EXPERIMENTAL RESEARCH ON VARIATION OF PORE WATER PRESSURE IN CONSTANT RATE OF STRAIN CONSOLIDATION TEST

HOJJAT AHMADI, HASSAN RAHIMI, ABBAS SOROUSH and CLAES ALÉN

about the authors

Hojjat Ahmadi
Urmia University,
Water Engineering Department
Urmia, Iran
E-mail: hojjat.a@gmail.com

Hassan Rahimi
University of Tehran,
Irrigstion Department, Soil & Water Engineering College
Tehran, Iran

Abbas Soroush
Amirkabir University of Technology,
Civil and Environmental Engineering Department
Tehran, Iran

ClaesAlén
Chalmers University of Technology,
Civil Engineering Department
Gothenburg, Sweden

abstract

Constant rate of strain (CRS) consolidation is a rapid test method which is used for determination of compressibility of clayey soils. In a CRS test, the appropriate strain rate is selected based on pore water pressure ratio, i.e. the ratio of pore pressure to total stress. In the present study, to investigate the effect of strain rate on variation of pore water pressure ratio, four different clay samples of different plasticity were tested by CRS apparatus. The results of the experiments showed that the trend of variation of pore water pressure is dependent on the drained water flow regime which may be either Darcy or non-Darcy. The results also indicated that the plasticity of clay does not have considerable effect on variation of pore water pressure.

keywords

constant rate of strain, consolidation, pore water pressure ratio, soil plasticity, Darcy and non-Darcy flow

1 INTRODUCTION

In the standard CRS consolidation test a soil sample with thickness ranging from 10 to 30 mm (normally 20 mm), is contained within a consolidation cell, and compressed while subjected to a constant rate of displacement using a standard triaxial loading apparatus. During the test, specimen is drained from one side and developed excess pore pressure measured at the other side. Selection of appropriate displacement rate strongly depends on the development of pore pressure in the specimen. The theory of constant rate of strain consolidation or CRS was developed for the first time by Hamilton and Crawford [1] to overcome the constrains of ordinary consolidation test. They conducted several experiments using by strain rates of 0.015%/min to 0.005% /min and despite of such slow rates, were able to record noticeable pore water pressure development during the test. Theory and practice of CRS consolidation test were further developed, using simplifying assumptions similar to Terzaghi's ordinary consolidation test [2,3]. The most important assumptions were constant value for the ratio of permeability to compressibility and validity of Darcy's Law for water flow through the soil specimens [4]. The seepage flow velocity in porous media according to Non-Darcy flow is given by:

$$V = ki^n \quad (1)$$

where k is hydraulic conductivity, i is hydraulic gradient and n is a constant value which depends on the soil characteristics. When n is equal to 1, the flow regime is considered as Darcian.

Lee et al.(1993) employed theories of moving boundary and large strains and proposed a new method for selection of appropriate rate of strain and for determination of coefficient of consolidation under constant rate of strain [5]. Almeida et al (1996) conducted some laboratory tetsts on Rio De Janeiro Clay and showed that the method proposed based on moving boundary and large strains is not suitable for selection of the strain rate in CRS test [6].

During consolidation process, rate of dissipation of pore water pressure is the most important factor, which itself is a function of flow regime through the soil [7]. Several researchers have proposed the ratio of pore water pressure at the bottom of sample to total pressure (u/σ, known as pore water pressure ratio) as a criterion for selection of appropriate strain rate. The first criterion was first proposed by smith & Wahls in 1969 when they suggested a 50% as the maximum acceptable value for pore water pressure ratio [2]. Based on the results of experiments conducted by Wissa et al. [3], the maximum acceptable value for pore pressure ratio is 5%. Sallfor [8] recommended a value between 10% to 15% as acceptable pore water pressure ratio, while Gromen et al. [9] suggested values in the range of 30% to 50% . Sheahan and Watters [10] were able to achieve results similar to the ordinary consolidation test using a high pore water pressure ratio of 70%. Experiments conducted by Sheahan and Watters [10] showed a parabolic distribution of the pore water pressure in a CRS test.

Regarding the trend of variation of pore pressure ratio versus total stress, two patterns have been observed. One with a decreasing trend of pore pressure ratio with increasing total stress while in the other it initially decreases to some point, then increases to a peak and decreases again [11,12,13,14]. The wide range of values of strain rate of CRS test recommended by different researchers indicates a serious disagreement among the presented criteria for selection of a suitable strain rate in CRS test. This parameter is highly dependent on compressibility and hydraulic properties (permeability) of the soil specimen which both are considered as two major factors in consolidation process. The pore water flow during CRS consolidation test highly depends on the applied strain rate and may cause change in flow regime from Darcy to Non-Darcy besides its impacts on permeability and drainage rate of the soil [14].

The main objective of the present work is to investigate the effect of strain rates on pore water pressure in CRS consolidation test of clayey soils having different plasticity. Two goals are being investigated, firstly to check the validity of assumed flow regime (Darcy) in the relations referring to CRS consolidation and secondly to find a scientific justification and/or interpretation for the existing differences among the various available criteria.

2 MATERIALS AND METHOD

In the present study, several consolidation tests were conducted using two CRS apparatuses. To consider the effect of plasticity on consolidation behavior in CRS tests, four different clay samples of low to high plasticity from Karaj and Moghan provinces of Iran and Goteborg area in Sweden were employed.

The clay samples from Iran were prepared using slurry method. In this method, natural air dried soil samples from Moghan and Karaj were sieved through an appropriate sieve size. Then the soil powder was mixed with 200% water content in a mould of 400 mm internal diameter and 800 mm height. The prepared slurry was stirred for 24 hours and let to settle for 48 hours. After settling of the slurry, some excess water at the top of the slurry was drained and it was thoroughly mixed again for 30 minutes. The prepared slurry was used to fill a cylindrical chamber with inner diameter and height of 650 mm and 300 mm, respectively. A filter paper and geotextile sheet were placed at the bottom of the chamber to facilitate drainage of water. The Slurry was left to self-weight consolidation for 2 weeks, during which it was able to be drained from both top and bottom faces. Water level at the top of the chamber was regulated by a drainage tap. To increase the rate of consolidation, a 20 kg stainless steel rigid plate was placed on the top of the slurry and left for one week and the settlement was recorded to confirm completion of consolidation. At the end of preparation period, thickness of the sample was reduced to 72 mm. Figure 1 shows steps of preparation of the initial sample. Finally the test specimens were retrieved by silicon oil coated cutter ring which was fitted into the retaining ring of CRS apparatus. Then, the specimens were trimmed at the top and bottom faces. The consolidation chamber was submerged at all times. Before starting the tests, saturation of the specimens were checked by Skempton's B value. The average value of B parameter for all specimens were about 0.97, hence, they were considered as being saturated.

