A procedure to analyze a one dimensional compression test

Jorge Abraham Díaz-Rodríguez
[1]*National University of Mexico, Civil Engineering Department, Av. Universidad 3000, 04510, Mexico*
[1]*jadrdiaz@unam.mx*

ABSTRACT

The subsoil of Mexico City (SMC) is highly compressible when loads are applied. This feature is associated with many uncertainties and complex issues during foundation design processes. The compressibility of the SMC has been investigated since the ´40s by using oedometer tests in most cases. However, the importance of the results in interpreting compression test data requires a systematic method of determining the end of consolidation (EOC) under a load increment. This paper offers an alternative view to analyze a one-dimensional compression test, decomposing the time-compression curve into two curves, one due to consolidation and another due to volumetric creep, using numerical processing. This approach has several advantages: (1) Data reduction allows us to know the contribution of consolidation and volumetric creep at any time and for each load increment. (2) a systematic method of determining the end-of-consolidation (EOC) under a load increment, and (3) long-term strains (settlements) can be estimated at any time. Finally, an example of the results obtained using the suggested method is presented.

Keywords: Mexico City; one-dimensional compression test; consolidation; creep

1. Introduction

Soil is a natural, particulate, and multiphasic material. The application of stresses on a given soil causes deformations. Nature and magnitude depend on the soil type, stress applied, stress path, rate of strain, load increment ratio (LIR), and load increment duration (LID), among others.

This paper offers an alternative view to analyze a one-dimensional compression test. This approach supports the view that the information in a time-compression curve is the soil response to an application of stress through two separate but interrelated phenomena: consolidation and volumetric creep. Then, the procedure consists of decomposing the time-compression curve into two curves, one due to consolidation and another due to volumetric creep using numerical processing.

Since the Terzaghi consolidation theory, there has been controversy about whether or not creep acts as a separate phenomenon during primary compression, while excess pore pressure dissipates during the compression of thick layers of clay. Taylor (1940); Ladd (1973); Mesri and Rakhsar (1974); Leonard (1977); Ladd et al. (1977), and Mesri (2003), among others, consider that secondary compression develops after primary compression is completed. In contrast, Taylor (1942); Suklje (1957); Barden (1965, 1969); Bjerrum (1967); Leroueil and Kabbj (1987); Leroueil (2006) have expressed the idea that there is a combination of both processes.

It has been shown by various authors (Crawford, 1953; Leonard and Ramiah, 1959; Hamilton and Crawford, 1959; Barden, 1965 and others) that under specific test conditions, the predictions of Terzaghi consolidation theory have shown an imperfect correlation between computed and observed settlements. Crawford (1964); Barden (1965), and Bjerrum (1967) have argued that the division of compression into primary and secondary components, at least as it is currently done, is an arbitrary border of a continuous compression process, and the relative contribution of each component, it is not clearly defined.

Observation of the consolidation process, both in the field and in the laboratory, has demonstrated that volume changes continued to occur after excess hydrostatic pressure had essentially dissipated. According to Leonards (1977), differences between field and laboratory curves result from secondary compression and other effects as the rate of effective stress increases, not considered by Terzaghi´s theory.

Currently, empirical methods can be used to calculate C_v. Casagrande and Fadum (1940) proposed the log time method, and Taylor (1942) the root time method. These empirical procedures were developed to fit the observed laboratory test data to Terzaghi´s consolidation theory.

In their state-of-the-art report, Ladd *et al*. (1977) proposed two creep hypotheses called creep hypotheses A and B. Hypothesis A assumes that creep occurs only after the end pore pressure dissipation; that is, that the "end-of-primary" (EOP) strain is the same irrespective of the thickness of the consolidating soil layer. Hypothesis B assumes that creep co-occurs during pore pressure dissipation, which predicts an increment in EOP strain with an increasing consolidation period due to creep effects.

Mesri and Choi (1985) investigated the behavior of thin and thick specimens; their results support the concept of a unique EOP void ratio-effective vertical stress relationship of clay and silt deposits. Mesri and Funk (2015) made detailed settlement analyses for the Kansai Airport islands based on the assumption of the uniqueness of the EOP $e - \sigma_v'$ relationship, concluding that the phenomenon at Osaka Bay of the airport reclamation can be explained according to the conventional concept of primary compression followed by secondary compression.

