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Evaluation of Coefficient of Consolidation in CH Soils

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ABSTRACT

The present investigation deals with an evaluation and comparison of C_v of CH soils using three methods: Casagrande logarithm of time fitting method, Taylor's square root of time fitting method and inflection point method. The soils considered in this study were collected from different places around Hyderabad, Andhra Pradesh, India, belonging to intrinsic class CH soils. Approximately, both Casagrande and Taylor's methods presented the same range of C_v values in comparison to the inflection point method. In addition, empirical relations for swell index (C_s) with compression index (C_c) and plasticity index (PI) with finer fractions were obtained. The relation between applied pressure and C_v values obtained from various methods was also derived. Finally, the study results revealed that Casagrande and Taylor's methods predict comparable values of C_v for CH soils and hence can be recommended in view of simplicity.

KEYWORDS: Consolidation, Taylor's method, Casagrande method, Inflection point method.

INTRODUCTION

Consolidation is one of the important processes of geotechnical engineering. It deals with the time rate of settlement. Based on Terzaghi's one - dimensional consolidation theory, several curve-fitting methods exist to interpret the laboratory oedometer test for the evaluation of coefficient of consolidation (C_v). Consolidation has crucial importance, especially in foundation areas, to predict/judge the future settlement caused due to loads imposed on structures. Soils experience deformation in the field due to various parameters, such as intrinsic assumptions involved in the consolidation theory. Principally, compressibility and permeability of soils induce changes in consolidation coefficient and *in situ* settlements are exaggerated by numerous variables such as: drainage condition, thickness of soil layer and macro-structure characteristics of the stratum. These aspects may not be taken into account in the laboratory during testing. Therefore, differences between predicted behavior and real behavior of soils are anticipated. It is an obligation to predict the exact time rate of settlement, as this gives an idea about the fortune of the structure.

The consolidation coefficient mainly depends on the selection of the initial dial gauge reading corresponding to the zero primary consolidation, which requires values from a family of curves involved much in subjectivity (Scott, 1961). Sometimes, initial or final dial gauge readings may lead to errors or produce improper values of C_v. To avoid uncertainty, two new diagnostic curve methods are available for reliable identification of the coefficient of consolidation and the range of primary settlement. These methods do not require implicit or explicit determination of initial or final dial gauge readings. The time factor T and the degree of consolidation U relationships from one –

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dimensional consolidation theory have shown to be in the form of a rectangular hyperbola over the degree of consolidation range of 60% - 90%. The similarities between theoretical *T/U versus T* and experimental *t/δ versus t* plots (where *t* is time and δ is the compression) are used to present a simple method for the evaluation of coefficient of consolidation based on non-ideal conditions. If the readings are available at long or unequal time intervals, a reliable estimation of the coefficient of consolidation cannot be obtained. As a solution, a finite – difference approximation was proposed to locate the inflection point (Robinson, 1997; Mesri et al., 1999; Cour, 1971). This method does not require the definition of beginning and end of the primary consolidation stages that are required by other methods over an average degree of consolidation of 70%, which is within the midrange of the compression curve and is slightly affected by the initial and secondary compressions (Mesri et al., 1999). A simple and reliable user friendly approach proposed for the evaluation of the coefficient of consolidation without applying elaborative curve – fitting methods. The percentage deviations of C_v values obtained by this method from those obtained by using other methods are much less than the large variations observed between values of C_v obtained from the popular methods of Taylor and Casagrande (Sridharan and Prakash, 1998).

From the aforementioned literature review, it is noticed that many researchers focused on the evaluation of consolidation coefficient using various methods. Many important observations were brought out about the factors influencing the consolidation coefficient, such as permeability and compressibility of soil. Generally, Taylor’s method yields higher values of C_v than those obtained from Casagrande method. If the trends of laboratory time fitting curves are exactly similar to the hypothetical shapes, Casagrande and Taylor’s methods would result in the same value of C_v. However, due to the fact that clay compressibility varies with effective stress and stress rate of strain, and perhaps due to other effects as well, the actual shapes diverge from the hypothetical shapes (Duncan, 1993). In spite of their wide acceptance, some difficulties are still experienced in real practice when using Casagrande and Taylor’s methods. Further details of soil samples and a comparison of the data have been discussed in the following sections.

METHODOLOGY

In this study, three available methods are considered for the evaluation of consolidation coefficient: Casagrande logarithm time fitting method, Taylor's root of time method and inflection point method. Most of the methods are using to evaluate C_v from the laboratory time - settlement curves and are based on Terzaghi’s consolidation theory. The existence of different procedures indicates that all methods can’t be used for the evaluation of consolidation coefficient under all circumstances. The values of C_v obtained by different methods may diverge drastically depending on the laboratory testing conditions. It is difficult to judge which value of C_v is a reasonable estimate for the soil.

Casagrande logarithmic time fitting method is one of the basic and simple approaches to determine the consolidation coefficient. The time for achieving 100% primary compression, t_{100}, and the corresponding deformation, R_{100}, of the sample is estimated by finding the intersection point of the line tangent to the straight line portion of the primary compression curve and the line tangent to the secondary compression curve. The initial time, t_0, or the starting of primary compression and the corresponding deformation, R_0, are deduced. The time required for 50% consolidation, t_{50}, can then be recognized as the midpoint between the 0% and 100% consolidation points on the deformation axis. The consolidation coefficient from Casagrande method can be defined as:

$$C_v = 0.197 \left(\frac{H_{dr}^2}{t_{50}}\right); \tag{1}$$

where H_{dr}^2 = Average longest drainage path during consolidation.

t_{50} = Average degree of 50% Consolidation.