The samples from Sweden were taken by thin wall Shelby tube under groundwater table and thus were assumed to be undisturbed and fully saturated. To prevent any change in the moisture content of the samples, they were kept in closed tube container in a moist room at a temperature of 7 degrees Celsius. Table 1 shows the physical properties of the clay samples under investigation. Figures 2 and 3 depict the schematic diagrams of the two CRS consolidation apparatuses employed for tests in Iran and Sweden, respectively. The testing methods employed in the two apparatuses were nearly the same. In both apparatuses, the soil sample is allowed to drain at the top end and the valves at the bottom end were closed during the test. Since back pressure was not required, the cell was not filled with any fluid. The pore water pressure at the bottom end was measured using an accurate, calibrated diaphragm type pressure transducer with an accuracy of 0.02% F.S. (The most accurate device available in the Iranian market).

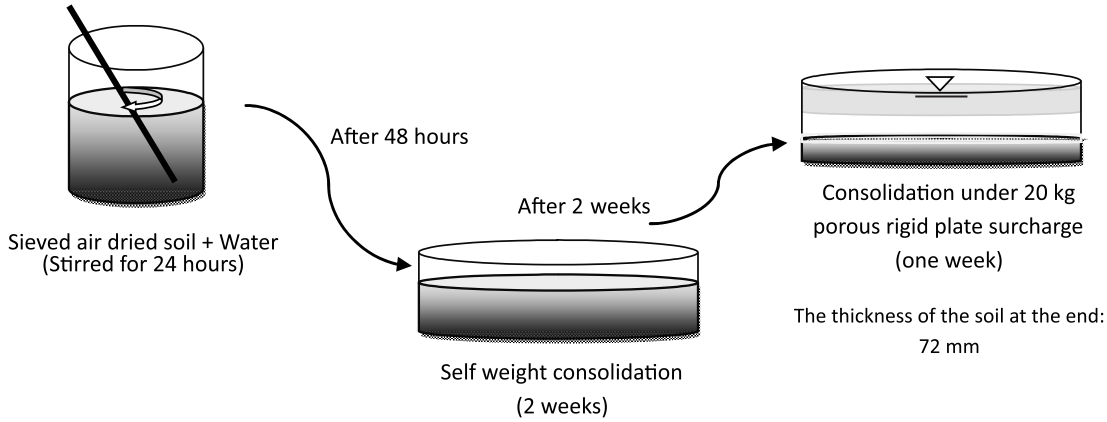

Figure 1. Steps of preparing reconstituted sample using slurry method.

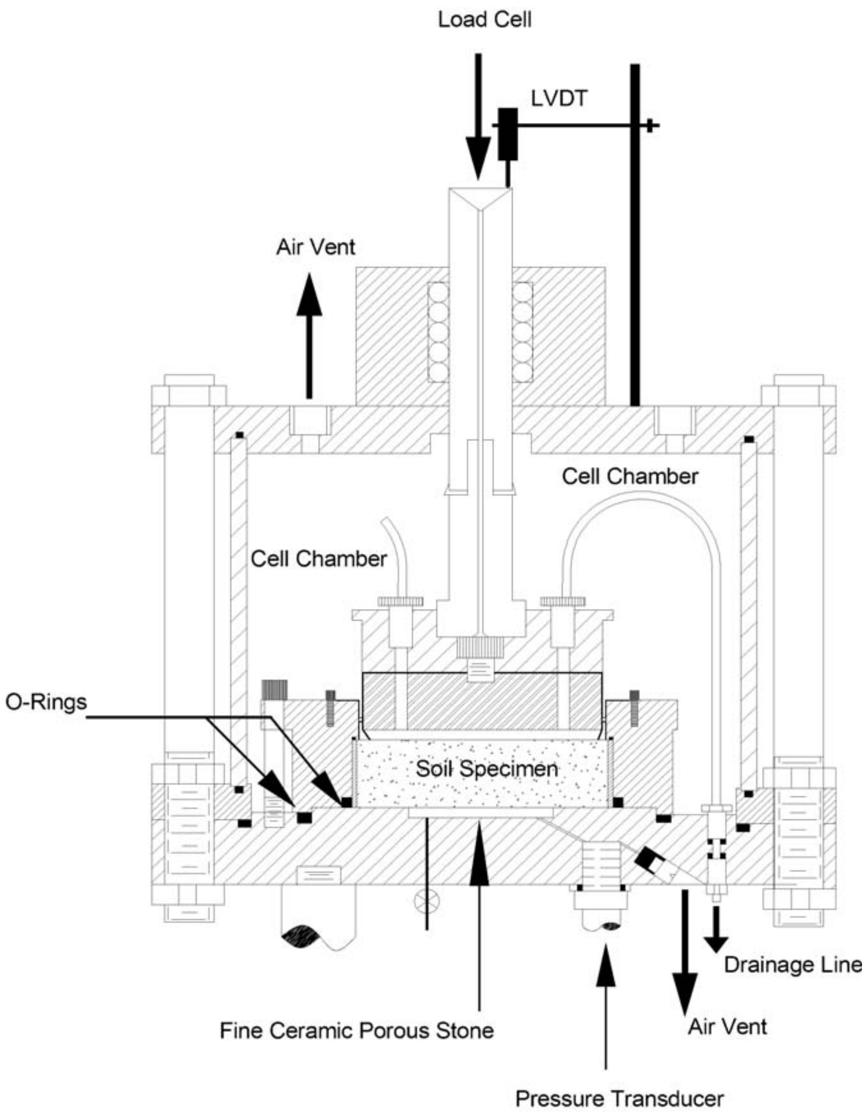

Figure 2. Schematic diagram of CRS apparatus employed in Iran.