Degago *et al.* (2009) critically evaluated the validity of the uniqueness of the EOP strain concept and concluded that the EOP strain-effective stress relationship is not unique. Degago *et al*. (2013) provided explanations using a consistent framework as to why hypothesis A seemed to be wrongly substantiated and clearly showed that there are definitive data to demonstrate that the creep hypothesis B agrees very well with the measured behavior of clayey soils.

Concluding this section, it may be said that both hypotheses seem experimentally supported, leading to confusion about the correct one.

It is not the intention herein to discuss-existing theories but to suggest a procedure for analyzing a one-dimensional compression test. It is realized that the behavior of natural material is highly complex, and a description of the deformation often becomes a questionable task. In this paper, strains in a one-dimensional test are the result of three processes:

- **Immediate or initial strains** (δ_i) that occur simultaneously with load application.
- **Consolidation** strains (δ_c) that occur during the change in effective stress due to pore pressure dissipation over time, inducing soil volume reduction as a consequence of water expulsion from void spaces. This process is time-dependent.
- **Volumetric creep** is used to denote a process of soil volume reduction controlled by the structure´s viscous resistance of the soil over extended periods under constant load. This process is time-dependent.

Volumetric creep in soils will be considered here as the process occurring from the beginning of loading, during and after the consolidation. Both process consolidation and volumetric creep can occur.

2. The Subsoil of Mexico City

The SMC is unique in the context of most other natural soils (Díaz-Rodríguez, 2003). The grain size distribution of Mexico City soils corresponds to silty clays or clayey silts. The water content, void ratio, and plasticity are typically very high ($w \approx 220 - 430\%$; $e \approx 5 - 10$; $w_L \approx 140 - 380$; $w_P \approx 55 - 112\%$). A characteristic of MC´s soil is that it is diatomaceous soil (Diaz-Rodriguez, 2003). The SMC's open structure created by diatoms is reinforced by the salty groundwater, which has a flocculating effect on the smectite-rich clay minerals. The normalized strength properties of the SMC vary with the yield stress σ_Y' and with the diatom content (Díaz-Rodríguez and Santamarina, 2001). While the friction angle of the soil decreases, as the plasticity index increases, the high plasticity of the SMC presents a friction angle ($\varphi = 43° - 47°$) comparable in magnitude to those of sands (Díaz-Rodríguez et al. 1992). Diatom content has a significant effect on the strength and stiffness of these soils (Díaz-Rodríguez, 2011). Another feature of soils with high diatom content is the low excess pore pressures under undrained shearing, both monotonic and cyclic. Furthermore, significant degradation of this diatomaceous soil only occurs once cyclic shear stresses exceed about 80% of the static undrained strength (Díaz-Rodríguez, 1989).

The shear wave velocity (V_s = 70 - 90 m/s) of SMC is relatively constant with depth in the upper sequence (7 to 30 m depth), and the yield pressure, σ'_y is higher than the *in situ* effective stress σ'_{v0}, in agreement with the stress-independent formation process (Díaz-Rodríguez et al. 1998). Diagenesis has led to apparent preconsolidation.

3. Material and testing procedure

SMC samples for the test were obtained from the lake zone of Mexico City (RCDF 2004); the depth of the samples ranged from 8.1 to 9.4 m. The soil samples have the following average properties: natural water content of 356%; void ratio of 8.7; and yield stress, σ_Y' of 85 kPa. The soil samples were recovered using Shelby tubes (OD = 128 mm ID = 125 mm, area ratio 4.9%). Each tube was X-rayed, and no evidence of cracks or edge effects was found.

Two one-dimensional compression tests were carried out on 63.3 mm diameter and 25.4 mm high soil specimens through the incremental load (IL). Test A using a typical load schedule of 20, 40, 80, 160, 320, and 640 kPa, (load increment ratio, $LIR = 1$). Test B uses equal stress increments of 10 kPa from 10 to 100, and then 150, 200 and 400 kPa. The 24h compression curve was used to estimate the vertical yield stress, as conventionally done.