Taylor's square root time fitting method is one of the simplest reliable methods to find the coefficient of consolidation. In this method, a tangent is drawn to the straight line, as a part of the curve and the abscissa, Δ, at some arbitrary deformation. A straight line is then drawn through the origin with an abscissa of 1.15Δ. The point of intersection of this line with the experiment curve is then taken to indicate the time, t_{90}, and the corresponding deformation, R_{90}, for 90% primary consolidation. The coefficient of consolidation from Taylor's method can be defined as:

$$C_v = 0.848 \left(\frac{H_{dr}^2}{t_{90}}\right); \quad (2)$$

where H_{dr}^2 = Average longest drainage path during consolidation.

t_{90} = Average degree of 90% consolidation.

One-point method is a simple method which can be easily adopted. Basically, one-point method can be expressed in the form of $\log_{10}$ (H^2/t) *vs*. U curves. It could be noticed that the effects of primary and secondary compressions are negligible in the range 40% < U< 60% and hence, the obtained C_v values in that range might be considered to represent the soil reasonably good. The important observation is that the experimental behavior of soil without considering the correction for initial and secondary compression effects competing well with the theory in the range 40% <U< 60% leads to the development of the method. Robinson (1997) has recommended using elapsed time at the inflection point, t_i, for computing the coefficient of consolidation from oedometer tests. The main advantage of this method is that it doesn't require the definition of the beginning and end of the primary consolidation stages that are required for other widely used methods (Casagrande and Fadum, 1940; Taylor, 1948). Another advantage of this approach is that the inflection point at the average degree of consolidation of 70% is within the midrange of compression curve and is least affected by initial compression and secondary compression. Therefore, when the inflection point is carefully identified, the computed C_v should be as reliable as the C_v value obtained from Casagrande procedure. The coefficient of consolidation from the inflection point method can be defined as:

$$C_v = 0.405 \left(\frac{H_{dr}^2}{t_i}\right); \quad (3)$$

where H_{dr}^2 = Average longest drainage path during consolidation.

t_i = Inflection point.

EXPERIMENTAL INVESTIGATION

Soils Used in the Study

In this study, soils were collected from various locations; namely: Ghatakesar-***G***, Shamshabad -***S***, Jeedimetla-***J***, Khushayiguda -***K***, Moulali – ***M*** and Cherlapalli – ***C****,* around Hyderabad, Andhra Pradesh (A.P.), India. Atterberg's limits were quantified and soils were grouped according to IS (Indian Standard) classification types. The soil samples collected from different places fall under the CH classification as per the IS classification system of soils. Belonging to intrinsic class CH, the selected six soil samples are confirmed to have major percentages of fine grained soil. Testing was carried out by maintaining the laboratory controlled conditions, and the samples were processed and stored in airtight containers. The basic characteristics of all six soil samples are presented in Table 1.

Table 1. Characteristics of soils used in the study

S. No.	Soil	Liquid limit (W_L), %	Plastic limit (W_P), %	Plasticity index (PI)	Specific gravity (G_s)	Gravel (%)	Sand (%)	Silt and Clay (%)	IS Classification
1	Ghatekeshar	68	36	32	2.67	0.62	9.38	90	CH
2	Shamshabad	64	24	40	2.65	1.6	10.4	88	CH
3	Jeedimetla	63	29	34	2.65	0.46	11.54	88	CH
4	Khushayiguda	70	31	39	2.67	0.94	9.06	90	CH
5	Moulali	70	39	31	2.68	1.22	7.78	91	CH
6	Cherlapalli	63	35	28	2.7	2	14	88	CH

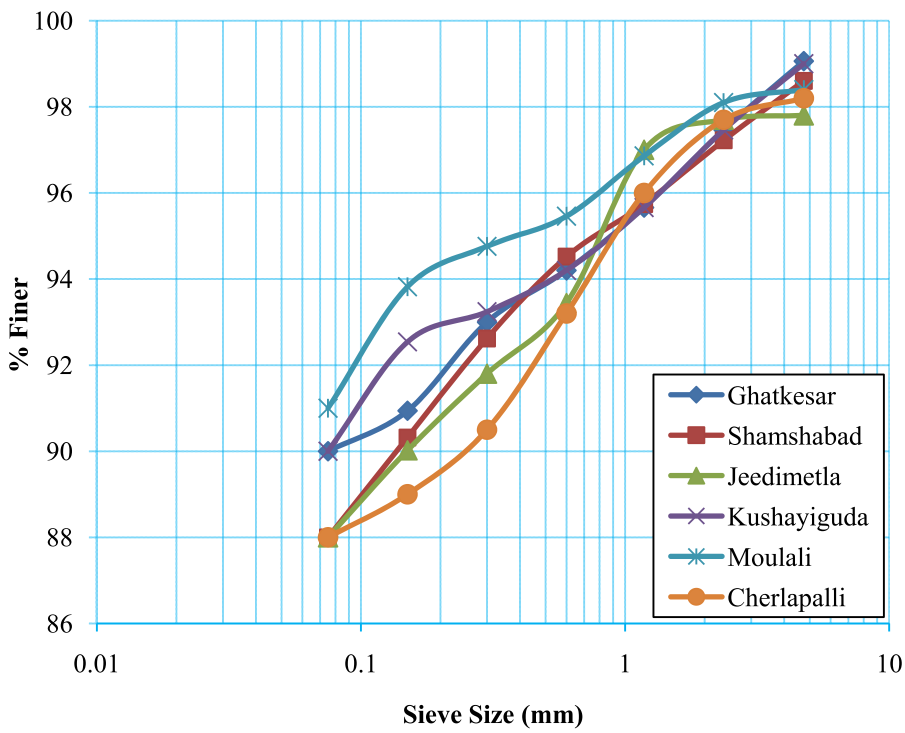

Figure (1): Grain size distribution curves for soils tested

The grain size distribution curves (sieve analysis) for all soils used in the study are presented in Fig. 1. From the grain size distribution of the soil samples, it is observed that the fines' content (passing 0.075 mm sieve) varies from 88% to 91%.

