

Experimental observation of a soft clay consolidation behavior using constant-rate-of-strain (CRS) test

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ABSTRACT

This paper reports a laboratory characterisation of a soft clay behaviour with constant-rate-of-strain (CRS) test using a novel oedometer setup developed at Université de Sherbrooke. Different strain rates were applied to observe the evolution of stress-strain relationships, pore water pressure distribution and volume characteristics. The CRS results were then compared to the conventional oedometer test results in order to assess the effect of strain rate variation on pre-consolidation pressure. The results indicated that strain rate affects the determination of the pre-consolidation pressure and the compressibility index of the tested material. In addition, it was observed that the evaluation of CRS test results is limited to the generation of excess pore water pressure within the tested specimens and that a greater care must be taken in the analysis of cases with drainage paths.

RÉSUMÉ

Cet article présente la caractérisation du comportement d'une argile molle à taux de déformation constant (CRS) à l'aide d'un nouvel œdomètre mis au point à l'Université de Sherbrooke. Différents taux de déformation ont été appliqués pour observer l'évolution de la relation contrainte-déformation, la distribution de la pression d'eau interstitielle et les caractéristiques volumiques. Les résultats de CRS ont ensuite été comparés aux résultats des essais œdométriques conventionnels afin d'évaluer l'effet de la variation du taux de déformation sur la pression de pré-consolidation. Les résultats indiquent que le taux de déformation affecte la détermination de la pression de pré-consolidation et l'indice de compressibilité du matériau testé. De plus, on a observé que l'évaluation des résultats d'analyse du CRS se limite à la génération d'une pression d'eau interstitielle en excès dans les échantillons testés et que la présence de tout type de chemin de drainage peut compliquer le processus d'analyse des données.

1 INTRODUCTION

Constant-rate-of-strain (CRS) oedometer test is an alternative testing approach to the conventional incremental loading (IL) oedometer test to estimate the K_0 volumetric characteristics and hydraulic conductivity of fine-grained soils. This test was first suggested by Hamilton and Crawford (1959) to obtain the pre-consolidation pressure along with the deformability indices and to overcome the limitations of the conventional IL method such as its time-consumption and the scattered measurement of stress-strain data points. The testing procedure of CRS test is similar to IL test whereas during the test, the generated excess pore pressure is allowed to drain from the top of the soil specimen but not from the bottom where the pore water pressure is measured using a pressure transducer. The specimen is then deformed at a constant rate of deformation with time to obtain the stress-strain curve while the variation of the coefficient of consolidation and the hydraulic conductivity can be calculated through data analysis. The trade-off as suggested by Ozer et al. (2012) is that a fast rate is desired to perform the test in a rapid fashion, but the chosen rate should not be so high to induce enough excessive excess pore pressure in the specimen. The main disadvantage of CRS test is to find a suitable strain rate prior to the loading of the soil specimen that does not create the structural instability while it generates adequate excess pore water pressure to measure (Lee et al., 1993; Sheahan and Watters 1997; Fox et al., 2014). As studied by many researchers including Maleksaeedi et al. (2018), Ozer et al. (2012),

the suitable strain rate depends on the initial void ratio, the soil structure and the corresponding generated pore water pressure. Throughout the CRS test, the distribution of excess pore water pressure and the effective stress across the height of specimen are essentially unknown. Various approaches have been suggested to estimate the proper strain rate depending on the physical characteristics such as liquid limit or using u_b/σ_v ratio defined as the ratio of excess pore water pressure to the vertical total stress (Wissa et al. 1971; Leroueil et al. 1983, 1985). However, these solutions are valid only for the soils for which they were developed.

Another approach suggested by researchers such as Smith and Wahls (1969), Lee (1981) and Ozer et al. (2012) involves performing at least one trial or reference IL consolidation test to obtain the compressibility characterisations including compressibility indices in both overconsolidated and normally consolidated states. Afterwards, different spectra of stress-strain curves are developed and best fitted to the experimental results from IL test to find the proper strain rate. Although this approach is more costly, it could be a practical solution for projects that require several oedometer tests. This highlights the importance of the quality of reference test results on the selection of the proper strain rate. Despite significant applications of CRS test in laboratory studies, the literature lacks a thorough database in order to further develop and verify the consolidation theories dedicated to CRS test.