The equipment used was similar to that described by Head (1982). Preparing specimens for the oedometric tests was obtained by trimming to a diameter close to the size of the device-cutting ring using a fine wire, and then the ring, with a sharp-edged, was pushed inside the recovered soil sample. The specimens were separated from the porous stones with a single thickness of Whatman No 50 filter paper. Calibrations were made of each apparatus to reduce the effects of compliance in the measurement of sample displacement.

4. Suggested procedure

According to Barden (1969), it is helpful to represent the assumed mechanism responsible for compression in soils in terms of a rheological model, allowing the behavior of an element to be expressed in mathematical terms. Then, the rheological model proposed by Zeevaert (1986) was chosen to fit the displacement-time curve of each load increment in the tests by the theoretical equation:

$$\delta = \delta_v F(T_v) + C_t \log(1 + \xi T_v) \quad (1)$$

The term $\delta_v F(T_v)$ represents the displacement due to consolidation with time associated with an effective stress increase, where δ_v represents consolidation compression, and T_v is the time factor. The term $C_t \log(1+\xi T_v)$ represents the displacement due to creep with time, where C_t represents the final slope of log behavior, and ξ is a dimensionless parameter.

Numerical data processing was carried out through a computer program developed based on Zeevaert´s (1986) fitting method, but incorporating features such as (1) using the coefficient of determination R^2 as a parameters choice criteria, (2) The fit was improved using the Nelder-Mead (1965) optimation algorithm. The criteria to determine the end-of-consolidation (EOC) was to use a time factor, $T_v = 2$ (i.e., 99.42% degree of consolidation).

The results of the procedure followed to analyze a one-dimensional compression test is illustrated in Figures 1 to 2, and 4 to 5 which show a set of curves of time-vertical displacement from a compression test for effective vertical stress of $\sigma_v' = 40$ and 160 kPa, respectively. Every set of curves of Figures 1 to 2, and 4 to 5 consists of:

- Data points (full circles)
- The curve fitted by theoretical equation (curve 1)
- Consolidation curve (curve 2)
- Volumetric creep curve (curve 3).

This approach has several advantages: (1) Data reduction allows us to know the contribution of consolidation and volumetric creep at any time and for each load increment. (2) a systematic method of determining the end-of-consolidation (EOC) under a load increment, and (3) total strains (settlements) can be estimated at any time.

5. Results

5.1. Test A

Test A results are shown in Figures 1 to 3. A set of curves time-vertical displacement from a one-dimensional compression test for effective vertical stresses of $\sigma_v' = 40\ kPa$ (below yield stress) is shown in Figure 1. The soil specimen showed an immediate displacement of $\delta_i = 0.078$ mm for the load increment of 40 kPa. The consolidation curve (Curve 2) indicates that the end-of-consolidation (EOC) occurred in $t_{EOC} = 7$ minutes. The total displacement in 7 minutes was 0.328 mm, where 0.181 mm (55%) corresponded to consolidation and 0.069 mm (21%) corresponded to volumetric creep. After EOC, the specimen continues deforming under volumetric creep. At the end of 24 h, the sample exhibited a total displacement of 0.481 mm, a total creep displacement of 0.222 mm (46%).

Figure 2 shows the results for effective vertical stresses of $\sigma_v' = 160\ kPa$ (above the yield stress). The soil specimen showed an immediate displacement of $\delta_i = 0.1048$ mm. The consolidation curve (Curve 2) indicates that the end-of-consolidation (EOC) occurred in $t_{EOC} = 90$ minutes. The total displacement in 90 minutes was 3.60 mm, where 2.45 mm (68%) corresponded to consolidation and 1.05 mm (29%) corresponded to volumetric creep. After EOC, the specimen continues deforming under volumetric creep. At the end of 24 h, the sample exhibited a total displacement of 4.79 mm, a total creep displacement of 2.222 mm (61%).

The behavior below yield stress ($\sigma_v' < \sigma_Y'$) may be characterized by:

- The soil volume changes that can be attributed to volumetric creep are secondary compared with those that develop due to the gradual dissipation of excess pore pressure (consolidation).
- The volumetric creep occurs during the consolidation but reaches its maximum only after it is completed.
- After the end of the consolidation process (t_{EOC}) the change in volume is only due to volumetric creep.

