CONSOLIDATION OF SOIL FOR FOUNDATION BY USING SAND DRAINS

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ABSTRACT

This paper describes the advantage of using sand drains in consolidation of saturated compressible soils. It also discusses the applicable theory, design procedure; type and technique of drain installation as well as monitoring and control procedures. The most significant part of this paper is the experimental evaluation of the validity of design and its efficiency.

INTRODUCTION

When loads are to be applied to saturated compressible soils, particularly if they affect large loaded areas, pre-consolidation of the soil to prevent excessive differential settlement is usually required when one or more of the following conditions exist:

1. Calculation of stability in un-drained conditions shows the soil to have inadequate load carrying capacity for the desired applied load.
2. Monitoring reveals that settlement is excessive and/or time-delayed with respect to the time of consolidation.
3. When the loaded area is large, the soil is heterogeneous in nature, and significant differential settlement is predicted.

One of the most effective techniques for consolidation of such soils is to make use of "pre-loading embankment" in conjunction with "vertical drains" in the existing soil. Since the soil involved in the treatment have high water content and low permeability (e.g. saturated clays or soft clayey silts), the surcharge loading initially produces an increase in interstitial pressure. This dissipates gradually, leading to soil settlement and a corresponding increase in the mechanical properties.

The time of consolidation in saturated soil of very low permeability varies directly with the length of the drainage path. As the coefficient of permeability is large in sand than that in situ soil, the sand columns called sand drains become the path of low energy potential and sub soil water flows vertically and radially through sand columns under the hydraulic gradient produced by the fill. As a result, the length of the drainage path becomes very short which helps to speed up the drainage process and consequently the consolidation process is accelerated. After about a decade of study and experience it has been found that, sand drains together with the surcharge preloading are considered as the most cost and time constrain effective solution for the consolidation of saturated compressible soil.

The sand drains are mainly used in consolidation of extensive areas of loading, such as airport runways, road embankments, large storage areas and reservoirs etc.

This paper outlines the method of designing sand drains in conformity with the characteristic of soil in situ and in compliance with the consolidation theory [2, 3]. Then the methods of execution and their merit and de-merits are briefly discussed and the quality of sand used in sand drains is elaborated. Necessary steps for monitoring and quality control are also described. An experimental programme was carried out for the verification of the validity of the design and its output data is also given in this paper. Asaoka's method [4] was adopted for the analysis of the measured settlements which confirms that the total settlement achieved was about 100% of the estimated value. Finally the validity of the design and conclusions are given.

BASIC PRINCIPLE AND THEORY OF SAND DRAINS

Sand drains or sand blankets are based on the application of the Consolidation Theory [1, 2]. Accordingly the modulus of deformation (E_S) is computed based upon stress-strain relation of the soil. The deformation or settlement (ΔH) in soil under the effect of stress (Δq) over the influence length L_0 is given by the equation:

$$\Delta H = \int_0^{L_0} \frac{\Delta q}{E_S} dZ = \int_0^{L_0} \epsilon dZ \quad \text{---(1)}$$

Or

$$\Delta H = \epsilon Lo \quad \text{---(2)}$$

Where ϵ is the strain. Equation 2 provides settlement but without the consideration of time required, which is a vital parameter in fine-grained saturated soil. Therefore, laboratory tests are required to estimate the compression parameter for the amount of settlement and the consolidation parameter for the settlement rate.

The time-deformation data obtained from the laboratory test are plotted on either a semi logarithmic plot or $\sqrt{t}$ plot, in order to obtain the time at some percent consolidation. The most commonly used parameters are D_{50} (50% consolidation) at t_{50} (time at 50% consolidation).

The coefficient of consolidation (c_v) is obtained from the following relation:

$$c_v = \frac{T_i H^2}{t_i} = \frac{K}{r_w m_v} \quad \text{---(3)}$$

Where T_i is the time factor depending on % of consolidation and is about 0.848 and 0.197 for 90% and 50% consolidation respectively. H is the length of the longest drainage path for a particle of water and is taken as half of the sample thickness when the drainage is from both faces. t_i is the time required for i% consolidation to take place (t_{50} is normally used) and K is the coefficient of permeability. The primary settlement is computed either by using compressive index c_c or comp. factor c'_c which is given as:

$$c'_c = \frac{c_c}{1+e_0} \quad \text{---(4)}$$

DESIGN OF SAND DRAINS

In order to decide the method of consolidation and use of sand drains to accelerate the process of consolidation in saturated compressible soils, the following steps are undertaken in chronological order:

1. Data about subsoil conditions are gathered which include:
   - SPT results of borings.
   - Measurements of 100 grain size distributions on samples of various layers.
   - Atterburg's limits measurements on samples of various layers.
   - Determination of topographical layout.
   - Analysis of the results of the soil survey.
   - Odometer tests to determine the compressibility ($c_c/(1+e_0)$) of the particular layer(s) to be consolidated.
   - The coefficient of consolidation c_v (m^2/sec.) adopted as the maximum value recorded under maximum applied load.
2. The height of fill to be placed in order to reach the design platform level at the end of any pre-defined time period is determined. Provision must be made for incomplete consolidation during works as well as some compensation for secondary settlement which may occur during any user-defined service time period.
3. Expected settlement and thickness of fill required above the compressible layer is estimated so that the consolidation proceeds both vertically and radially by using sand drains and 95% of the consolidation is obtained after the predetermined time.
   Based upon the soil data discussed above and using equation (1) and (2) or by using computer programme (TASDEJ) of M/S TERRASOL, one can determine the expected degree of consolidation, preloading time period, theoretical fill height, time period for compensating secondary settlement.
4. For final design of vertical sand drains, one can use the graphical method for determining the sizes of vertical drains as given in Appendix 1 which is adopted from Terrasol [5]. In this technique, based upon the following :
   i. Coefficient of consolidation c_v (m^2/s).
   ii. Preloading time period (months).
   iii. Degree of consolidation Ur (%).
   iv. Selected diameter of drains (Cm).

   One can determine the spacing of sand drains, which must be distributed along a triangular grid. Such drains, set in place using an auger, and must reach the top of the compacted, dense layer of soil. As such the required length of the sand drain could be up to 15 to 20 m.

EXECUTION OF SAND DRAINS

Three methods are used for the placement of sand drain. These techniques are:

- Driven or vibratory closed - end mandrel.
- Jetted.
- Hollow stem continuous - flight auger.

By using "Driven or vibratory closed-end mandrel" method of installation, a closed steel casing equipped with a detachable shoe is driven in the soil. The tube is filled with sand and then the tube is extracted leaving shoe in the hole. The jetting type method consists of using driven pipes where the soil inside is then jetted. The rest of the procedure is same as in Method 1. The continuous flight hollow auger method is described in the following paragraph. Some undesirable effects in the first two methods are summarized as following:

i. Smearing effect – During the extraction of the tube, there is possibility of reduction of the permeability of sand drain by clogging of the more permeable seams, specially if the mud is very laminated.
ii. Heave caused by driving – Heave (remoulding) effect in soil is considerable in a zone of about 4 x diameter of sand drain used.
iii. Extra pore pressure – Driving can generate some extra pore pressure during insertion of the casing. This excess pore pressure which is only significant close to the drain, will begin to dissipate in the sand as soon as the casing is withdrawn, which occurs in a time period of less than a few minutes.

In view of the smearing effect of driven or vibratory closed end mandrel and complex installation of Jetted sand drains, it is recommended to retain sand drains placed by hollow stem continuous – flight auger. This method is rather simple to carry out and limits the soil displacement and disturbance. The main steps for this method are illustrated in Figure1 and are as follows:

i. Placement of the equipment at the drain location.
ii. Screwing of the auger down to the chosen depth.
iii. Rotation of the auger at constant depth in order to separate it from the surrounding soil.
iv. Injection of sand while the auger is extracted (screwing is continuous during this phase).
v. Completion of the sand drains up to the platform level.

It should be noted that the rate of screwing and extraction of the auger must be such as, not to generate collapsing of the surrounding soil into the borehole.