This paper reports the results of a series of CRS tests on Saint-Charles-Borromée clay performed at different strain state using a setup prepared at Université de Sherbrooke (Maleksaeedi et al., 2018). A comparison is made between the results of IL and CRS tests regarding the effect of strain rate and interstitial pressure (i.e. pore-water pressure) on the obtained stress-strain curves.

2 ANALYSIS METHOD IN STEADY-STATE CONDITION

Unlike IL tests where effective vertical stress is considered uniform throughout the specimen at the end of each consolidation step, in CRS tests, the unknown distribution of pore water pressure makes it difficult to determine the Terzaghi's effective vertical stress (Ozer et al. 2012). Various methods of analysis exist when it comes to obtaining the compressibility, hydraulic conductivity and coefficient of consolidation for CRS tests depending on the state of flow within the specimen.

The steady-state condition during a CRS test is often corresponded to the dimensionless time factor $T = c_v t / H_o^2 > 0.5$ according to the studies of Wissa et al. (1971). Regarding stress-strain relationship in k_o state, for the cases at which the coefficient of compressibility a_v , is constant, the linear solution proposed by ASTM D4186/D4186M is sufficient to obtain the volumetric characteristics as presented by Eq. 1 to Eq. 4. The steady-state condition is then controlled by introducing a parameter called, the steady-state factor, F shown in Eq.1 which should be always higher than 0.4.

$$F = 1 - \frac{u_{ex,b,n}}{\Delta q_n} \quad [1]$$

where $\Delta q_n = q_n - q_0$ demonstrates the changes in applied vertical effective stress at the top boundary of the specimen; q_n is the applied vertical effective stress; q_0 is the initial applied vertical effective stress; $u_{ex,b,n}$ is the excess pore pressure measured at the base of the specimen. Since the distribution of excess pore water pressure is essentially parabolic in case of constant a_v , the vertical effective stress, σ'_v , hydraulic conductivity, k_n , and coefficient of consolidation, c_v are calculated as following:

$$\sigma'_v = q_n - \frac{2}{3} u_{ex,b,n} \quad [2]$$

$$k_{h,n} = \frac{\gamma_w r H_o H_n}{2 u_{ex,n}} \quad [3]$$

$$c_{v,n} = \frac{H_o H_n (\Delta q_{n+1} - \Delta q_{n-1})}{2 u_{ex,n} (\tau_{n+1} - \tau_{n-1})} \quad [4]$$

In which, H_o and r initial height and strain rate respectively. H_n is the height of specimen at a given time, n . τ_n is the elapsed time during the test.

On the other hand, when the compression index C_c , is constant, Fox et al. (2014) found out that ASTM D4186/D4186M nonlinear solution induces analytical error for determination of volumetric and hydraulic properties during CRS test. Thus, it is suggested to use Eq. 5 to Eq. 7 to calculate the steady-state factor, average effective stress, hydraulic conductivity, k_h and the coefficient of consolidation c_v .

$$F = \frac{\log(\sigma_v - u_b) - \log(\sigma_{v0})}{\log(\sigma_v) - \log(\sigma_{v0})} \quad [5]$$

$$\sigma'_v = (\sigma_v^3 - 2\sigma_v^2 u_{ex,b} + \sigma_v u_{ex,b}^2)^{1/3} \quad [6]$$

$$k_{h,n} = \frac{-0.434 \cdot r \cdot \gamma_w \cdot H_o \cdot H_n}{2 \sigma'_{v,n} \cdot \log\left(1 - \frac{u_{ex,b,n}}{\sigma_{v,n}}\right)} \quad [7]$$

$$c_{v,n} = \frac{-H_o H_n \cdot \log\left(\frac{\sigma_{v,n+1}}{\sigma_{v,n}}\right)}{2(\tau_{n+1} - \tau_n) \log\left(1 - \frac{u_{ex,b,n}}{\sigma_{v,n}}\right)} \quad [8]$$