The compaction curves corresponding to the standard compaction for all samples tested are presented in Fig. 2. Values of OMC and MDD for soils are presented in Table 2. From the standard compaction results, it is noticed that the OMC values for all soils tested are varying from 17% to 20%, while the MDD values are varying from15 kN/m^3 to19.2 kN/m^3.

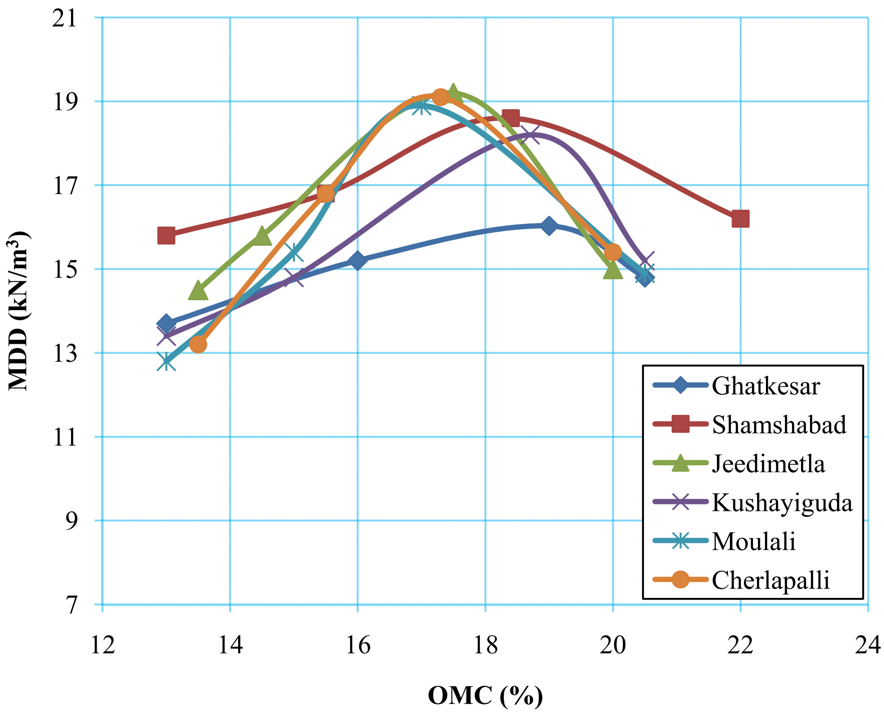

Figure (2): Standard proctor's compaction curves for soils tested

Table 2. Values of OMC and MDD for soils tested

S. No.	Soil	OMC (%)	MDD (kN/m^3)
1	Ghatekeshar	19	16.3
2	Shamshabad	20	15
3	Jeedimetla	17.5	19.2
4	Khushayiguda	18.7	18.2
5	Moulali	17	18.9
6	Cherlapalli	17.3	19.1

Tests Conducted

The tests of basic characteristics of soils tested were conducted as per the Indian Standard Code of Practice of Testing of Soils as mentioned below. Liquid limit (LL) and plastic limit (PL) tests were conducted as per IS: 2720 (Part 5) - 1985. Grain size distribution test was performed as per IS: 2720 (Part 4) – 1985. Standard proctor compaction tests were carried out according to IS: 2720 (Part 8)-1983. The determination of consolidation properties was carried out as per IS: 2720 (Part 15) - 1965.

RESULTS AND DISCUSSION

Based on Terzaghi's one-dimensional consolidation theory, many curve-fitting methods are available to interpret the laboratory oedometer compression for the determination of the coefficient of consolidation. In this study, three different methods for the determination of C_v have been evaluated and the values compared. The difference in the values of C_v determined from the laboratory test is sometimes large and depends on the method used. Typical laboratory consolidation tests were conducted in the laboratory as per the Indian Standard Code.