Table 1. Physical properties of samples.

Sample	Sampling location	Gs	Sand (%)	Silt (%)	Clay (%)	LL (%)	PL (%)	PI (%)	USCS Class	Diameter (mm)	Height (mm)
A	Moghan	2.67	6	54	30	35.5	16.5	19.0	CL	100	20
B	Karaj	2.71	4	59	37	41.5	19	22.5	CL	100	20
C	Goteborg	2.64	0	7	93	75	15	60.0	CH	50	20
D	Goteborg	2.68	0	5	95	82	16	66.0	CH	50	20

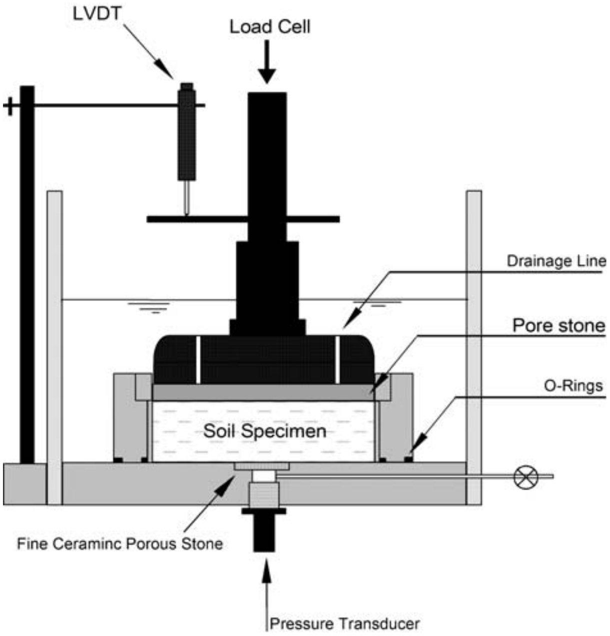

Figure 3. Schematic diagram of CRS apparatus employed in Sweden.

Table 2. Specifications of the samples and strain rates of different CRS tests.

Sample	Strain Rate %/min	Initial Water Content (%)	e_0	Maximum Pore Pressure (kPa)
A1	0.025	74.20	1.78	7.24
A2	0.050	70.02	1.80	94.00
A3	0.250	68.10	1.76	245.00
A4	0.375	68.10	1.77	446.00
B1	0.025	67.40	1.83	9.40
B2	0.050	66.20	1.81	132.60
B3	0.250	70.10	1.87	253.00
B4	0.375	68.40	1.80	432.00
C1	0.001	89.00	2.40	2.00
C2	0.006	88.00	2.40	14.00
C3	0.012	89.00	2.40	22.00
C4	0.024	84.00	2.50	50.60
D1	0.001	33.00	0.9	1.70
D2	0.006	35.50	0.95	9.60
D3	0.012	33.00	0.93	22.00
D4	0.024	33.00	0.92	35.50

Vertical deformations and applied stresses were measured using LVDT and load cell, respectively and recorded via a data logger. Constant strain rate was applied to the soil samples through a precision pressure controller. The tests were carried out in accordance with ASTM D4186-2000 [4]. Due to full saturation of the samples backpressure was not applied to the specimens [10,5].

To investigate the effect of different testing speeds, the samples from Iran (having higher permeability) were tested at higher speeds and samples from Sweden (having lower permeability), were tested at lower speeds. For this purpose, four strain rates of 0.001%/min, 0.006%/min, 0.012%/min and 0.024%/min were applied to samples from Sweden and strain rates of 0.025%/min, 0.050%/min, 0.250%/min and 0.375%/min were applied to samples from Iran. Table 2 shows the applied strain rates for different samples.

3 RESULTS

3.1 VARIATION OF PORE WATER PRESSURE

Variation of the pore water pressure at the bottom end of the samples with time indicates that the pore pressure increases gradually and reaches to a maximum value at the end of the test. Figures 4 to 7 present the results of the tests conducted on different samples. As these figures show, magnitudes of the pore water pressure at the bottom of the samples depend on the strain rate. As the strain rate increases, the pore pressure increases accordingly. This is due to lesser available time for dissipation of the developed pore water pressure. While, for the same samples tested under lower strain rates, there is enough time for dissipation of pressure, and thus, lower magnitudes of pore water pressure were recorded. The maximum pore water pressure developed in each test is given in Table 2.

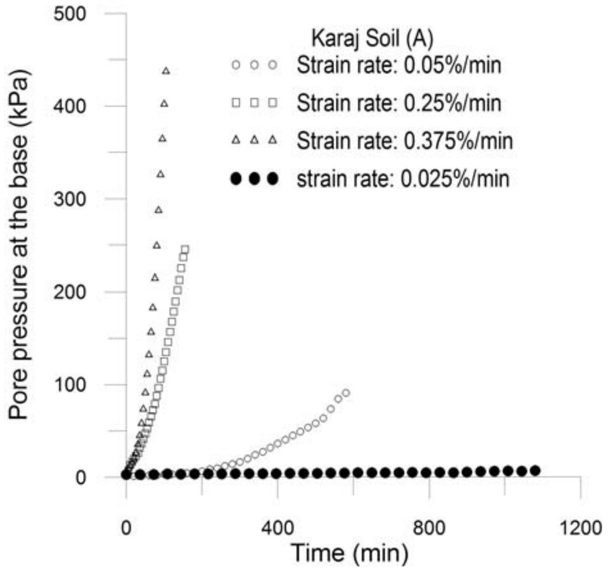

Figure 4. Variation of pore pressure with time for sample A (PI=22.5%).

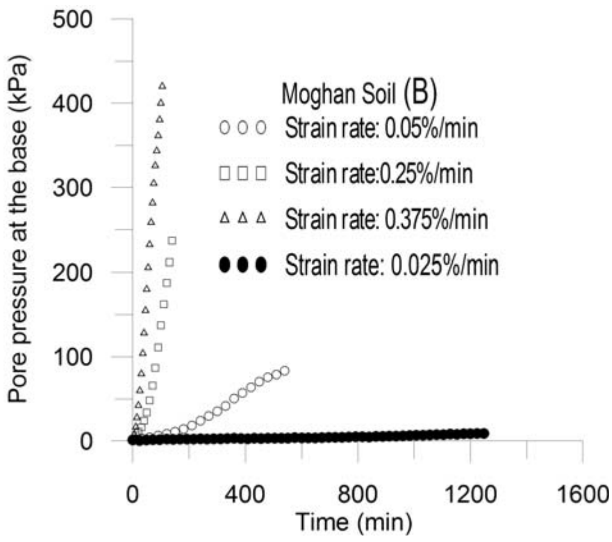

Figure 5. Variation of pore pressure with time for sample B (PI=19%).