Based on formulations above, the applied strain rate is critical parameter. A CRS test conducted at high strain rate cannot be interpreted correctly since it generally suggests an artificially high pre-consolidation pressure (Nash et al. 1992). Yet, too small strain rate may lead to problems in determining the consolidation properties. Normally, the strain rate value is chosen according to the variation of u_b / σ_v . Based on ASTM D4186/D4186M, u_b / σ_v should be limited to a value between about 3 % and 15 % at the end of the loading phase. Yet, Henriche and Belkacemi (2018) suggested that this range is strongly dependent on the value of initial void ratio and the specimen compressibility. As a result, the proper range of strain rate is also a matter of discussion.

3 MATERIAL PROPERTIES AND TEST PROCEDURE OF CRS TEST

In this study, a series of CRS tests were performed on undisturbed soft clay specimens of Saint-Charles-Borromée. The two blocks of soft clay were extracted from the bed of an excavated construction site. In general, the clayey soil on the site contains between 60 to 70% of clay particles and 40 to 30% silt. Based on the Casagrande diagram, this soft clay can be classified as CH. As shown in Figure 1, the cavities and holes were randomly distributed within the samples with different sizes and lengths. The presence of these holes brings certain complexities to the overall behavior of clay since the generated excess pore water pressure can be dissipated. It was observed that at the surface of these blocks, there were more cavities and holes compared to the deeper sections. Table 1 shows the physical properties of extracted clay blocks.

In order to perform CRS tests, five specimens were prepared from the extracted blocks. Special care was

taken to minimize specimen disturbance during extrusion and trimming. The specimens were then extruded into a lubricated ring with a height of 19.0 mm and a diameter of 63.5 mm using a standard extruder. The ring and soil specimens were weighted for unit weight and initial water content determination before testing. In addition, the initial height and weight of each specimens were measured to determine the initial unit weight, void ratio and water content respectively.

Table 1 Consistency limits of the extracted clay

Block ID	Liquid limit, W_L (%)	Plastic limit, W_p (%)	Plasticity index, I_p (%)	Liquidity index, I_L
Bloc 1	62.0	25.0	37.0	1.30
Bloc 3	66.0	25.0	41.0	1.60

To perform CRS test, a novel oedometer cell prepared at Université de Sherbrooke is used (Maleksaeedi et al. 2018). Shown in Figure 2. Prior to each testing, the water compartment and porous stone of the oedometer cell were saturated with using distilled water. Subsequently, the specimen was placed on top of the porous stone with a filter paper and then, the mechanical support and rigid wall loading were placed. The CRS cell was afterwards closed with rigid platen while the loading cap on top of the specimen covered with a filter paper. At the end, the CRS cell was fixed on the loading frame.



Figure 1. Cavities and holes on a block of soft extracted clay

To ensure the continuity between the ring and specimen during the test, each tested specimen was slightly loaded to 10 kPa. Afterwards, the cell is filled with de-aired water and the water volume/pressure controller at the base is connected to the cell to impose a back pressure of 30 kPa. Using GDSLAB software

programmed by GDSinstrument Inc., the back pressure and the vertical stress were gradually increased to reach a target effective vertical stress around 10 kPa. Once the target pressures were reached, each specimen remained under constant effective stress overnight. Periodically the CRS cell was flushed at the top to minimize the amount of trapped air in the cell and specimens. Afterwards, for all CRS consolidation tests, another water volume/pressure controller was connected to the cell at the top to impose the target pore water pressure of 30 kPa while the controller connected to the base was set into volume control condition to act as a pressure transducer. Prior to loading, the settlement due to saturation phase and initial loading were measured to correct the initial void ratio during the loading phase.