The behavior above yield stress ($\sigma_v' > \sigma_Y'$) is similar to the behavior for $\sigma_v' < \sigma_Y'$, but is approximately ten times greater.

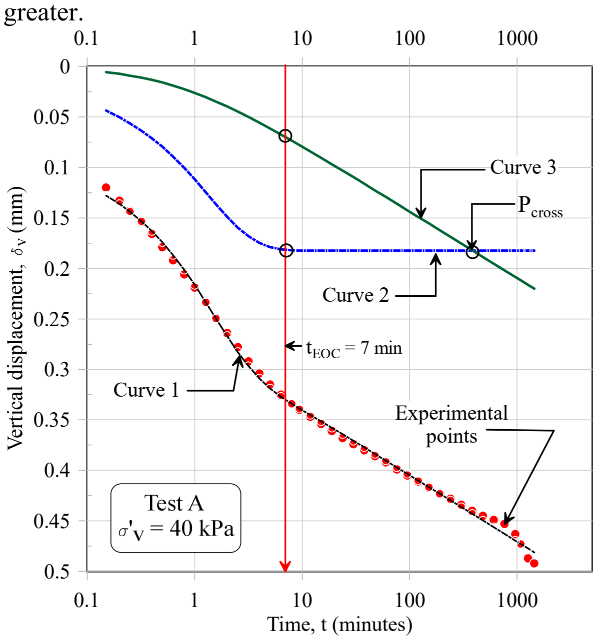

Figure 1. Time-vertical displacement curves from Test A on Mexico City soil under incremental loads: $\sigma_v' = 40\ kPa$ (below yield stress); LIR = 1; LID = 24 h.

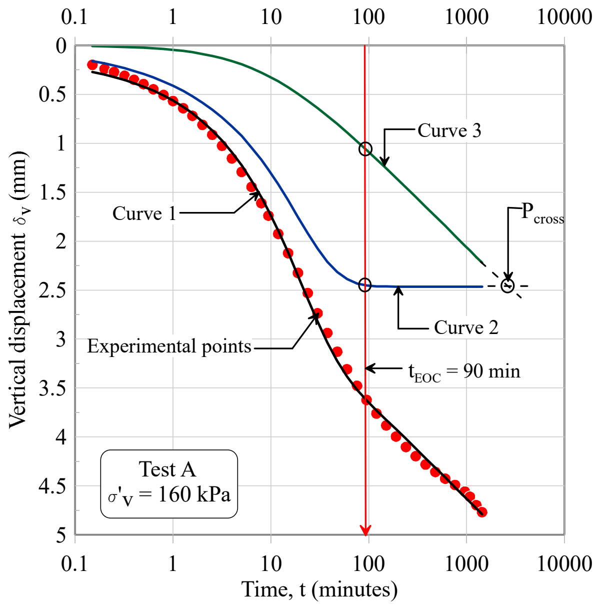

Figure 2. Time-vertical displacement curves from Test A on Mexico City soil under incremental loads: $\sigma_v' = 160\ kPa$ (above the yield stress); LIR = 1; LID = 24 h.

The summarized response patterns observed in soil specimen are shown in Table 1 and Figure 3. In Figure 3, results are presented in vertical strains, $\varepsilon_v = \delta/H$, where δ is the vertical displacement, and H is the height of the specimen before testing. The figure shows three curves: the upper curve shows the immediate strain for each increment; the intermediate curve indicates the compression at the end-of-consolidation (EOC) for each pressure increment. The lower curve represents the compression at the end of the loading period (LID), in this case, after 24 hours. The difference between the two last curves represents the contribution of creep under each increment. The results in Figure 3 show no difference from those obtained with the current practice (ASTM D 2435). Despite this result, this study offers an alternative view or interpretation that can provide a starting point for further examination or research

5.2. Test A

Test B using equal stress increments of 10 kPa from 10 to 100 kPa, and then 150, 200, and 400 kPa, were applied to know the effect of a change in the loading sequence; the results are shown in Figures 4 to 6, and Table 2. It can be seen that the consolidation is carried out more slowly for small increments than for large ones. Similar findings have been reported by Taylor (1942) and Crawford (1986).