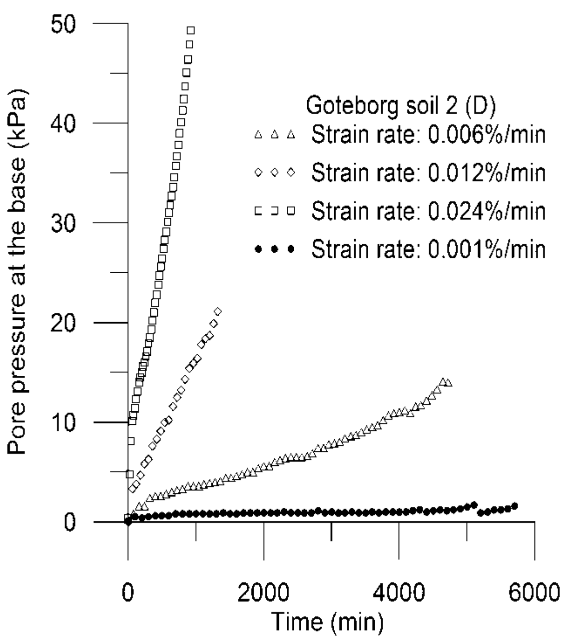

Figure 6. Variation of pore pressure with time for sample C (PI=60%).

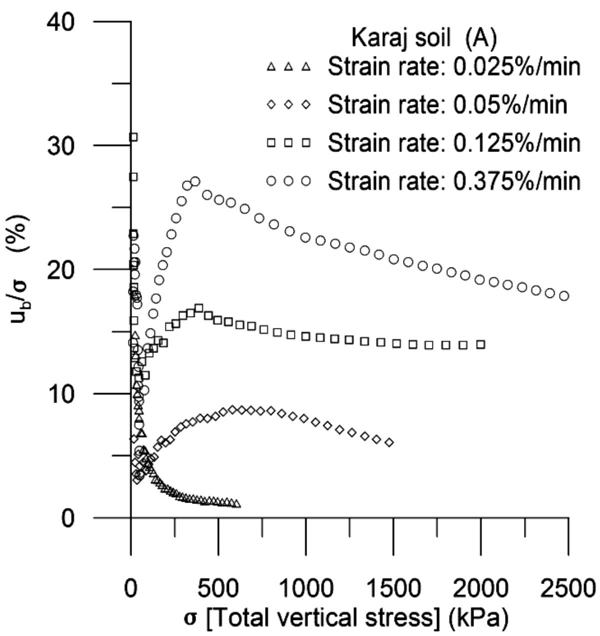

Figure 7. Variation of pore pressure with time for sample D (PI=66%).

3.2 RELATIONSHIP BETWEEN TOTAL STRESS AND PORE WATER PRESSURE

The main reason for development and increasing of pore pressure in a CRS consolidation test is the applied total vertical stress which gradually increases during the course of the test (due to very slow process of pore pressure dissipation). To study the trend and pattern of variation of pore pressure in different samples, the ratio of developed pore water pressure at the bottom of sample to the total stress (u/σ) versus total stress (σ) has been depicted in Figures 8 to 11. Figures 8 and 9 depict such variations for samples A and B which are less plastic and have been tested at higher strain rates. As these figures show, the curves at the very beginning indicate a sharp drop. The reason for such a drop is the fact that at the very beginning there is some residual pore water pressure due to the weight of the sample itself, while the total stress is still zero and thus, their ratio is a large value. After starting of the test, the total stress increases

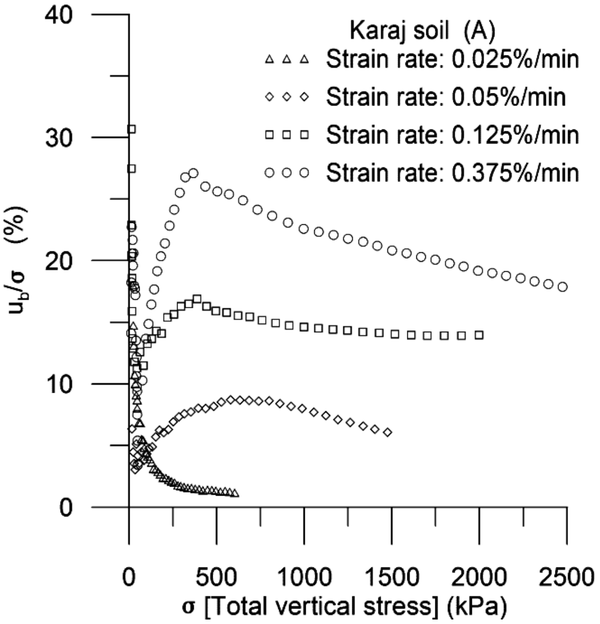

Figure 8. Variation of relative pore pressure versus total stress for sample A.

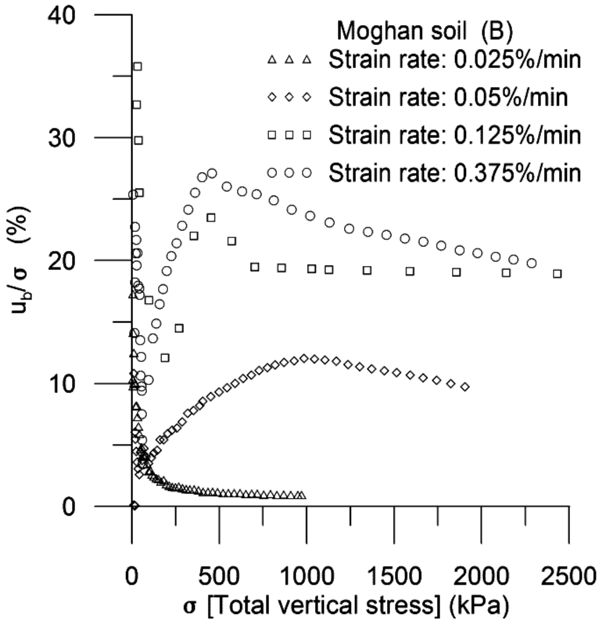

Figure 9. Variation of relative pore pressure versus total stress for sample B.