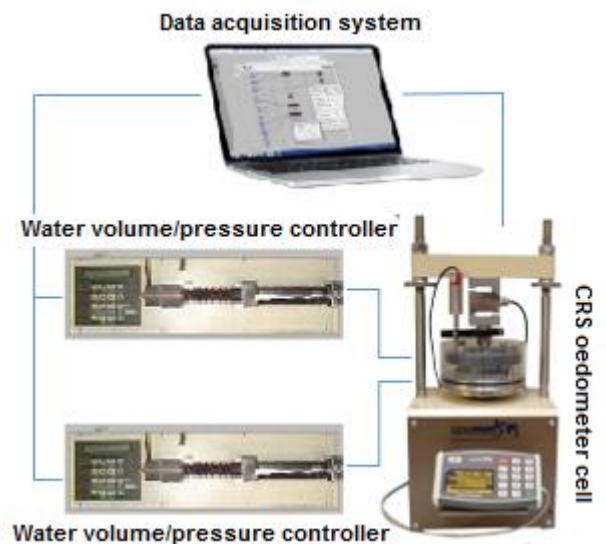


Figure 2. CRS oedometer setup used in this study

In total, four CRS tests, denoted here as CSR1, CSR2, CSR3, CSR4 were performed. The strain rates varied from 0.05%/hr to 2.0%/hr. Both CSR3 and CSR4 underwent a similar strain rate. The only difference between these two specimens were the abundance of cavities where CSR3 had more holes and possible drainage paths compared to CSR4.

After completion of loading path, the induced excess pore water pressure was allowed to dissipate under virtually zero unloading strain rate level (i.e. 0.0001%/hr). After finishing each test, the water content of specimen was measured and recorded.

4 CONVENTIONAL OEDOMETER TEST

In order to have reference observations for CRS tests, two conventional oedometer tests, denoted here as Bloc1 and Bloc3 were performed. These specimens were extruded from the blocks of clay extracted from the site. For each test, after placing the specimen in the consolidation cell, the loading cap was carefully put on

the specimens in order to transmit vertical loads. No filter paper was used to avoid enmeshment of fine soil particles with fibre of filter paper. The specimens were put under minimal stress before adding the distilled water at room temperature to the conventional oedometer cell to saturate them. The Bloc1 and Bloc3 were subsequently loaded incrementally up to around 300 kPa. Each loading step was held for 24 hours. It is assumed that at the end of 24-hour, the excess pore water pressure is back to zero and new applied total stress is the summation of previous pore water pressure and new effective stress.

During each load increment in the standard oedometer apparatus, the settlement was monitored to decide when to proceed to the next load increment. Using the recorded settlements, the induced deformations under each vertical stress were calculated. After finishing the tests, the water content of specimens was measured and recorded.

5 RESULTS AND DISCUSSION

5.1. Generation of excess pore water pressure

Figures 3a and 3b illustrate the variation of excess pore water pressure over time and with the increase of average effective stress for tested specimens. As it can be seen in Figure 3a., little amount of excess pore water pressure were generated at relatively low strain rates and as strain rate increased, the pore water pressure increased. It seems that the pace of pore water pressure generation increased over time as strain rate increased. Based on Figure 3a, lower than 1.0%, the u_b/σ_v ratio remained within the range of 3% to 16% which is close to the range set by ASTM D4186/D4186M for CRS tests. On the other hand, beyond the 1.0%, higher excess pore water pressures were recorded while the u_b/σ_v ratio varied from 5.0% to 25%. As stated by Mesri and Feng (2018), higher values of u_b/σ_v ratio indicate a non-uniform distribution of void ratio and effective stress through the specimen height. This observation might be related to role of cavities and drainage paths during the loading phase. For relatively low strain rates, the presence of cavities and holes did not allow the excess pore water pressure to be built. On the other hand, at 2.0%/hr strain rate, relatively rapid loading possibly caused the blockage in the cavities and thus, generated higher excess pore water pressure. From practical perspective, this indicates that at shallow depth of a soft clay deposit where cracks and cavities might be present, if the soil deposit is slowly loaded, it might act as the drainage boundary of deeper depth while it can undergo deformation. However, if loading rate increases, the blockage of the drainage paths might result in changing the location of drainage boundaries. In mine tailing management, this is particularly of important for subaerial depositional techniques where bled, dried and consolidated tailing zone receives tailing discharges at various velocities.