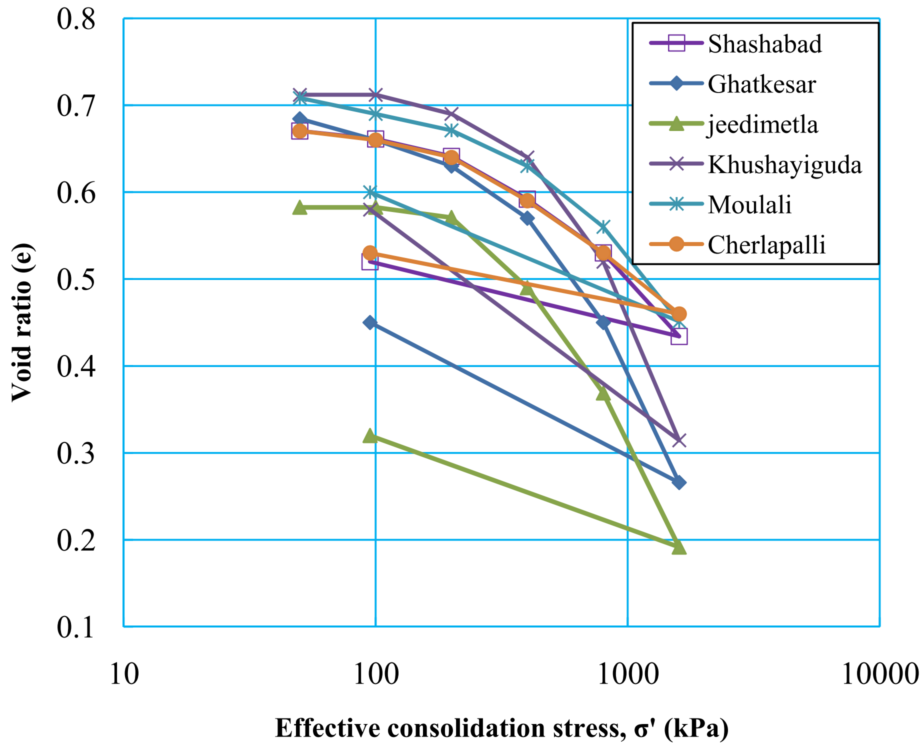

Figure (3): e - log σ' curves for soils tested

e - log σ' Curves for Soils

Fig. 3 shows the relation between the void ratio (e) and the applied effective consolidation stress (σ') for all six samples considered in the study. From the figure, it can be seen that as the effective consolidation stress increases the void ratio decreases. After achieving a certain point, the trend followed a straight line. All soils followed the similar trend for the different applied pressures ranging from 50 kPa to 1600 kPa. Corresponding to the applied pressures, void ratios are varying from 0.68 to 0.26, 0.67 to 0.43, 0.58 to 0.19, 0.71 to 0.31, 0.70 to 0.45 and 0.67 to 0.46, respectively for all six soil samples. The values of swell index (C_s) and compression index (C_c) are evaluated from virgin compression curves.

Comparison of C_v Values Obtained from Various Methods

The C_v values obtained from various methods used in the study are compared in this section.

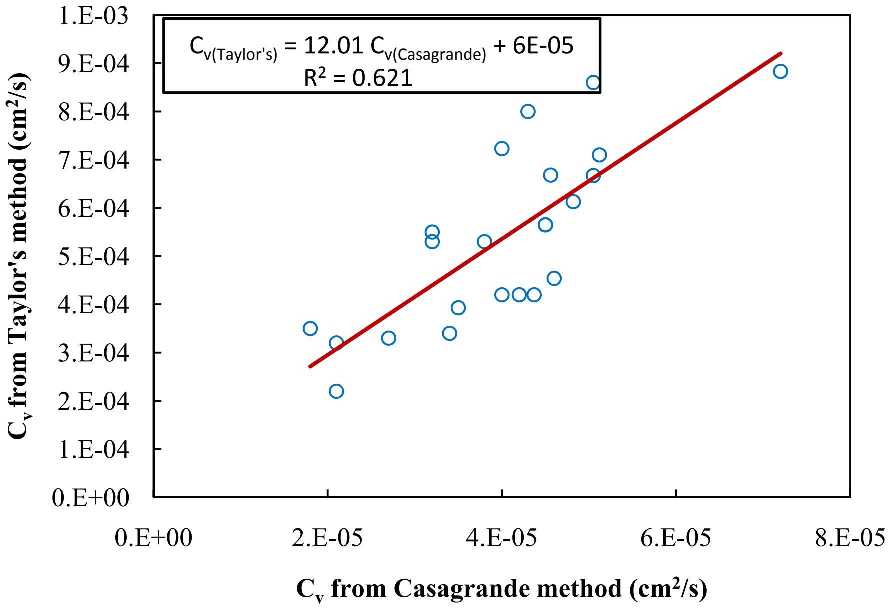

Figure (4): Comparison of C_v values obtained from Taylor's method with those obtained from Casagrande method

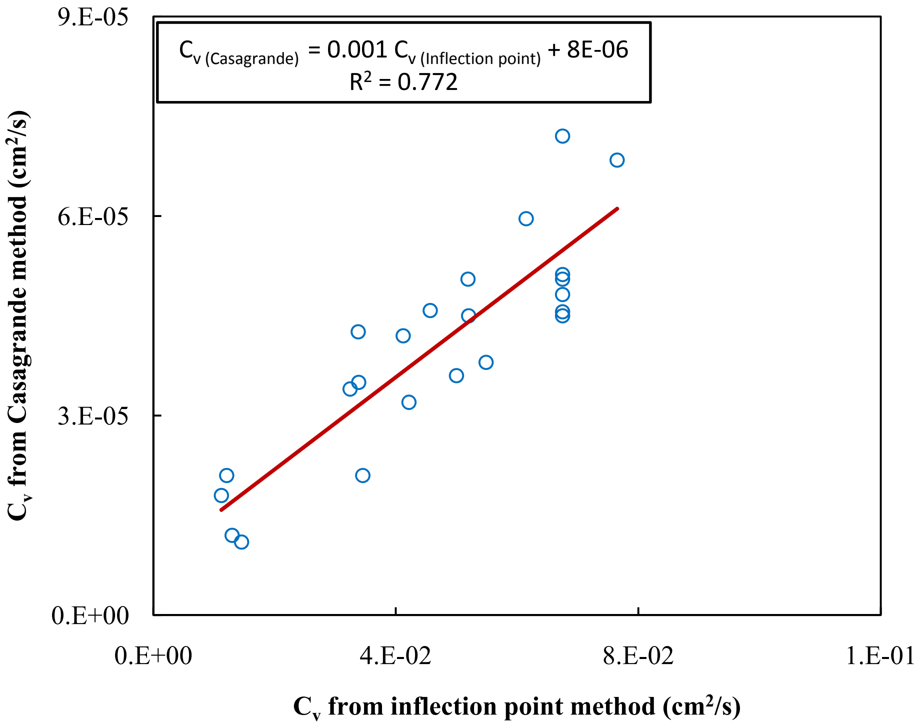

Figure (5): Comparison of C_v values obtained from Casagrande method with Those obtained from inflection point method