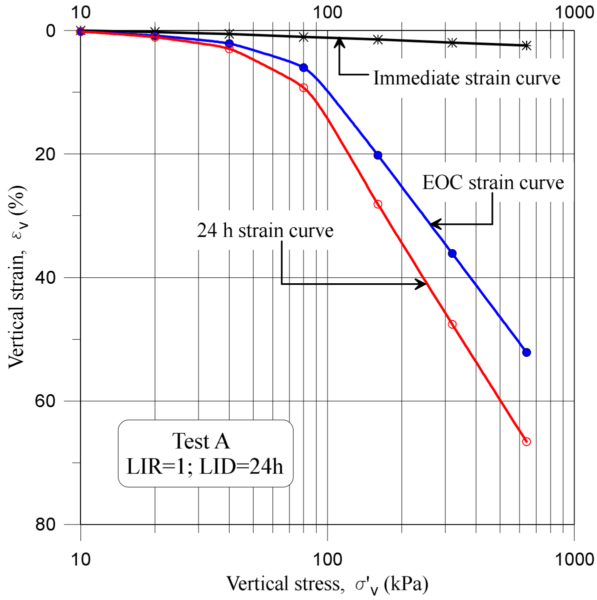

Figure 3. Strain-vertical effective stress relationships from Test A on Mexico City soil under incremental loads: LIR = 1; LID = 24 h.

Table 1. Summary of test results from Test A

Vertical stress	Time EOC	Displacement at end of consolidation				End of 24 h	
		Immediate	Consolidation	Creep	Total	Creep	Total
σ'_v (kPa)	Min	δ_i (mm)	δ_c (mm)	δ_{creep} (mm)	δ (mm)	δ_{creep} (mm)	δ(mm)
10	13.78	0.00077	0.037222	0.003541	0.041532	0.010266	0.048476
20	4.94	0.05835	0.087567	0.023994	0.16991	0.07982	0.22625
40	6.68	0.07838	0.181476	0.068774	0.328629	0.220145	0.481065
80	17.38	0.1301	0.521999	0.335995	0.988094	0.941733	1.596893
160	90.44	0.1085	2.449914	1.049485	3.604249	2.221814	4.790944
320	148.16	0.13044	2.939186	0.969381	4.039007	1.854318	4.941178
640	297.41	0.11657	2.773766	1.181919	4.072255	1.922444	4.829044

Table 2. Summary of test results from Test A

Vertical stress	Time EOC	Displacement at end of consolidation				End of 24 h	
		Immediate	Consolidation	Creep	Total	Immediate	Consolidation
σ'_v (kPa)	Min	δ_i (mm)	δ_c (mm)	δ_{creep} (mm)	δ (mm)	δ_{creep} (mm)	δ(mm)
20	56.468	0.004	0.104	0.026	0.134	0.059	0.168
30	60.869	0.024	0.094	0.052	0.17	0.119	0.237
40	55.619	0.021	0.062	0.079	0.162	0.184	0.268
50	54.817	0.016	0.081	0.059	0.156	0.198	0.296
60	68.535	0.01	0.092	0.05	0.152	0.222	0.325
70	78.783	0.011	0.093	0.054	0.158	0.283	0.388
80	101.028	0.011	0.096	0.065	0.172	0.373	0.48
90	104.685	0.009	0.088	0.083	0.18	0.508	0.606
100	108.372	0.008	0.078	0.095	0.181	0.603	0.689
150	606.328	0.04	1.248	1.438	2.726	1.924	3.219
200	826.69	0.025	0.71	1.06	1.795	1.288	2.027
400	384.046	0.063	2.486	0.969	3.518	1.475	4.039