5.2. Stress-strain relationship

Figure 4 shows the variation of stress-strain curves obtained from CRS test with different strain rates. Clearly, as the strain rates increased from 0.05%/hr to 1.0%/hr, the curves were shifted forward indicating the increase in pre-consolidation pressure. The results herein confirm the observation made by others such as Claesson, (2003), Vaid et al. (1979) and Nash et al. (1992) Jarad et al. (2017) about the effect of strain rate on the pre-consolidation pressure. Comparing with the IL test results for Bloc3, it seems that the proper strain rate to reproduce the Bloc3 is between 0.5%/hr to 1.0%/hr.

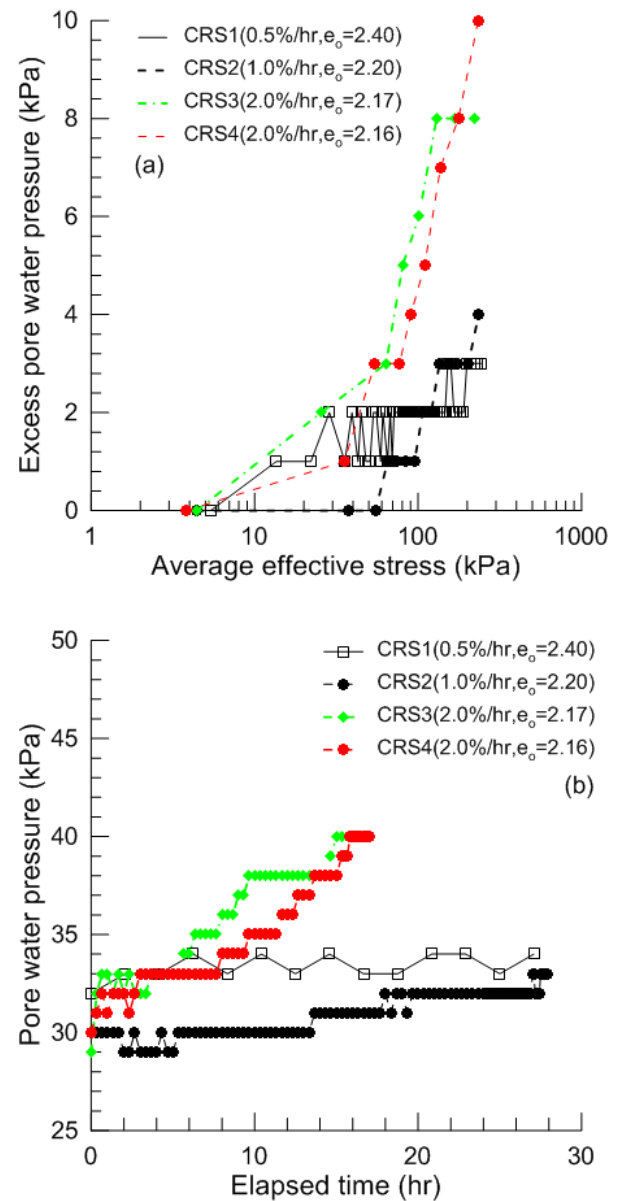


Figure 3. Generation of pore water pressure (a) versus effective stress (b) versus time

For specimens CRS3 and CRS4 with the strain rate of 2.0%/hr, similar pre-consolidation pressures were observed compared to the IL test of Bloc1. This indicates the fact that for a given soil deposit, there might be more than one proper strain rate to reproduce the stress-strain behavior depending on the depth of extraction. In the meantime, Figure 4 shows that as strain rate changed beyond 1.0%/hr, the pre-consolidation pressure slightly decreased while the slope of normally consolidated state, the compression index, increased compared to strain rates lower than 1.0%/hr. This variation can be attributed to the local rearrangement of particles and possible collapse of cavities and holes under relatively higher strain rates. The observation made here confirmed the findings of Watabe et al. (2012) and Watabe and Leroueil (2015) regarding the relationship between compression index and strain rate changes.

