = \left(\frac{\pi}{4}\right)U^2$

$U > 0.6$

[5] $T_v = -\frac{4}{\pi^2}\left\{\log(1-U) + \log\left(\frac{\pi^2}{8}\right)\right\}$

The Eq. [5] can be obtained by approximating $U = 1 - \frac{8}{\pi^2}\left(e^{-\frac{\pi^2}{4}}\right)$ by taking first two terms of the Eq. [3], but Eq. [4] seems to be purely is empirical.
A sample consolidation test data presented by Punmia et al. (2005) is used to carry out a detailed analysis by calculating Cv corresponding to each time-interval using the Eq. [4] or Eq. [5] as applicable. This is presented in the **Table 1**

Table 1. Determination of Coefficient of Consolidation using the conventional method using Eq.[3]

Pressure Range: 100 KN/m^2 (KPa) to 200 KN/m^2 (KPa)
Initial Height of Sample: 1.738 cm & Final Height of Sample: 1.611 cm

t (min)	**t (s)**	**Reading**	**Diff.**	**ΔH_t**	**H_t**	$U = \frac{\Delta H_t}{\Delta H_f}$	$1-U$	**T_v**	$C_v = \frac{T_v d^2}{t}$
0	0	340	0	0	1.738	0	1.00000	0	*undefined*
0.25	15	360	20	0.02	1.718	0.15748	0.84252	0.0195	0.000910
1	60	370	30	0.03	1.708	0.23622	0.76378	0.0438	0.000512
2.25	135	378	38	0.038	1.7	0.299213	0.70079	0.0703	0.000365
4	240	386	46	0.046	1.692	0.362205	0.63780	0.1030	0.000301
6.25	375	394	54	0.054	1.684	0.425197	0.57480	0.1420	0.000265
9	540	402	62	0.062	1.676	0.488189	0.51181	0.1872	0.000243
12.25	735	410	70	0.07	1.668	0.551181	0.44882	0.2386	0.000228
16	960	416	76	0.076	1.662	0.598425	0.40157	0.2813	0.000205
20.25	1215	422	82	0.082	1.656	0.645669	0.35433	0.3354	0.000194
25	1500	426	86	0.086	1.652	0.677165	0.32283	0.3732	0.000174
36	2160	434	94	0.094	1.644	0.740157	0.25984	0.4611	0.00015
49	2940	440	100	0.1	1.638	0.787402	0.21260	0.5425	0.000129
60	3600	445	105	0.105	1.633	0.826772	0.17323	0.6255	0.000122
120	7200	454	114	0.114	1.624	0.897638	0.10236	0.8387	8.17E-05
180	10800	456	116	0.116	1.622	0.913386	0.08661	0.9064	5.88E-05
300	18000	459	119	0.119	1.619	0.937008	0.06299	1.0355	4.03E-05
480	28800	462	122	0.122	1.616	0.96063	0.03937	1.2260	2.98E-05
1440	86400	467	127	0.127	1.611	1	0	*undefined*	

Mean=0.002236; Standard Deviation=0.000213; Coefficient of Variation=0.903807

The values of T_v using the Eq. [4] when $U \leq 0.6$ and Eq. [5] when $U > 0.6$ where

$U = \frac{\Delta H_t}{\Delta H_f}$ are computed and corresponding value of *Cv* is calculated for **Table 1** preparation. Then by definition $T_v = \frac{C_v}{d^2}.t$ and hence $C_v = \frac{T_v d^2}{t}$ must remain constant [*d is not a variable for a particular test. In this case it is double drainage and hence* $d = \frac{H}{2}$]. On detailed review of **Table 1**, it could be seen that C_v do not remain a constant. As mentioned in the introduction, based on the Eq. [1] i.e., $\frac{\delta u}{\delta t} = C_v \frac{\delta^2 u}{\delta z^2}$, mathematically C_v must be a constant, but it shows a large variation. The least value in the **Table 1** is 2.98E-05 and the highest value is 0.000910. The ratio of highest value to the least value is approximately 30 and the coefficient of variation is 0.9038. Moreover the mean value of C_v in this case works out to 0.00023 and C_v evaluated by graphical methods are different (By Root-T Method is 0.000394 cm^2/sec and Logarithmic method is 0.000271 cm^2/sec). As stated earlier, this is mainly because while solving the Eq. [1], m_v is taken as constant.