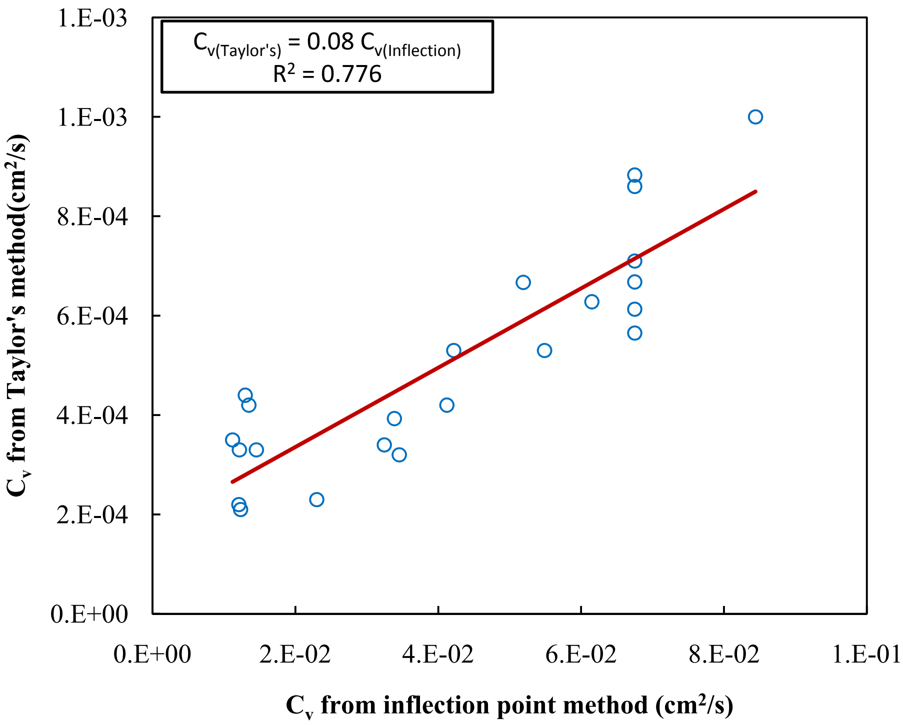

Figure (6): Comparison of C_v values obtained from Taylor's method with Those obtained from inflection point method

Figs. 4 - 6 show a comparison of C_v values obtained by Taylor's –Casagrande, Casagrande – Inflection point and Taylor's – Inflection point methods, respectively. The relationship between the methods for C_v determination was derived. Fig. 4 shows the comparison of C_v values obtained by Taylor's method and Casagrande method. The consolidation coefficient values are following almost a similar range and those values appear to be good. The C_v values are satisfying the power variation law with a regression coefficient of 0.621 for Taylor's - Casagrande methods. The relation between the Taylor's – Casagrande methods is presented in Eq. 4. Similarly, Figs. 5 and 6 show the comparison of Casagrande – Inflection point and Taylor's –Inflection point methods, respectively. The comparison between Casagrande – Inflection point methods is satisfying a regression coefficient of 0.772. The comparison between Taylor's – Inflection point methods is satisfying the power variation law with a regression coefficient of 0.776. The relationships between Casagrande – Inflection point methods and between Taylor's – Inflection point methods are presented in Eq. 5 and Eq. 6, respectively.

$$C_{v\,(Taylor's)} = 12.01\ C_{v\,(Casagrande)} + 6E\text{-}05 \quad R^2 = 0.621. \tag{4}$$

$$C_{v\,(Casagrande)} = 0.001\ C_{v\,(Inflection\ point)} + 8E\text{-}06 \quad R^2 = 0.772. \tag{5}$$

$$C_{v\,(Taylor's)} = 0.08\ C_{v\,(Inflection\ point)} \quad R^2 = 0.776. \tag{6}$$

Variations of C_v with Applied Pressure

It is an accustomed practice in highly compressible soils to plot the obtained C_v as a function of applied effective vertical consolidation pressure (σ'). For all the soils, variations of consolidation coefficient with the applied pressure were observed. Almost all the soils are following a similar trend. The variations in regression coefficients are found, and shown in Table 3.

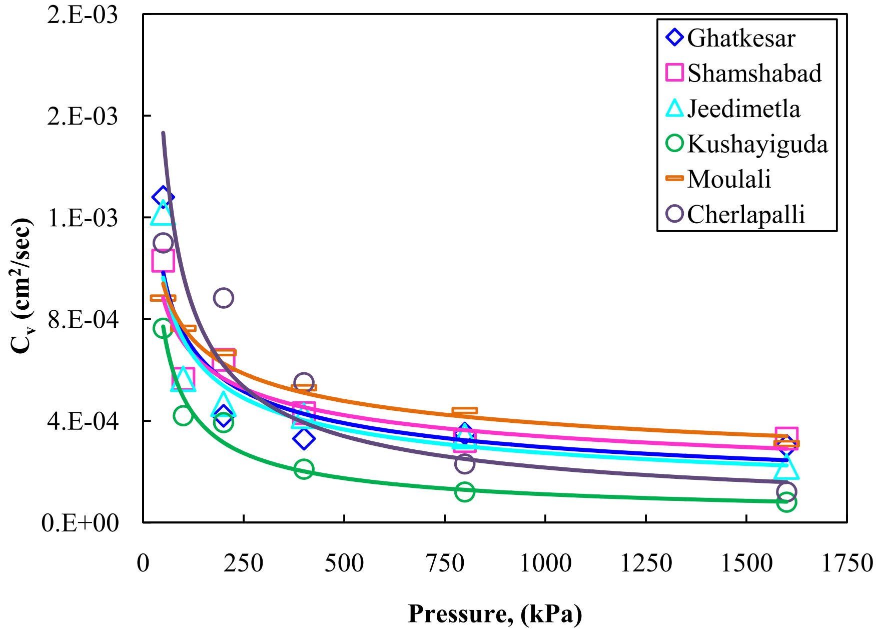

Figure (7): Variation of coefficient of consolidation (C_v) obtained by Taylor's method with applied pressure for soils tested