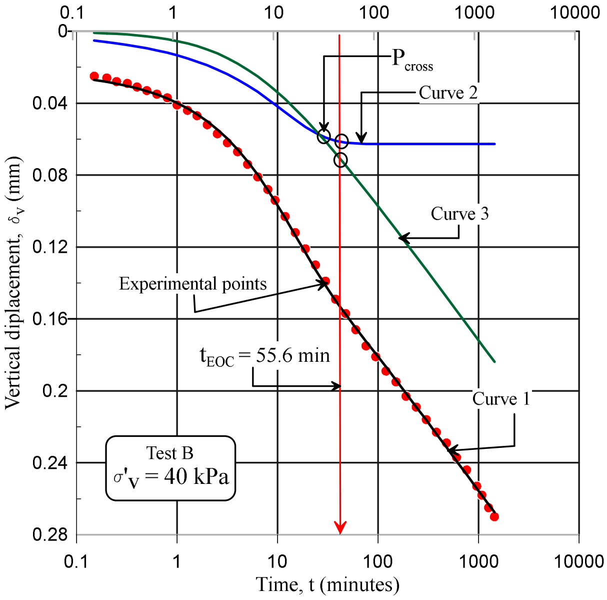

Figure 4. Time-vertical displacement curves from Test B on Mexico City soil under incremental loads: $\sigma_v' = 40\ kPa$ (below yield stress); LIR = variable; LID = 24 h

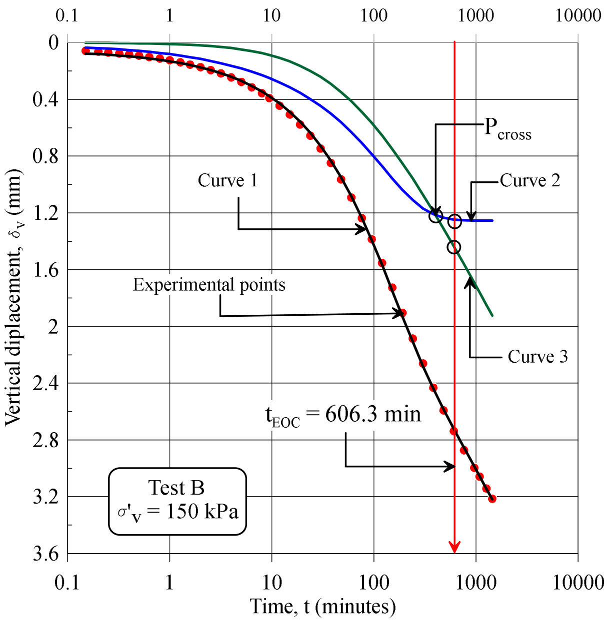

Figure 5. Time-vertical displacement curves from Test B on Mexico City soil under incremental loads: $\sigma_v' = 150\ kPa$ (above yield stress); LIR = variable; LID = 24 h

The behavior below yield stress ($\sigma_v' < \sigma_Y'$) may be characterized by:

- Volume changes due to consolidation are slightly larger than due to creep.
- The volumetric creep occurs during the consolidation and reaches its maximum before consolidation is completed.
- After the end of the consolidation process (t_{EOC}), the volume change is only due to volumetric creep.
- The crossover time (t_{cross}) occurs before EOC.

The above suggests that in the limit (i.e., tiny increments), the consolidation contribution will be minimal, and creep deformation will be predominant. The behavior above yield stress ($\sigma_v' > \sigma_Y'$) is similar to the behavior for $\sigma_v' < \sigma_Y'$ but is approximately 25 times greater.

The summarized response patterns observed in soil specimen are shown in Table 2 and Figure 6. The results in Figure 6 clearly show the effect of load increment size (i.e., LIR <1).

5.3. Comparison

To compare and observe more clearly the differences between the results of tests A and B, Figure 7 depicts both. Figure 7 shows that the immediate strains for small increments (Test B) are lower than those for large ones (Test A; LIR = 1). The EOC strain curves clearly show a difference between Tests A and B results. The size of load increment in incremental loading tests affects the 24 h strain curves.