Regarding the determination of pre-consolidation pressure in conventional oedometer testing, Kirstein and Liu (2017) suggests that the Casagrande (1936) method results in less accurate determination of pre-consolidation pressure compared to bilogarithmic approaches like Onitsuka et al. (1995) approach. In addition, these approaches are essentially graphical methods without solid physical meaning.

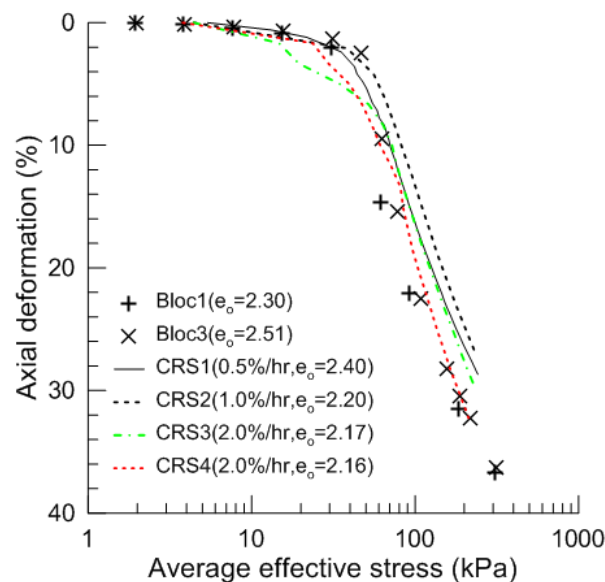


Figure 4. Stress-strain curve obtained by CRS and IL tests on Saint-Charles-Borromée clay

Recently, Karray et al. (2015), Ethier et al. (2016) and Elbeggo et al. (2019) suggested the use of shear wave measurement with piezoelectric ring-actuator technique (P-RAT) to determine and verify the value of pre-consolidation pressure of soft clays. Such approach can also be explored to complement traditional approaches like Casagrande (1936) method in finding the pre-consolidation pressure. Regarding the CRS test, studies of Holm (2016) suggests that the use of

Casagrande (1936) method in the evaluation of test results is questionable since it is designed for IL tests. Nevertheless, it is concluded that the strain rate might not have significant effect on the applicability of pre-consolidation pressure determination methods. Further studies are required to develop a better representative evaluation method designed for CRS tests.

5.3. Hydraulic conductivity

The data analysis also suggests that the steady state factor (F) for all CRS tests were higher than 0.4. Yet, this factor fluctuated and slightly decreased during the loading phase. Consequently, the transient condition was minimized, and analysis can be continued as steady state condition using Eq. 5 to Eq. 8. Using these equations, the coefficient of consolidation and hydraulic conductivity of Saint-Charles-Borromée clay were obtained.

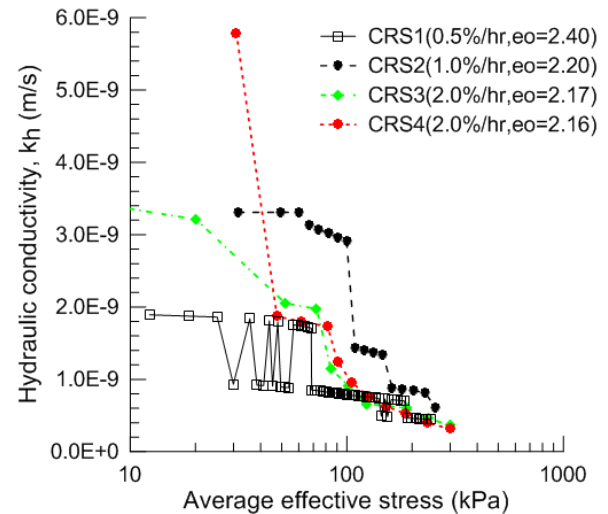


Figure 5. Variation of hydraulic conductivity with average effective stress at different strain rates

Figure 5 and Figure 6 depict the variation of hydraulic conductivity and the coefficient of consolidation at different strain rates. As it can be seen, within the over-consolidation region, the Eq. 7 and Eq. 8 failed to accurately capture reliable results particularly for relatively low strain rates. This is probably attributed to the low generation of excess pore water pressure in this region as specimens were loaded slowly which may result in mathematical instability of Eq. 7 and Eq. 8. In fact, the presence of drainage paths such as holes and cavities due to the plant roots throughout the height of specimens resulted in an accelerated dissipation of excess pore water pressure. The obtained results highlight the fact that the evaluation of CRS test results is limited to the generation of excess pore water pressure within the tested specimens and the presence of any type of drainage path might complicate the data analysis

process. However, as excess pore water pressure is built, Eq. 7 and Eq. 8 permit the calculation of $k_{h,n}$ and $c_{v,n}$.