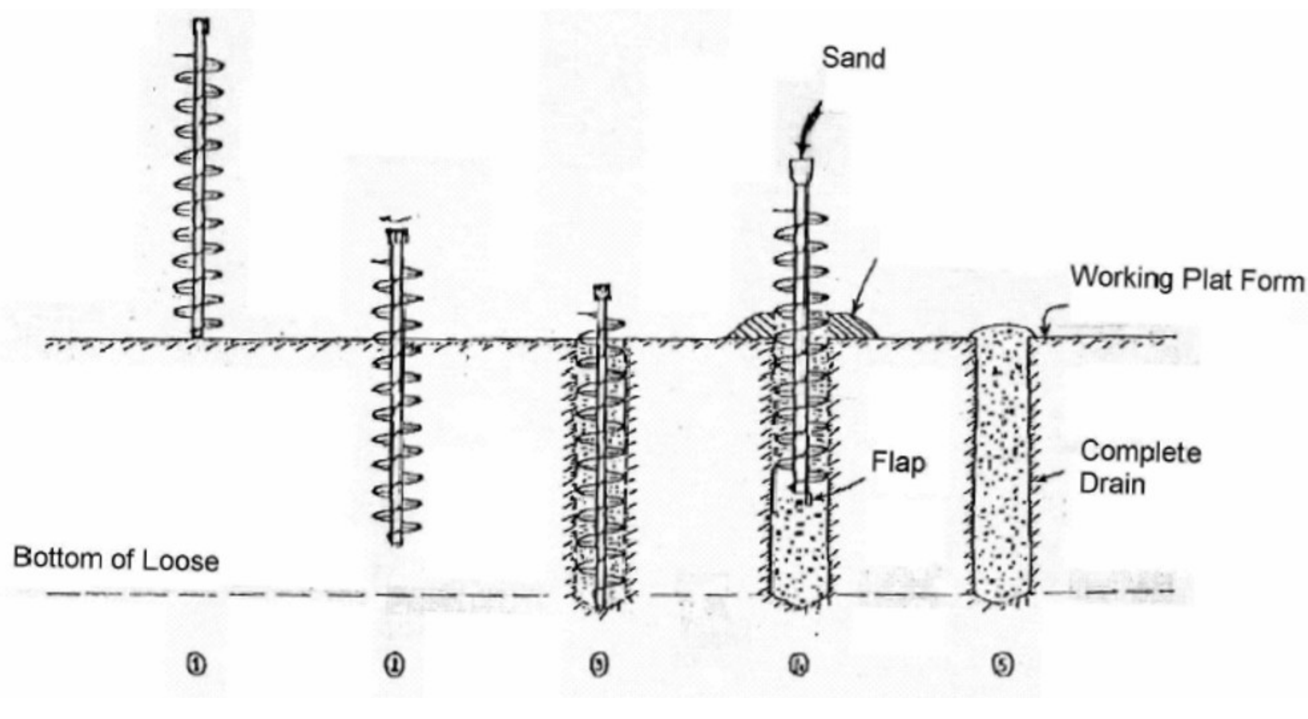


Figure 1: Execution of Sand Drains Using Hollow Stem Continuous Flight Auger

MONITORING AND CONTROL OF SAND DRAINS

Quality of sand drains must be ensured by the following monitoring and control measures:

i. Examining grain size: The sand should be clean, uniform, fine sand with a d_{50} between 0.4 mm and 1.2 mm and less than 5% particles finer than the N 200 sieve (75 μm). The d_{100} must be lower than 5 mm.
ii. Using only specified quality of sand. In case of sand drains placed with inappropriate sand, a new drain has to be carried out to replace it.
iii. Checking the topography of the saturated compressible layer of soil (sea bottom in coastal areas) along the referenced profiles before the beginning of the filling.
iv. The thickness of the fill must be checked during the filling operation along the referenced profiles. Drilled boreholes with recording of drilling parameters, static or dynamic penetrometer can be used for this purpose. Each method must be calibrated with a cored borehole or an investigation pit.
v. Settlement plates should be laid immediately after reaching top elevation of the platform. These should consist of steel plates on concrete base, embedded at least 1.0 meter under the platform grade, with a protected vertical steel rod. The settlement plates should be distributed along the refilled area at a spacing of about 50 m c/c.
vi. It is necessary to monitor that consolidation attains its design value (95% in the area with drains) at the end of the specified time. Asaoka's Method [4] enables to define the extrapolated real settlement for each location. From this result it should be verified that measured settlement represents at least 95% of the extrapolated value.

APPLICATION EXAMPLE

The author was involved in an extensive experimental program on this subject. The following trial test illustrates the design parameters, method of execution and analysis of the test results [5-6].