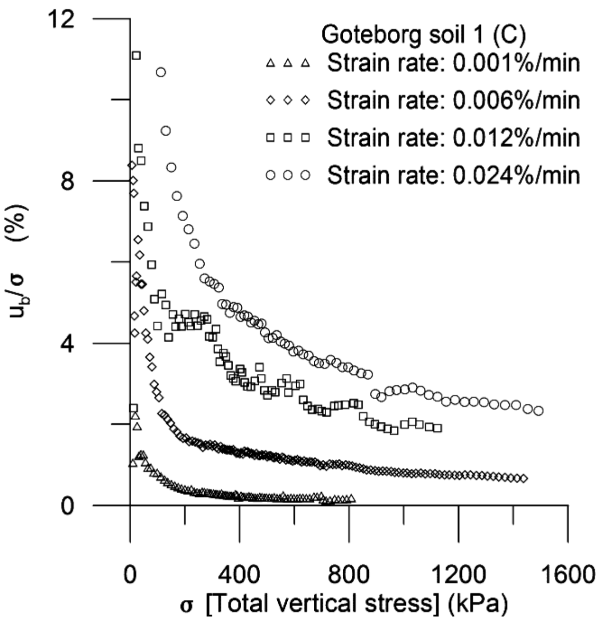

Figure 10. Variation of relative pore pressure versus total stress for sample C.

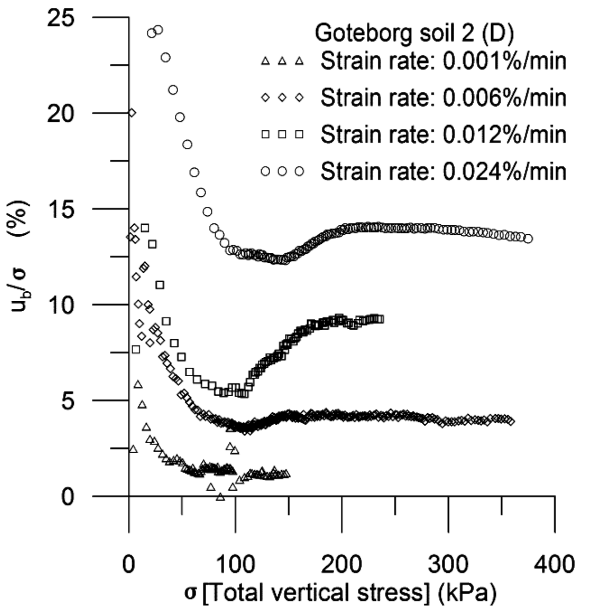

Figure 11. Variation of relative pore pressure versus total stress for sample D.

and the pore pressure ratio decreases sharply for a short period of time. For samples A and B, which were tested at higher strain rates, there is not enough time for the built-up pore water pressure to dissipate and thus the curves indicate an increasing trend up to a peak point, beyond which the pore pressure ratio drops gradually due to increase in total stress.

Figures 10 and 11 depict the test results for samples C and D which were of higher plasticity, and tested at lower strain rates. According to these figures, the pore water pressure ratio decreases by increasing total stress. The trend of the curves for all applied strain rates show a steep slope at the beginning, which becomes less towards the end of the test, showing no maximum or minimum value. The reason for such a continuously decreasing trend is lower strain rates which provide the opportunity for the samples to be drained. However, there are some differences between the results of samples D and C, due to their different water content and void ratio. Sample D is more plastic and has less void ratio in comparison with sample C and thus, dissipation of pore pressure is slower for the former. This fact causes an increasing trend in pore water pressure ratio at some stages of the tests.

3.3 Flow regime in CRS consolidation test

In a CRS consolidation test, by increasing the applied total stress, excess pore water pressure is developed and dissipated gradually as the sample is drained. If similar to a trixial CD test, the load is applied slowly; pore pressure has the opportunity to dissipate in a short period of time and thus, the ratio of pore pressure to total stress would decrease. However, in a CRS test, since increasing rate of the total pressure is not constant and in the mean time the permeability of the soil sample decreases due to its compression, thus, dissipation of the pore pressure is much slower [7]. This would cause an increase in the ratio of pore pressure to total stress. After some times, the rate of increase in this ratio becomes slower which is indicative of an ease in draining of the sample and it means the hydraulic conditions through the specimen changes. Since the resistance against flow is always higher in a non-Darcy flow than a Darcy flow [15], thus, easier dissipation of pore pressure, in spite of lower void ratio and lower permeability, is due to the change in flow regime. This means that during the course of a CRS consolidation test, the flow regime changes from non-Darcy to Darcy. In fact, for those samples, where the trend of variation of pore pressure ratio versus total stress at the beginning is ascending and then descending, the flow regime changes from non-Darcy to Darcy. The peak point on the curve of pore pressure ratio versus total stress shows the moment at which the flow regime changes from Non-Darcy to Darcy.

For better understanding of the flow regime during the tests, variation of hydraulic gradient against velocity is normally plotted. Since in a CRS consolidation test the velocity of outflow is equal to the strain rate, thus, it should always be constant. Due to compression of the sample, its void ratio and consequently permeability is decreasing and thus, it is not possible to plot the variation of hydraulic gradient against velocity for a given permeability, unless to stop the test and measure coefficient of permeability at a given time. In such a case, variation of hydraulic gradient against velocity will be indicated by a horizontal line (dashed line on Figure 12). If the hydraulic gradient and velocity are plotted on horizontal and vertical axes respectively, then every point on the plotted line is indicative of a given permeability. As shown in Figure 12, for any given value of coefficient of permeability, a single curve can be plotted which crosses the dashed horizontal line at the given point (given hydraulic gradient and velocity). As the time passes and consolidation progresses, the curve of hydraulic gradient versus velocity moves towards left. This is similar to the findings of Hansbo [16,17,18] who plotted hydraulic gradient versus velocity in an ordinary consolidation test, by measuring coefficient of permeability of the sample at each loading stage.