Since $m_v = \frac{\Delta H}{H.\Delta\sigma}$, when it is assumed constant, it implies a linear variation of settlement to the stress (load) which is not true and it contradicts its well established logarithmic relationship.

[6] $$\Delta H_f = H.\frac{C_c}{1+e_0}\left\{\log_{10}\left(1+\frac{\Delta\sigma}{\sigma_0}\right)\right\}$$

Thus the equation used derivation of C_v does not have any mathematical backing as m_v is not a constant.

$$\Delta H_t = m_v H \Delta\sigma(1-u)$$

As long as the value of Cv in the Eq. [1] does not remain a constant, the realistic settlement prediction becomes impossible. Thus, there is a need for an accurate expression giving the settlement at any time which is dependent of coefficients that are constant for a given type soil under any loading condition. Thus, the need for a revisit of the conventional is established. In order to proceed with modification the simpler form of Eq. [1] as shown under Eq. [8] chosen for the formulation of the new equation.

3 Proposed New Equation

The pore pressure dissipation being a never-ending process, the relationship between pore pressure and time is exponential. This is similar to radioactive decay. The significant finding proposed in this paper is that the above exponential variation is with respect

to the square root of time and not with the real time as assumed in the Terzaghi's conventional Equation [1].
Following equation is derived by a new approach (Derivation not included in this paper).

[7] $\Delta H_t = \Delta H_f[(1-u)\{1-\log(1-u)\}]$ where $u = \frac{\bar{u}}{\Delta\sigma}$ and $\bar{u}$ is the average (mean value) of pore pressure at any given time. This pore pressure can be directly found from the following proposed equation.

[8] $u = e^{-\sqrt{T_R}}$

Here T_R is a "Modified Time Factor" analogues to T_v.
This modified Time factor is then evaluated using the following relation.

[9] $T_R = 2.\left(\frac{C'_v}{H^2}\right).t$

Here C'_v is defined as the modified coefficient of consolidation which is as given below.

[10] $C_v' = \frac{k}{m_v'.\gamma_w}$

[11] $\Delta H_f = m'_v.H.\Delta\sigma$

Here m'_v is the modified coefficient of volume change. $m'_v = \frac{\Delta H_f}{\Delta\sigma.H}$, which will remain constant throughout the settlement process under the given applied load. Thus the proposed equation [7] can be rewritten as

[12] $\Delta H_t = m_v H\Delta\sigma'[(1-u)\{1-\log(1-u)\}]$

Similarly since the Degree of Consolidation (U) is equal to $\frac{\Delta H_t}{\Delta H_f}$, equation [7] can also be written as follows also.

[13] $U = (1-u)\{1-\log(1-u)\}$

However, the final settlement will be computed using the conventional relation Eq. [6]

[14] $\Delta H_f = H.\frac{C_c}{1+e_0}\left\{\log_{10}\left(1+\frac{\Delta\sigma}{\sigma_0}\right)\right\}$

[15] $m_v' = \left(\frac{C_c}{1+e_0}\right).\left(\frac{\log_{10}\left(1+\frac{\Delta\sigma}{\sigma_0}\right)}{\Delta\sigma}\right)$

Further for Single drainage case:

$$[16]\ C'_v = \frac{k(1+e_0)}{\gamma_w.C_c}\left(\frac{\Delta\sigma}{\log_{10}\left(1+\frac{\Delta\sigma}{\sigma_0}\right)}\right)$$