A site to be reclaimed occupied an approximate area of 45000 m^2 of a bay like stretch of shallow sea water with a maximum water depth around 1.5 m. Based on the available data, the maximum tide level was +0.6 MSL, and the minimum tide level -0.5 MSL.

The platform for the external work (roads, utilities etc.) was to be raised to an elevation of +2.5 MSL using a fill volume estimated around 200,000 m^3. Average distributed load at the foundation level was estimated as 50 KPa (0.5 bar).

The soil profile within the project limit is summarized in Table 1 and the available oedometer test results on samples from layer 2 are given in Table 2.

Based upon the procedure described earlier, the expected settlement and thickness of fill while using sand drains were calculated. The calculations have shown the following results related to layer 2 and expected settlement and thickness of fill required (refer Figure 2).

Table 1: Soil Profile for the Required Work

Layer	Litho logy	Thickness (m)	Levels	Density-Consistency	N (lb/ft)
1 (Only onshore)	Silty/clayey sand and gravel	3.0	+2.5 → -0.5	Medium dense	15
2	Silty/clayey sand and gravel	7.0	-0.5 → -7.5	Very Loose	6 (1) 0 (2)
3	Silty/clayey sand and gravel	10.5	-7.5 → -18	Compact/ dense	33
4	Mixture of clay, clayey sand, silt and coral	20.0	-18 → <-35	Compact	30

1. on shore
2. off shore

Table 2: Available Oedometer Test Results on Samples from Layer 2

	W_n (%)	PI (%)	e_0	c_c	$c_c/(1+e_0)$
BH3 S1 (0. – 0.75 m)	46	19	1.24	0.65	0.29
BH7 S2 (1.95 – 2.25 m)	39	15	1.02	0.28	0.14
BH2 S1 (3.5 – 4.5 m)	54	32	1.39	0.42	0.175

c_c (compressibility index)	= 0.175
c_v (coefficient of consolidation)	= 10 m^2/year
Theoretical fill height (HR)	= 4.0 m.
Expected settlement (TR)	= 0.71 m. (in 6 months)
Fill unit weight	= 18.0 KN/m^3
Total equivalent water depth	= 1.3 (Elve. = 0.8 MSL)
Preloading time	= 6 months.
Final time of secondary settlement	= 3.0 (years)
Degree of consolidation	= 1.0
ER, the residual height to be scraped at the end	= 0.25 m.

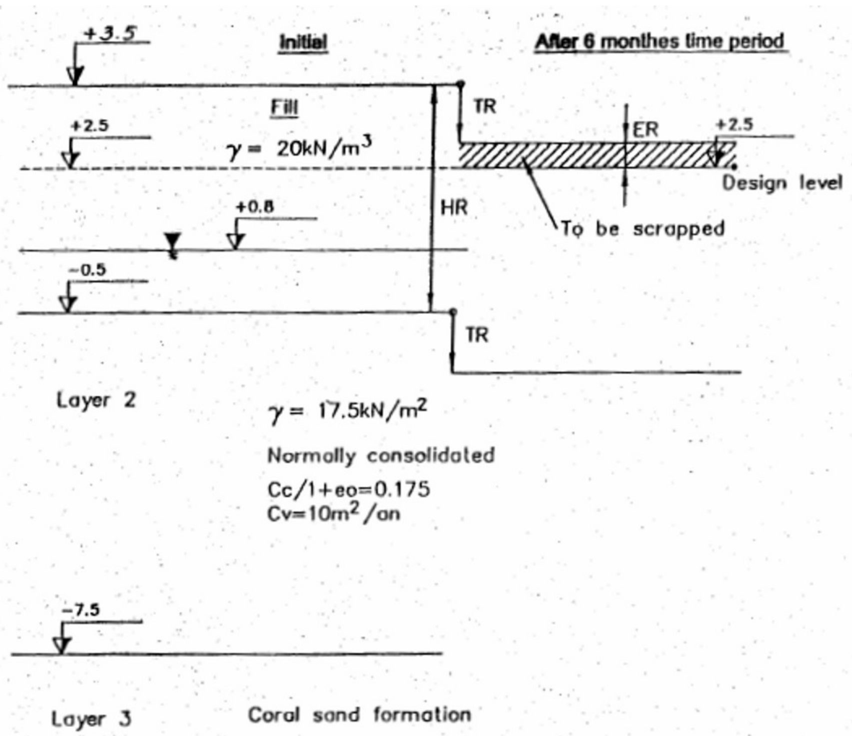


Figure 2:

Determination of the height of fill to be placed in order to reach design platform level at the end of the 6 months time period of consolidation

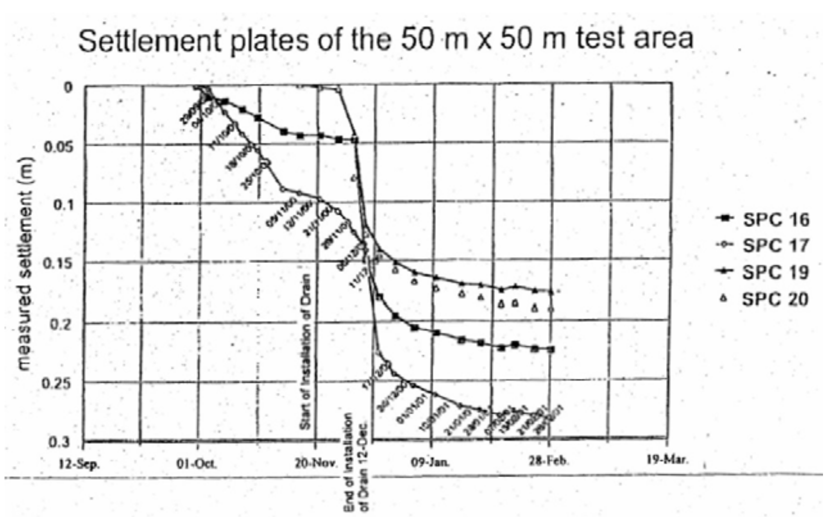


Figure 3: Settlement Vs Time (Ref. 5 & 6)

(TR+ER), defines the over height to compensate settlement, partial consolidation during work and part of the secondary settlement will be occurring in the next 3 years.

It is noticed that part of the expected settlement (TR) occurred during the filling operation. Thus a rigorous levelling at - 0.5 + 4.0 = + 3.5 MSL could lead to some additional fill to keep the fill thickness HR = 4.0 m.

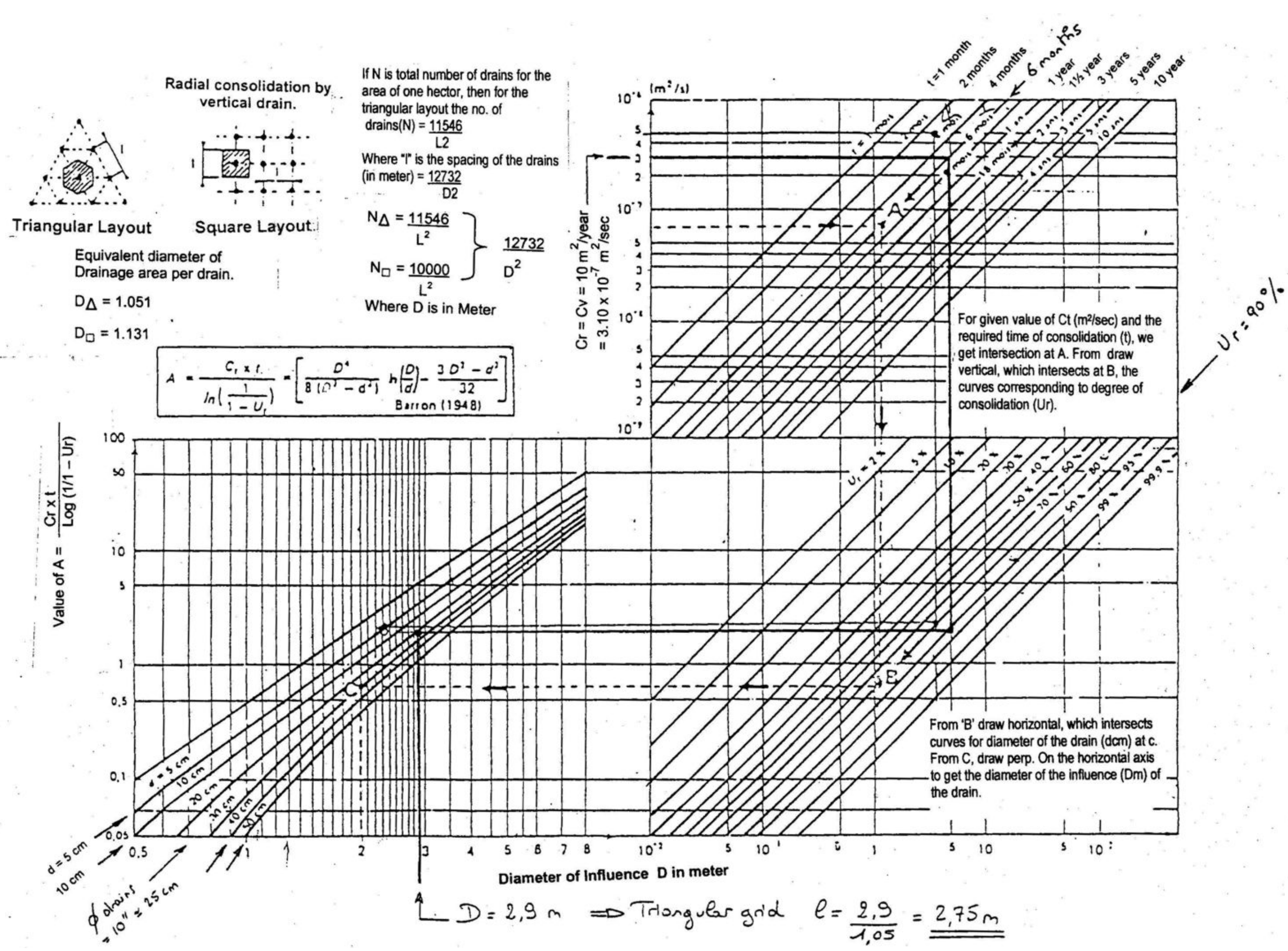
Consequently, a design of vertical drains was adopted, using Appendix 1. It was found that the degree of consolidation u = 95% can be obtained using sand drains with the following characteristics:

- Diameter of drains = 0.25 m
- Triangular grid size= 2.75 m
- Length of drains = 15.0 m

For settlement analysis, the settlement plates were located at the trial test location. For this purpose, a test area of 50 m x 50 m was selected to test the efficiency of the sand drains. In this area, installation of sand drains started on Nov. 21, 2002, and the work completed on Dec. 12, 2002.

Figure 3 summarizes all measurements in the test area from Sept. 29, one month after the end of filling and about two months prior to installation of sand drains till end of Feb. These measured settlements are only part of the total settlement, since these do not include the settlement part that occurred during the filling operation.

Observation made since the construction of drains can be analyzed by the Asaoka's method [4]. It is evident that on Feb. 28, the consolidation degree reached on any plate is higher than 99%. The estimated final settlement is between 0.18 m and 0.28 m. It can be confirmed that the total settlement achieved is nearly 100% of the estimated value. This means that around 0.4 to 0.3 m of the total settlement (or roughly 50% of the total settlement) occurred during the filling period itself and before the settlement plates were installed.



Appendix 1: Graphic Method for Determining the Sizes of Vertical Sand Drains (Ref. 5)

CONCLUSIONS

The sand drain trial test has demonstrated the validity of design and proved the efficiency to get the required 95% degree of consolidation within the imposed time period. Based on this experimental evidence, the concept was adopted for the entire project area and 100 % expected results were achieved.

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NOTATIONS

- A_v = Coefficient of compressibility ($\Delta e/\Delta p$)
- M_v = Coefficient of volume compressibility = $A_v/1+e_o$
- Atterberg's limits = Liquid limit (W_L), plastic limit (W_p) and plasticity index (PI)
- K = Coefficient of permeability (m/sec)
- r_w = Unit weight of water (pcf, Kn/m^3)
- Es = Modulus of deformation (ksf or MPa)
- C_v = Coefficient of consolidation (m^2/s)
- Cc = Compression index (void ratio Vs log. pressure)
- C'c = Compression ratio (compressibility) = $Cc/1+e_0$
- e_0 = Average in situ void ratio in the stratum for which Cc applies.
- Cr = Re-comp. index
- C'r = Re-comp. ratio
- Grain size D85 = 1.1 mm implies size for which 85% of sample is smaller and is about 1.1 mm
- SPT = Standard penetration test (ASTM D1586), no. of blows/12 inches or 30 cm penetration.
- Wn = Insitu (natural) water content.