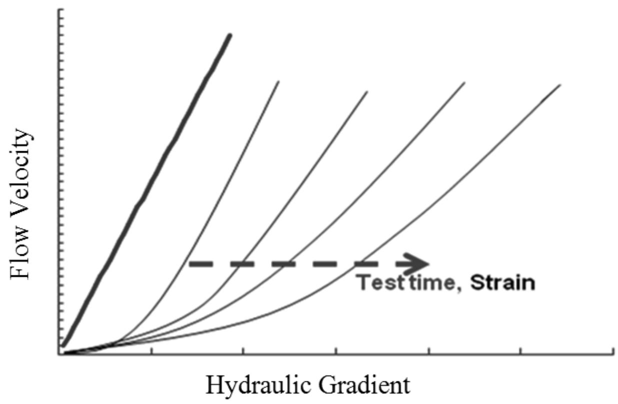

Figure 12. Variation of hydraulic gradient versus velocity in a CRS test.

In a CRS consolidation test, the thickness of the sample and pore pressure at both ends at any time are known and thus, the mean hydraulic gradient at any time can be determined by dividing the difference between pore pressures at the top and bottom faces of the sample to its height. The exit velocity of water in CRS test is equal to speed of the compressing piston or compression [13]. Therefore, considering the four applied strain rates, for each given void ratio one point is found. Thus, in total four points related to four strain rates are found which are used to plot the trend curve.

In the present study, variation of hydraulic gradient (i) versus flow velocity (v) has been plotted by conducting the CRS consolidation tests at different strain rates for samples C and D. The linearity or non-linearity of the plotted curves is indicative of flow regime. A linear relationship is indicative of Darcy flow and non-linear depicts a non-Darcy flow. Figures 13 and 14 show the curves obtained in CRS tests conducted on soil samples from Sweden. As these figures show, the relationship between i and v is non-linear, which is indicative of a non-Darcy flow. The same behavior was observed for two other soil samples as well.

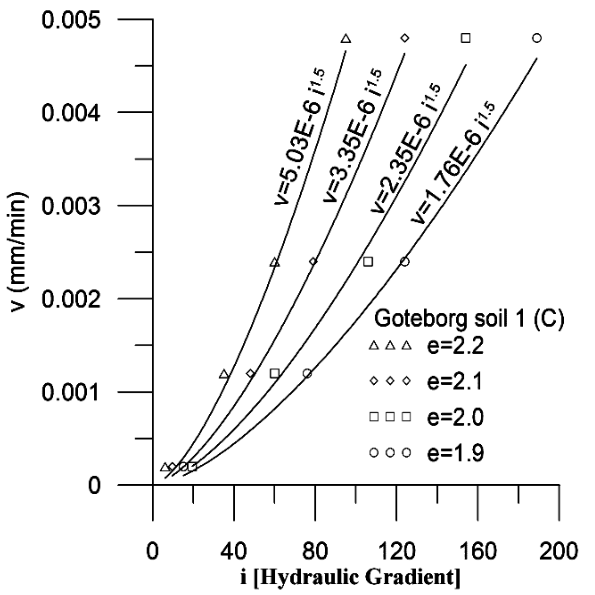

Figure 13. Variations of hydraulic gradient versus flow velocity for sample C.

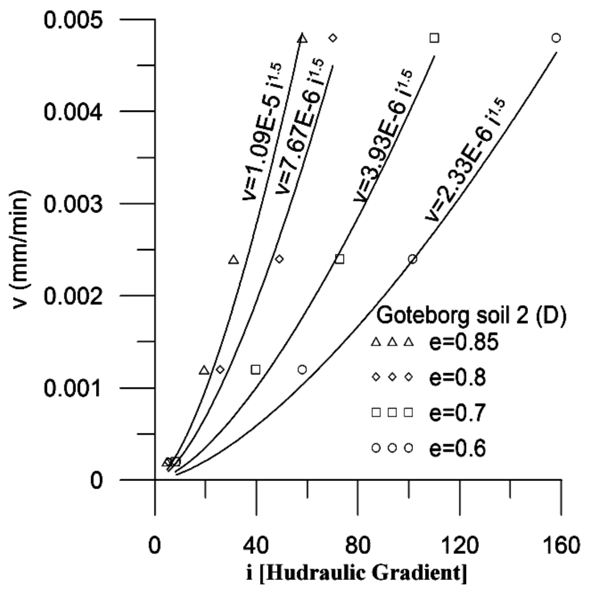

Figure 14. Variations of hydraulic gradient versus flow velocity for sample D.

These curves are related to the middle time period of the tests. An exponential curve with exponent of 1.5 has shown a good correlation with obtained data, though other types of curves may also be satisfactory. However, in general, the trend s of the curves show that relationship is not linear and the flow is considered as non-Darcy.

In a CRS consolidation test, the flow regime may be Darcy, non-Darcy or both, depending on the strain rate which is equal to the velocity of the drained water flow. The non-Darcy flow during a consolidation test occurs due to continuous blockage of flow channels or changes in the direction of flow [19]. All relationships obtained for CRS consolidation in the past, were developed by assuming a Darcy flow for the whole duration of the test. While, the results of the present study show that the flow regime in a CRS test could be non-Darcy, and thus, it is necessary to develop the relevant relationships. In other words, to obtain more accurate results in a CRS test, the consolidation parameters should be determined using the appropriate relationships relevant to the expected flow regime. To develop such a proper consolidation equation, the non-Darcy flow regime (Eq. 1, $n \neq 1$) should be considered in Darcy's Equation to develop the required realtionships. The development of such equation is out of the scope of this paper.

3.4 STRESS-STRAIN BEHAVIOR

In a CRS consolidation test the normal stress in compression curve is often presented by effective stress. Based on parabolic distribution of pore water pressure in sample during CRS consolidation, the mean effective stress is determined as follows [3]:

$$\bar{\sigma}' = \sigma - \frac{2}{3} u_b \quad (2)$$

During consolidation, the pore water pressure distribution in the sample is also parabolic, when non-Darcy flow rule is applied [20]. Therefore, to determine the effective stress, equation 2 is employed. The compression curves for tested samples are presented in Figures 15 to 18.