For double drainage case:

$$[17]\ C'_v = \frac{4k(1+e_0)}{\gamma_w.C_c}\left(\frac{\Delta\sigma}{\log_{10}\left(1+\frac{\Delta\sigma}{\sigma_0}\right)}\right)$$

In general, *modified coefficient of consolidation* will be [17] $C_v^{'} = V_R E_R D_R A_R$

$V_R =$ Velocity- Consolidation Coefficient

(Soil Specific)= $\frac{k}{\gamma_w C_c}$;

$E_R =$ Void's Ratio Factor (Site Specific) = $(1+e_0)$;

$D_R =$ Drainage Factor [1 or 4 depending on the drainage condition]

A_R = Applied load (Pressure/Stress) Factor

$$= \frac{\Delta\sigma}{\log_{10}\left(1+\Delta\sigma/\sigma_0\right)}$$

Now the same test data discussed in Section 2 is analyzed using the proposed settlement equation as given in **Table 2** below.

Table 2. Determination of 'Modified Coefficient of Consolidation" using Eq[21]

Pressure Range: 100 KN/m^2 (KPa) to 200 KN/m^2 (KPa)
Initial Height of Sample: 1.738 cm & Final Height of Sample: 1.611 cm

t (min)	t (s)	Reading	Diff.	ΔH_t	H_t	$\frac{\Delta H_t}{\Delta H_f}$	$u = 1 - U$	*log* u	$\sqrt{C_v}$
0	0	340	0	0	1.738	0		undefined	
0.25	15	360	20	0.02	1.718	0.157480315	0.963455	-0.03723	0.01138
1	60	370	30	0.03	1.708	0.236220472	0.937344	-0.06471	0.00989
2.25	135	378	38	0.038	1.7	0.299212598	0.913093	-0.09092	0.00927
4	240	386	46	0.046	1.692	0.362204724	0.885702	-0.12137	0.00928
6.25	375	394	54	0.054	1.684	0.42519685	0.854897	-0.15677	0.00959
9	540	402	62	0.062	1.676	0.488188976	0.820277	-0.19811	0.01009

t (min)	t (s)	Read-ing	Diff.	ΔH_t	H_t	$\frac{\Delta H_t}{\Delta H_f}$	$u = 1 - U$	*log* u	$\sqrt{C_v}$
12.25	735	410	70	0.07	1.668	0.551181102	0.78127	-0.24683	0.01078
16	960	416	76	0.076	1.662	0.598425197	0.748662	-0.28947	0.01106
20.25	1215	422	82	0.082	1.656	0.645669291	0.712667	-0.33874	0.01151
25	1500	426	86	0.086	1.652	0.677165354	0.686483	-0.37617	0.0115
36	2160	434	94	0.094	1.644	0.74015748	0.627695	-0.4657	0.01186
49	2940	440	100	0.1	1.638	0.787401575	0.576466	-0.55084	0.01203
60	3600	445	105	0.105	1.633	0.826771654	0.52746	-0.63968	0.01262
120	7200	454	114	0.114	1.624	0.897637795	0.416871	-0.87498	0.01221
180	10800	456	116	0.116	1.622	0.913385827	0.386203	-0.95139	0.01084
300	18000	459	119	0.119	1.619	0.937007874	0.333256	-1.09884	0.0097
480	28800	462	122	0.122	1.616	0.960629921	0.26715	-1.31995	0.00921
1440	86400	467	127	0.127	1.611	1	0	*undefined*	

Mean=0.01075; Standard Deviation=0.001133; Coefficient. of variation=0.10536

The modified coefficient of Consolidation, *Cv*' remains constant as seen from the **Table 3** which validate the proposed expression for calculating settlement at any given time after the application of the load. (The ratio of highest value to least value is only 1.2 and Coefficient of variation: 0.105)

4 Rate of Settlement prediction using the proposed method

When above factors/coefficients are known, the settlement of the soil at a time 't' after loading, having' H' layer thickness under $\Delta\sigma$ load on a layer having normal consolidation pressure σ_0, can be calculated as follows:

1. **Establish the Velocity-Consolidation Coefficient VR.**
   Out of the four factors controlling the value of CV', VR is only a new soil parameter, which needs to be established for each type of soil. This can be done using the conventional consolidation test as described below, Using similar procedure as used for preparation of **Table 3** the mean value of $\sqrt{C_v'}$ shall be arrived after carrying out usual consolidation test and then $\sqrt{V_R}$ shall be calculated using the relation $\sqrt{V_R} = \frac{\sqrt{C'_v}}{\sqrt{E_R.D_R.A_R}}$ where $E_R, D_R \,\&\, A_R$ are as defined earlier.
2. **Calculate T_R using the Eq. [9];** $T_R = 2.\left(\frac{C'_v}{H^2}\right).t$
3. **Calculate Pore Pressure Ratio (u), using the Eq. [13] ;** $u = e^{-\sqrt{T_R}}$

4. **Calculate Final settlement (ΔH_f) using Eq. [6];**

$$\Delta H_f = H.\frac{C_c}{1+e_0}\left\{\log_{10}\left(1+\frac{\Delta\sigma}{\sigma_0}\right)\right\}$$

5. **Settlement at any time will be (ΔH_t) using Eq. [8]**

$$\Delta H_t = \Delta H_f\{(1-u)(1-\log(1-u))\}$$

Using the average value of $C'_v = 0.0002347$ that was arrived earlier using **Table 3**, the settlement is predicted and is compared with the actual and presented in **Table 3** the maximum variation is only ~10% and thus the proposed method is validated.

Table 3. Prediction of Settlement using the proposed method using Eq[18]

Pressure Range: 100 KN/m^2 (KPa) to 200 KN/m^2 (KPa)
Initial Height of Sample: 1.738 cm & Final Height of Sample: 1.611 cm

t (min)	**t (s)**	**Read-ing**	**Diff.**	**ΔH (actual)**	**H**	$\sqrt{T_R}$	u	ΔH_t *(predicted)*	*error* %
0	0	340	0	0	1.738	0	1	undefined	
0.25	15	360	20	0.02	1.718	0.035163	0.9654482	0.019155265	-4.2
1	60	370	30	0.03	1.708	0.070326	0.9320902	0.031820901	6.1
2.25	135	378	38	0.038	1.7	0.105489	0.8998848	0.041976509	10.5
4	240	386	46	0.046	1.692	0.140651	0.8687922	0.050506298	9.8
6.25	375	394	54	0.054	1.684	0.175814	0.8387738	0.05784284	7.1
9	540	402	62	0.062	1.676	0.210977	0.8097927	0.06424716	3.6
12.25	735	410	70	0.07	1.668	0.24614	0.7818129	0.069895175	-0.1
16	960	416	76	0.076	1.662	0.281303	0.7547998	0.074913905	-1.4
20.25	1215	422	82	0.082	1.656	0.316466	0.7287201	0.079399482	-3.2
25	1500	426	86	0.086	1.652	0.351628	0.7035415	0.083427185	-3.0
36	2160	434	94	0.094	1.644	0.421954	0.6557642	0.090340012	-3.9
49	2940	440	100	0.1	1.638	0.49228	0.6112313	0.096020374	-4.0
60	3600	445	105	0.105	1.633	0.54474	0.5799923	0.099613323	-5.1
120	7200	454	114	0.114	1.624	0.770379	0.4628375	0.110615045	-3.0
180	10800	456	116	0.116	1.622	0.943518	0.389256	0.115809782	-0.2
300	18000	459	119	0.119	1.619	1.218076	0.2957986	0.120797119	1.5
480	28800	462	122	0.122	1.616	1.540758	0.2142186	0.123852299	1.5
1440	86400	467	127	0.127	1.611	2.668672	0.0693443	0.126687339	-0.2

5 Conclusion

As the accurate prediction of consolidation settlement is very crucial and important from both design and construction point of view when structures are proposed to be founded on clayey soils. The conventional methods have many limitations primarily because the coefficient of consolidation (*Cv*) obtained from different method is different. Moreover the past literature survey reveals that most of the attempts in the past were to refine the *Cv* which is hardly sufficient to address the problem.
A need for revisit of the fundamental equation put forth by Terzaghi is identified and a modification to this equation is proposed in this paper. Thus a new expression as given below under the section (3) is arrived to find the settlement of primary consolidation of clays at any stage (time period). The following are the new Equations derived in this paper.