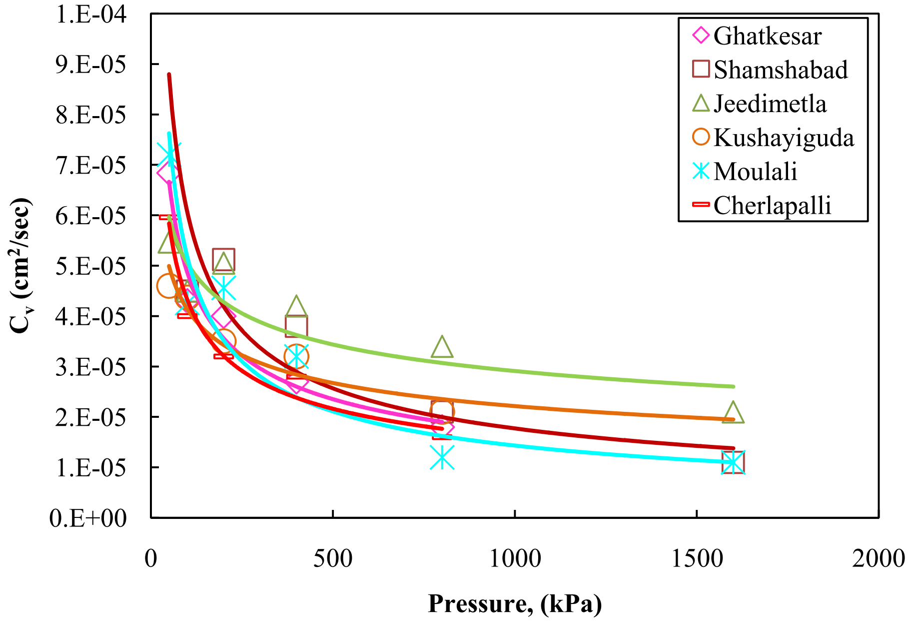

Figure (8): Variation of coefficient of consolidation (Cv) values determined by Casagrande method with applied pressure for soils tested

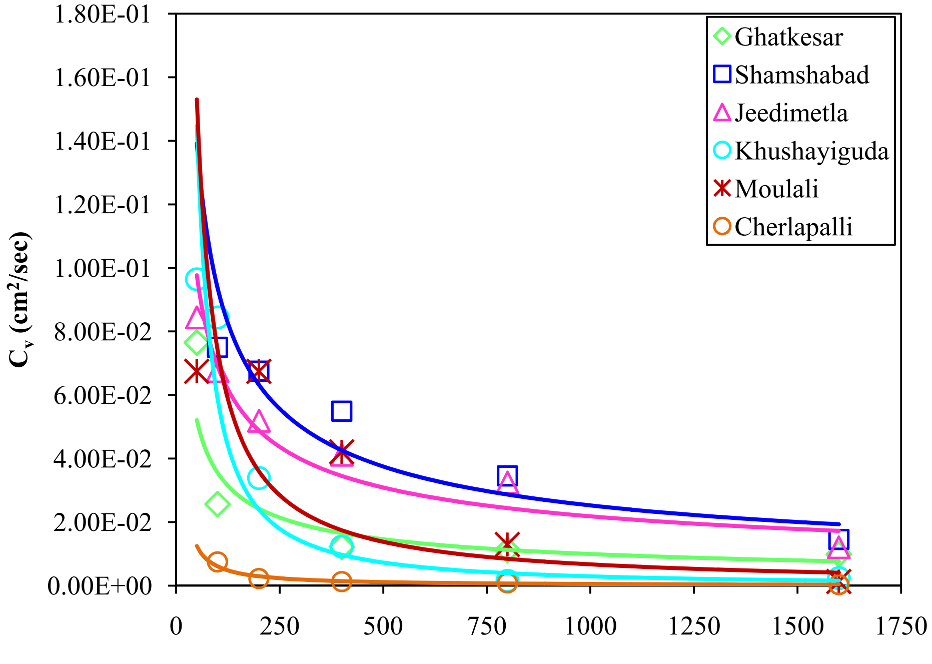

Figure (9): Variation of coefficient of consolidation (Cv) values determined by inflection point method with applied pressure for soils tested

Figs. 7 to 9 show the variation of consolidation coefficient obtained by methods considered in the study with applied effective stress. The applied effective stresses have been considered to be from 50 to 1600 kPa. The relation of consolidation coefficient and applied pressure, as well as the regression coefficients are presented in Table 3. All soils followed the power variation law for the relationship between consolidation coefficient and applied pressure. The consolidation coefficients obtained from Taylor's method with applied pressure are between 0.807 and 0.977, whereas regression coefficients obtained from Casagrande method are between 0.792 and 0.966 and those obtained from inflection point method are between 0.801 and 0.966.

Table 3. Best fits for obtained Cv values from different methods with applied pressure

S.No.	Soil Name	Taylor's method		Casagrande method		Inflection point method	
		Equation	Regression coefficient	Equation	Regression coefficient	Equation	Regression coefficient
1	Ghatkesar	$y = 0.004x^{-0.40}$	0.807	$y = 0.000x^{-0.45}$	0.966	$y = 0.453x^{-0.55}$	0.860
2	Shamshabad	$y = 0.003x^{-0.32}$	0.875	$y = 0.000x^{-0.53}$	0.842	$y = 1.293x^{-0.57}$	0.869
3	Jeedimetla	$y = 0.005x^{-0.42}$	0.907	$y = 0.000x^{-0.24}$	0.792	$y = 0.691x^{-0.5}$	0.888
4	Kushayiguda	$y = 0.009x^{-0.64}$	0.977	$y = 0.000x^{-0.27}$	0.904	$y = 23.19x^{-1.29}$	0.905
5	Moulali	$y = 0.003x^{-0.29}$	0.971	$y = 0.000x^{-0.55}$	0.901	$y = 9.124x^{-1.04}$	0.801
6	Cherlapalli	$y = 0.019x^{-0.65}$	0.875	$y = 0.000x^{-0.43}$	0.954	$y = 0.723x^{-1.03}$	0.966

y = coefficient of consolidation (C_v) in cm^2/s and x= applied pressure on the sample in kPa.