Figures 15 and 16 show the compression behavior of the two samples having lower plasticity. These curves resemble similarity to the behavior of remolded samples, though the samples were reconstituted. This behavior is due to high water content and high softness of the samples. These figures also show a shift to the right of compression curves for the three rapid tests in comparison with slower tests. This is more evident in Moghan sample. The compression curves for all rapid tests show almost the same trend (Figures 17 and 18). It means that during primary consolidation, creep has not occurred, which is in agreement with findings of Leroueil [21].

In samples with high plasticity, similar behavior has been observed and the compression curves for rapid tests were shifted to the right in comparison with the slow tests. This is more evident in sample D. Thus it can be concluded that in these samples also creep was negligible. The virgin curves of these samples have steep slope which is indicative of their higher sensitivity [22]. However, the compression behavior of all samples is similar and effect of strain rate on pore water pressure is considerable.

As it was shown in the previous section, the trend of curves of pore pressure ratio versus total stress in sample C differs from other samples, while there was no differ-

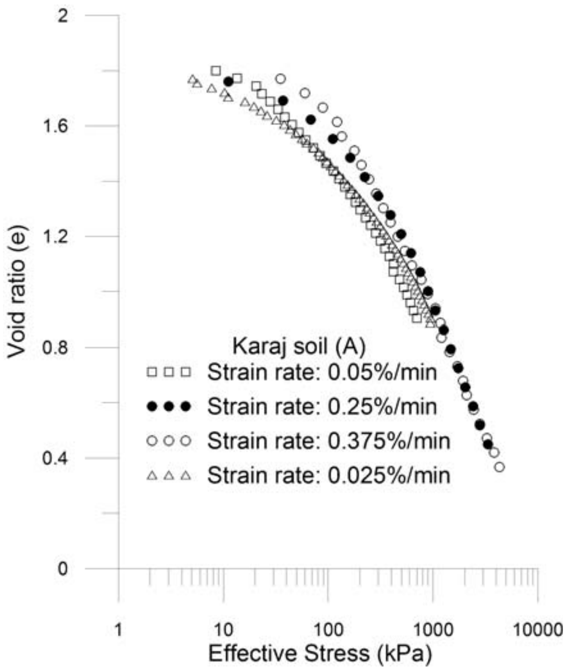

Figure 15. Compression curve for different rate of strains, Sample A (PI=22.5).

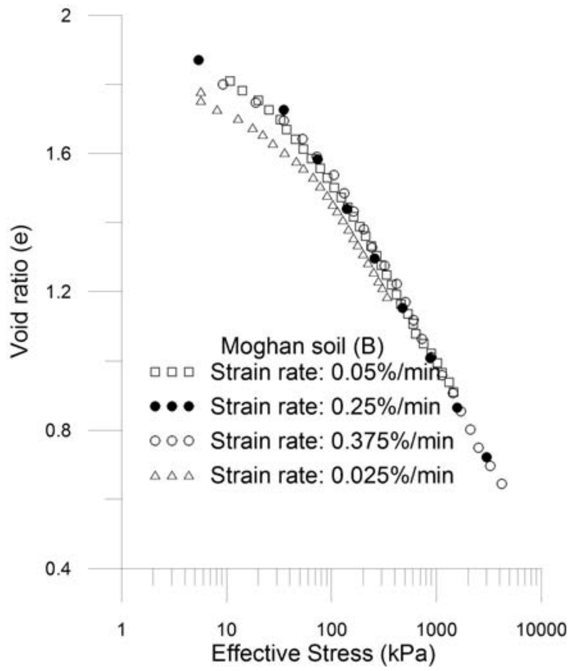

Figure 16. Compression curve for different rate of strains, Sample B (PI=19).

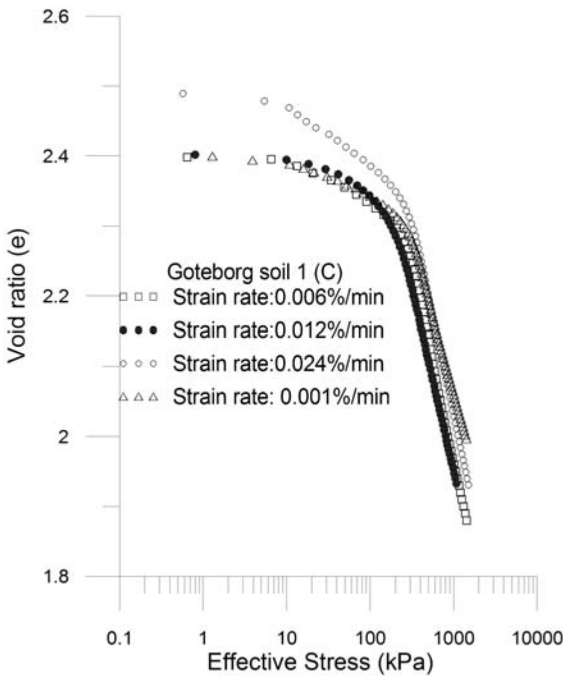

Figure 17. Compression curve for different rate of strains, Sample C (PI=60).

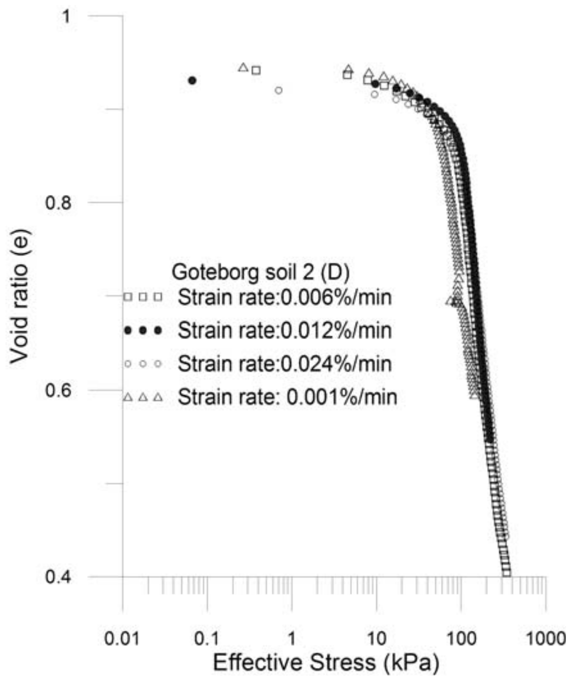

Figure 18. Compression curve for different rate of strains, Sample D (PI=66).

ence among the compression curves. It is believed that the different impact of strain rate on variation of relative pore pressure versus total stress is not related to the stiffness of the samples, rather it is due to flow regime during consolidation process.