1. Settlement at any time

   $\Delta H_t = \Delta H_f (1-u)(1-\log(1-u))$ where $u = \frac{\bar{u}}{\Delta\sigma}$ and $\bar{u}$ is the average (mean) of pore pressure at any given time.
2. Mean pore pressure at any time

   $\bar{u} = \Delta\sigma . e^{-\sqrt{T_R}}$; where T_R is the "Modified Time Factor" analogues to Tv. and it is evaluated using the relation.

   $T_R = 2.\left(\frac{C'_v}{H^2}\right).t$, where Here C'_v is defined as the modified coefficient of consolidation which is as given below.
3. Modified Coefficient of Consolidation

   $C_v' = V_R E_R D_R A_R$ (Refer back for details).

Advantages of proposed equation are summarized as follows:

1. The proposed formula will be able to predict the settlement at any time (t) after the application of load with a high degree of accuracy (error ±10%) from the actual at any stage (time period).
2. The proposed formula is direct and simple and will eliminate graphical methods, the required parameters can be established as shown in **Table 2**;
3. The new parameter (VR) can be established by using the usual consolidation test;
4. The various factors affecting the consolidation process are properly reflected in the proposed Equations enabling proper conceptualization of the process.

The scope for the further study is the establishment of the new soil parameter for different types of clay and thus the experiential validation of the proposed method. Further, it could be extended to the three-dimensional consolidation scenario also. An alternate solution based on the postulation by discrete mathematical method is also feasible.

References

1. Terzaghi, K. Peck, R. B and G.Mesri, 1995, Soil Mechanics in Engineering Practice (Third Edition), John Wiley and Sons, New York.
2. Punmia.B.C, Ashok Kumar Jain, Arun Kumar Jain , 2005; Soil Mechanics & Foundations, Laxmi Publications (P) Ltd, New Delhi.
3. Carrier, W. D., III, 1985, "Consolidation Parameters Derived From Index Tests," Geotechnique, Vol. 35, No. 2, pp. 211–213.
4. Olson, R. E., 1986, "State of Art: Consolidation Testing, Consolidation of Soils, Testing and Evaluation," ASTM STP 892, R. N.Yong and F. C. Townsend, Eds., pp. 7–70.
5. Raju Narasimha, P. S. R., Pandian, N. S., and Nagaraj, T. S.,1995, "Analysis and Estimation of Coefficient of Consolidation, "Geotechnical Testing Journal, Vol. 18, No. 2, pp. 252–258.
6. Robinson, R. G. and Allam, M. M., 1998, "Effect of Clay Mineralogy on Coefficient of Consolidation," Clays and Clay Minerals, Vol. 46, No. 5, pp. 596–600.
7. Sridharan, A., Murthy, N. S., and Prakash, K., 1987, "Rectangular Hyperbola Method of Consolidation Analysis," Geotechnique,Vol. 37, No. 3, pp. 355–368.
8. Sridharan, A. and Nagaraj, H. B., 2000, "Compressibility Behaviour of Remoulded, Fine-Grained Soils and Correlation with Index Properties," Canadian Geotechnical Journal, Vol. 37,No. 2, pp. 712–722.
9. A. Sridharan and H. B. Nagaraj; "Coefficient of Consolidation and its Correlation with Index Properties of Remolded Soils", Geotechnical Testing Journal, Vol. 27, No. 5.pp 01-06