Relationships among LL, C_s, C_c and % Finer

The coefficient of consolidation is a parameter that indicates the rate of compression of a saturated soil undergoing compression, which in turn directly depends on the hydraulic conductivity of the soil medium undergoing compression. The relationships between LL - % finer, C_s – LL, C_c – LL and C_c – C_s are furnished in the following figures.

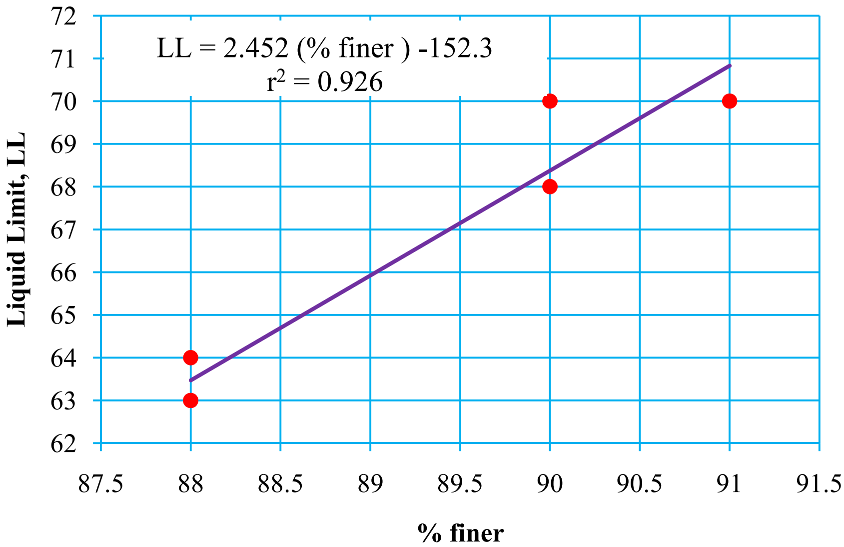

Figure (10): Relationship between % finer and liquid limit for soils tested

Fig. 10 shows the relationship between % finer and liquid limit for soils considered in the study. The fine particles (< 0.075 mm) are ranging between 88 % and 91% and liquid limit values are ranging between 63 % and 70% for all soils used in the study. The soils considered in the study are satisfying linear variation with a regression coefficient of 0.926 and the relation between them is presented in Equation 7.

$$LL = 2.452\ (\%\ finer) - 152.3$$
$$R^2 = 0.926. \quad (7)$$

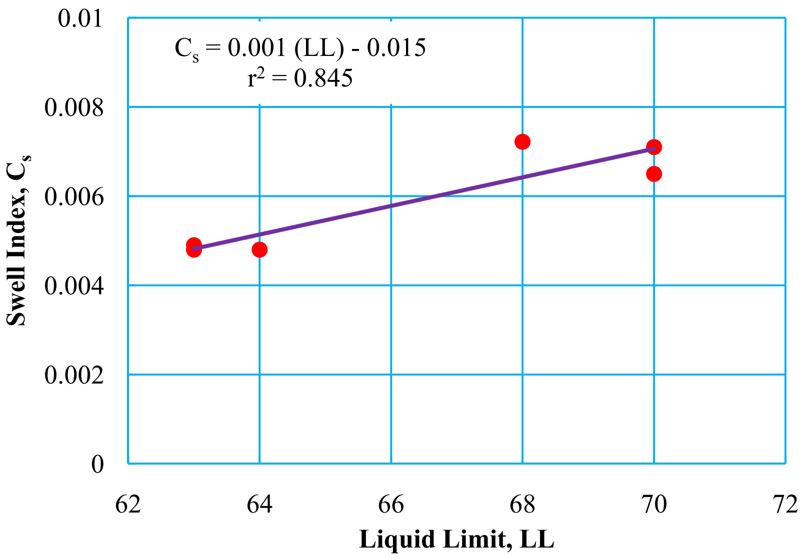

Figure (11): Relationship between liquid limit (LL) and swell index (C_s) for soils tested

Fig. 11 shows the relation between liquid limit and swell index for soils used in the study. The swell index (C_s) values are varying between 0.0048 and 0.0072 and liquid limit values are varying between 63% and 70% for the soils tested. The variation of liquid limit with swell index shows a linear trend having a regression coefficient of 0.845. The relation between them is presented in Eq. 8.