4 CONCLUSIONS

Based on the results of the present study, the following conclusions are made:

- The pore pressure induced at the bottom of the sample in a CRS consolidation test is variable and depends on the strain rate. The higher soil plasticity, the higher is the induced pore pressure.
- For both low and high plastic samples, the relation between pore water pressure and time for a given strain rate is almost linear; however, at the beginning of the tests some nonlinearity was observed.
- Variation of pore pressure ratio (u_b/σ) against total stress (σ) in CRS test is related to the strain rate. In smaller strain rates, the pore pressure has the opportunity to be dissipated easily, and thus no excess pore pressure is developed. While for higher strain rates, the induced pore water pressure does not have enough time to be dissipated and thus, excess pressure is developed in early stages of the test, but will gradually decrease during final stages due to change in the flow regime.
- Variation of pore pressure ratio is related to the flow regime. The increasing trend is indicative of non--Darcy flow, while the decreasing trend is indicative of Darcy flow. The point of maximum on the curve of pore pressure ratio versus total stress shows the moment at which the flow regime changes from Non-Darcy to Darcy.
- Plotting hydraulic gradient against flow velocity in the CRS tests conducted in the present study, has very well demonstrated that the flow regime is mostly non-Darcy. This is an important fact which should be considered in analysis of the results of CRS consolidation tests.
- In both low and high plasticity samples, the pore pressure behavior was almost the same. However, due to lower permeability of the high plasticity samples, the impact of strain rate is comparatively more evident and considerable variation were observed in the drainage behavior at lower strain rates.
- As shown, validation of Darcian flow through specimens under CRS consolidation test is in question and developing new equations based on more realistic Non-Darcian flow is essential.

REFERENCES

[1] Hamilton, J.J., Crawford, C.B. (1959). Improved determination of preconsolidation pressure of a sensitive clay. Papers on Soils, ASTM Spec. Tech. Pub. 1, 254-270.

[2] Smith, R. E., Wahls, H. E. (1969). Consolidation under constant rate of strain., Journal of Soil Mech. Found. Div. 95 (SM 2).

[3] Wissa, A. E. Z., Christian, J. T., Davis, E. H., Heiberg, S. (1971). Consolidation testing at constant rate of strain. Journal of Soil Mechanic and Foundation Div., ASCE 97 (10), 1393-1413.

[4] ASTM, (2002). Annual Book of ASTM Standards. American Society for Testing and Materials, Soil and Rock, D4186-89, 500-505.

[5] Lee, K., Choa, V., Lee, S. H., Quek, S. H. (1993). Constant rate of strain consolidation of Singapore Marine Clay. Geotechnique 43 (3), 471-488.

[6] Almeida, M. S. S., Martins, I. S., Carvalho S. R. L. (1995). Constant rate of strain consolidation of Singapore Marine Clay, Discussion to paper (Lee, K. et. al.). Geotechnique 45 (2), 333-336.

[7] Dobak, P. (2003). Loading velocity in consolidation analysis. Geological Quarterly 47 (1), 13-20.

[8] Sallfor, G. (1975). Perconsolidation pressure of soft highly plastic clays. Chalmers Univ. Tech. Goteborg.

[9] Gorman, C. T., Hopkins, T. C., Deen, R. C., Drenvich, V. P. (1978). Constant rate of strain and controlled gradient consolidation testing. Geotech. Testing Journal1(1), 3-15.

[10] Sheahan, T. C., Watters, P. J. (1997). Experimental verification of CRS consolidation theory. Journal of Geotechnical and Geoenvironmental Eng. 123, 5, 430-437.

[11] Armour, D. W. Jr., Dernevich, V. P. (1986). Improved techniques for the constant rate of strain consolidation test, Consolidation of Soils: Testing and Evaluation, ASTM STP 892, R. N. Yong and F. C.

[12] Seah, H. T., Juirnarongrit, T. (2003). Constant rate of strain consolidation with radial drainage. Geotechnical Testing Journal 26,4, 1-12.

[13] Yune, Y. C., Chung, K. C. (2005). Consolidation test at constant rate of strain for radial drainage. Geotechnical Testing Journal 28, 1, 71-78.

[14] Ahmadi.H, Rahimi. H., Soroush. A. (2011) Investigation on the Characteristics of pore water flow during CRS consolidation test. Geotechnical and Geological Engineering 29, 6, 989-997.

[15] Mitchell, K. J, Younger, J. S. (1967). Abnormalities in hydraulic flow through fine-grained soils. In Idriss, I., M. (Ed) (2001). Selected geotechnical papers of James K. Mitchell, ASCE, 142-178.

[16] Hansbo, S. (1960). Consolidation of clay, with special reference to influence of vertical sand drains. A study made in connection with full scale investigations at Skä-Edeby. Doctoral thesis, Swedish Geotechnical Institute.
[17] Hansbo, S. (2001). Consolidation equation valid for both Darcian and non-Darcian flow. Geotechnique 51, 1, 51-54.
[18] Hansbo, S. (2003). Deviation from Darcy's Law observed in one-dimensional consolidation. Geotechnique 53, 3, 601-605.
[19] Mitchell, K. J., Soga, K. (2005). Fundamental of Soil Behaviors. Third Edition, John Wiley and Sons, New York, N.Y.
[20] Ing, C. T., Xiaoya, N. (2002). Coupled consolidation with non-Darcian flow. Computers and Geotechnics 29, 169-209.
[21] Leroueil, S. (1988). Recent developments in consolidation of natural clays. Tenth Canadian Geotechnical Colloquium. Canadian Geotechnical Journal 25, 85-107.
[22] Bowles, E. J. (1979). Physical and Geotechnical Properties of Soils. McGraw-Hill Book Company, New York, NY.