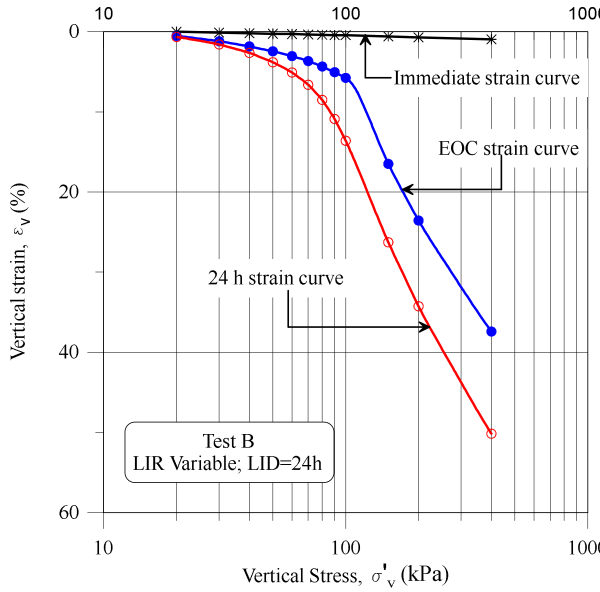

Figure 6. Strain-vertical effective stress relationships from Test B on Mexico City soil under incremental loads: small increments; LID = 24 h.

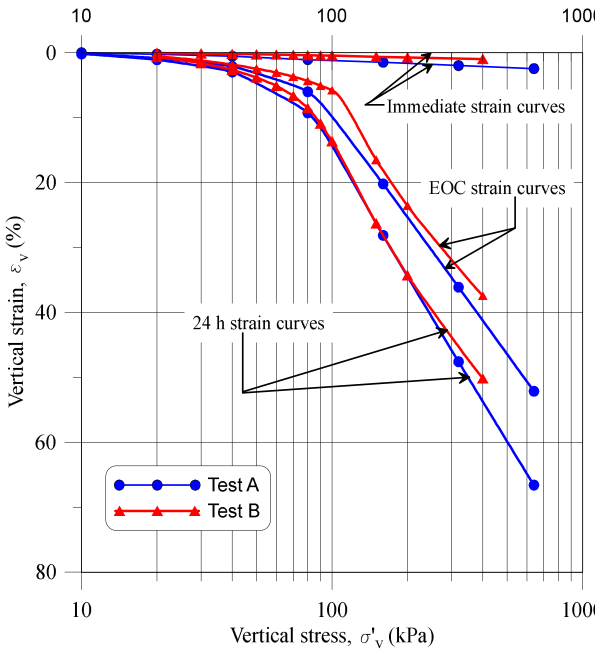

Figure 7. Strain-vertical effective stress relationships from Test A and B on Mexico City soil under incremental loads

6. Conclusions

For LIR = 1, the soil volume changes attributed to creep are secondary compared with those that develop due to the gradual dissipation of excess pore pressure (consolidation). However, for $LIR < 1$, some soils like SMC exhibit significant creep, and it is interesting to predict the amount and rate at which they will develop.

It is of greater significance the fact that a knowledge of the nature of the creep phenomenon would lead directly to a better understanding of the shear resistance, permeability, and other physical characteristics of fine-grained soils.

The author realized that the application of the described procedure is limited to the one-dimensional compression test. The observation applies strictly to the soil and conditions of the test employed in this study. Based on the data presented in this paper, the following conclusions may be drawn:

1. Observation of the compression process, both in the field and in the laboratory, demonstrated that volume changes continued to occur after excess hydrostatic pressure had essentially dissipated.
2. A systematic method of determining the end-of-consolidation (EOC) under a load increment was presented herein, which allows us to know the contribution of consolidation and volumetric creep at any time.
3. For conventional consolidation testing *(LIR = 1 and LID = 24h)*, the results are similar using the current methodology and the method presented herein. The creep phenomena occur during the consolidation phase but reach their maximum only after the consolidation is completed
4. For $LIR < 1$ (small increments) and $LID = 24\ h$, the results clearly show that the consolidation is carried out more slowly for small increments than large ones. The above suggests that in the limit (i.e., tiny increments), the consolidation contribution will be minor, and creep deformation will be predominant.
5. Although further refinement to the suggested procedure may arise from a rigorous examination, the author recognizes that a comprehensive investigation is needed. However, he believes that the right way to assess the interpretation of a one-dimensional compression test is by decomposing the time compression curve into its components: consolidation and creep.

Then, future developments related to the interpretation of one-dimensional compression tests should therefore be focused on enhancing numerical treatment of the soil compression data.

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