$$C_s = 0.001\ (LL) - 0.015$$
$$R^2 = 0.845. \tag{8}$$

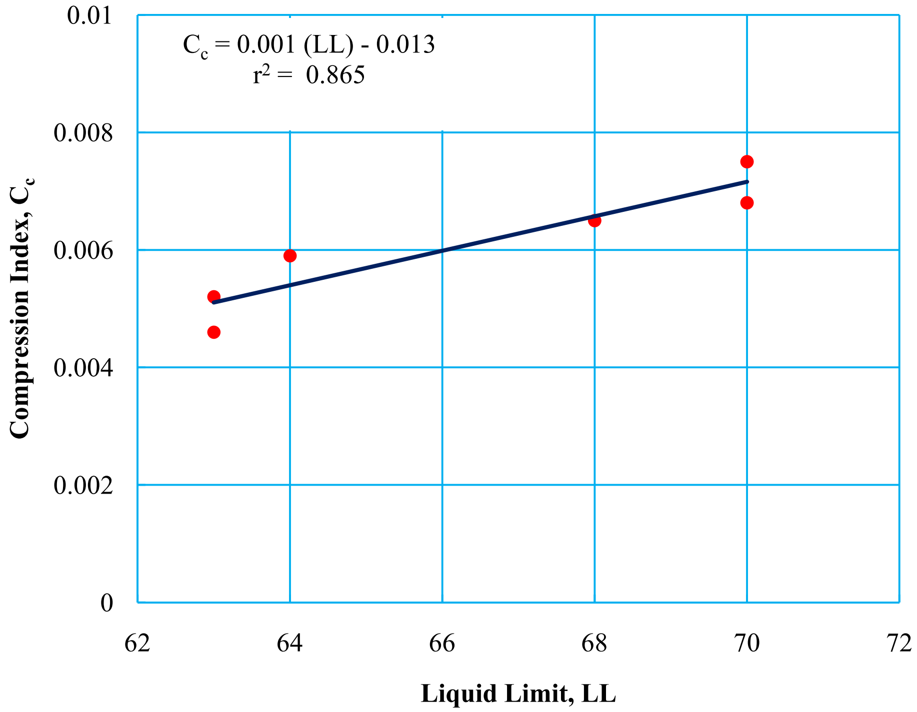

Figure (12): Relationship between compression index (C_c) and liquid limit (LL) for soils tested

Fig. 12 shows the relationship between compression index and liquid limit for soils used in the study. The compression index (C_c) values are varying between 0.0046 and 0.0068 and liquid limit values are varying between 63% and 70% for the soils tested. The variation of compression index with liquid limit shows a linear trend having a regression coefficient of 0.865. The relation between them is presented in Eq. 9.

$$C_c = 0.001\ (LL) - 0.013$$
$$R^2 = 0.865. \tag{9}$$

Fig. 13 shows the relationship between swell index (C_s) and compression coefficient (C_c) for soils used in the study. The coefficient of compression (C_c) values are varying from 0.0048 to 0.0072 and the coefficient of compression (C_s) values are varying from 0.0046 to 0.0075. The coefficient of compression with swell index shows polynomial variation with a regression coefficient of 0.773. The relation between them is presented in Eq. 10.

$$C_s = -960.7\ (C_c)^2 + 12.13\ (C_c) - 0.031$$
$$R^2 = 0.773. \tag{10}$$

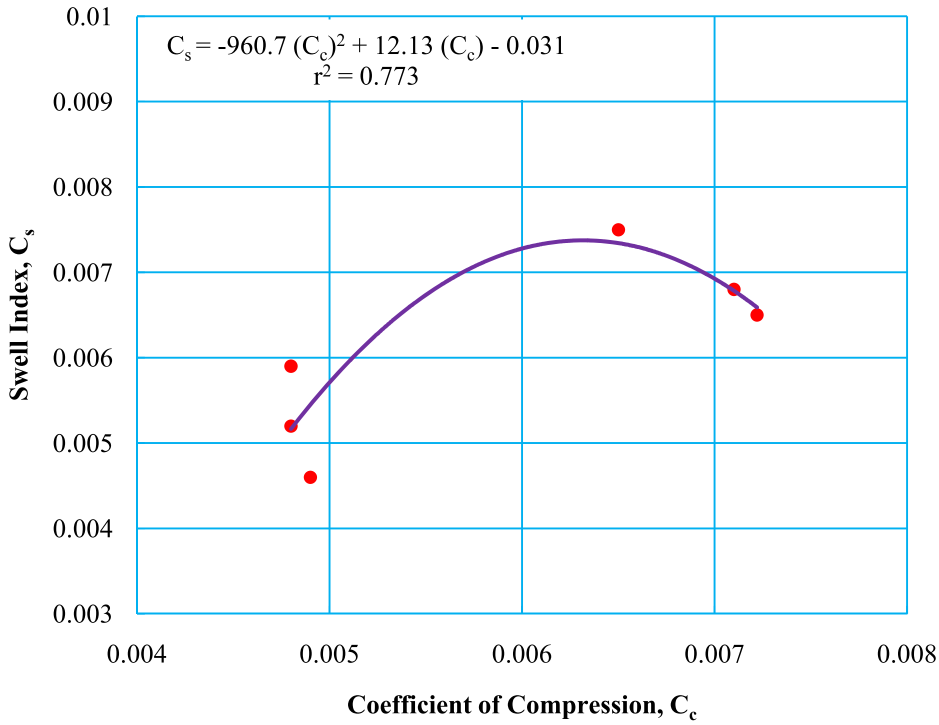

Figure (13): Relationship between swell index (C_s) and compression coefficient (C_c) for soils tested

CONCLUSIONS

Laboratory results show that Casagrande method in some cases may not determine values comparable with those obtained from other methods. As mentioned in the literature review, the rate of settlement in the field is faster than that evaluated experimentally. This is because the factors affecting the rate of settlement in the field are not considered experimentally. Therefore, it is better to choose a method that gives the highest value of C_v. This method is usually neither affected by the initial compression part of the consolidation curve nor by the secondary compression part of the consolidation curve. From the fitting curves for each soil type, the following conclusions may be drawn.

- For all the soils tested, regarding the variations of consolidation coefficient with the applied pressure, a similar trend was almost followed.
- Approximately, both Casagrande method and Taylor's method give the same range of C_v values in comparison to the inflection point method.
- Casagrande and Taylor's methods can be recommended in view of their simplicity.

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