Based on Figure 5, as the average effective stress increases the hydraulic conductivity decreases. Similar observation were made by Maleksaeedi et al. (2018) on sensitive clays. The $k_{h,n}$ for all specimens in this study ranged from around $6.0e-9$ m/s to $5.5e-10$ m/s. Studies of Adams (2011) on the hydraulic conductivity of fine grained soils suggest that a CRS test and constant head techniques provide the same measurement of average hydraulic conductivity even when non-uniformities are present. Thus, as long as the non-uniformities are spread over the area of specimen and perpendicular to the axial loading direction, it might not affect the hydraulic conductivity during CRS test. This assumption was supported by the observation made from CRS3 and CRS4 specimens.

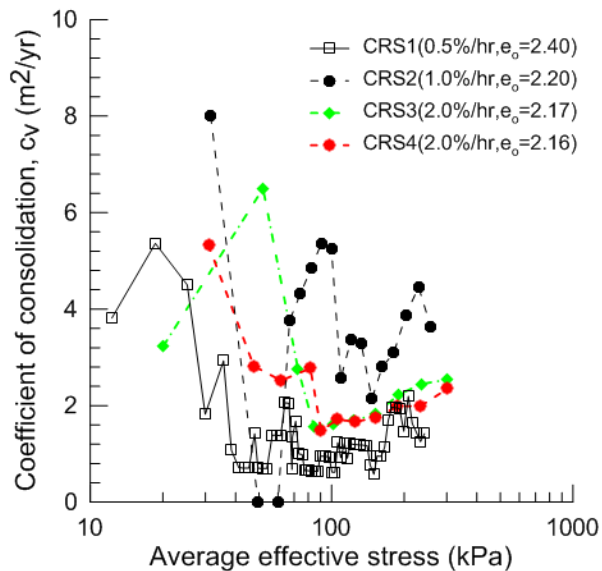


Figure 6. The effect of strain rate changes on the coefficient of consolidation

5.4. Coefficient of consolidation

Looking at Figure 6, the variation of coefficient of consolidation, c_v , depends on the strain rate as stated by Gorman (1981) particularly in the initial steps of consolidation. The excess pore water pressure decreases as the average effective stress increases and at the end, the c_v values seem to converge. Similar observations were made by Ferrari et al. (2016). They stated that the coefficient of consolidation decreases as effective vertical stress increases and reaches the yield stress while afterwards, it remains approximately constant beyond the post-yield state. Lui et al. (2012) stated that the coefficient of consolidation must be interpreted using both small-and large strain theory depending on the strain rate. It was suggested that the

large-strain theory should be adopted for calculating c_v when strain rate is higher than 0.03%/min. Based on the obtained results, it can be concluded that CRS test can be run with higher strain rate which saves both time and experimental efforts. However, a recommendation like this cannot be considered completely valid unless more extensive CRS tests are performed.

6 CONCLUSION

Using a modified oedometer apparatus designed at Université de Sherbrooke, a series of constant-rate-of-strain consolidation (CRS) tests were performed on Saint-Charles-Borromée clay to study the effect of strain rate changes on overall consolidation behavior. The CRS tests results were also compared with conventional incremental loading test results. The results suggest that the change in strain rate affects the measurement of pre-consolidation pressure, hydraulic conductivity and the coefficient of consolidation. The results suggest that the evaluation of CRS test results is limited to the generation of excess pore water pressure within the tested specimens and the presence of any type of drainage path might complicate the data analysis process.

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