E3S Web of Conferences **92**, 02011 (2019) https://doi.org/10.1051/e3sconf/20199202011
IS-Glasgow 2019

One-dimensional consolidation of overconsolidated clay using Constant Rate of Strain testing

Joren Andries[1,*], *Daniel* Verastegui[1], *Kim* De Beule[1] and *An* Baertsoen[1]

[1]Department of Mobility and Public Works, Flemish Government, Ghent, Belgium

Abstract. The main method for the determination of consolidation parameters in Flanders (Belgium) is still the incremental loading test (IL test). This method can take up to several weeks for some typical Flemish overconsolidated (OC) clays. In theory, the same relationship between settlement and vertical stress can be obtained by performing a constant rate of strain test (CRS test). The main advantages of a CRS test are that the data is continuous and that the test can often be completed considerably faster than an IL test. In this paper, results of both IL and CRS tests on two undisturbed stiff OC clay samples of the same geological formation (Maldegem formation deposited during the Paleogene period) were compared. CRS tests were performed based on ASTM D4186, but constant stress stages were controlled using effective vertical stress instead of total vertical stress as most important adjustment to the standard. In addition, special attention was paid to the development of initial swell pressure and selecting an appropriate rate of strain for this clay with a very high plasticity. Similar values for compressibility and hydraulic conductivity were found using both IL and CRS test results. As the duration of a CRS test on this clay with low hydraulic conductivity can also take up to a few weeks, the time saving aspect of the test was found to be limited for the stiff OC clay tested. The uncertainty in estimating the pre-consolidation pressure and swell pressure was smaller using the continuous CRS test results.

1 Introduction

In geotechnical engineering projects, settlement (or swell) of the soil is kept to a tolerable limit based on the results of Incremental Loading (IL) and/or Constant Rate of Strain (CRS) oedometer tests. The main method for the determination of consolidation parameters in Flanders (Belgium), is still the IL test as described in EN 17892-5 [1]. A disadvantage of this method is that it can take up to several weeks for typical Flemish stiff clays. In theory, the same relationship between settlement and vertical stress can be obtained by performing a CRS test. The main advantages of the CRS test are that the data is continuous and that the test can often be completed considerably faster than an IL test on the same sample. This paper summarises and compares both IL and CRS results of two undisturbed stiff OC clay samples from the Maldegem formation (Paleogene).

One of the most important challenges during a CRS test is selecting a proper strain rate. ASTM D4186-12 [2] prescribes to select a strain rate that will cause an excess pore pressure ratio between 3% and 15%. As, for a given sample height, the excess pore pressure at a certain strain rate highly depends on the hydraulic conductivity of the soil and as, in most cases, the hydraulic conductivity of the soil sample is not known at the beginning of the test, a number of methods have been developed and described in literature to make a first estimate for a proper deformation rate. The estimation of a suitable strain rate is often based on soil plasticity (liquid limit) [3] or an earlier (IL or CRS) consolidation test [4]. CRS tests at too high strain rates cannot be interpreted correctly, whereas too small strain rates will impede the determination of the hydraulic conductivity of the soil.

The compatibility of the assumed and actual pore water pressure distribution in the sample is also a subject that requires attention during a CRS test. In this paper, calculations are made using the steady state equations of ASTM D4186-12 [2] based on the linear model assuming that the soil has a constant coefficient of consolidation and the strain is parabolically distributed over the depth of the sample as elaborated by Wissa et al. [5] using the small strain theory.

2 Materials

The origin and some relevant geotechnical properties of the OC clays that are used in this study are summarised in Table 1. Both clay samples M1 and M2 were retrieved under the groundwater table using thin walled tubes (inner diameter 104 mm) according to EN 22475-1.

In the laboratory, the clay samples were extruded from the thin walled tubes. Undisturbed samples were pre-trimmed to a diameter slightly larger than the cutting ring that houses the samples in the testing devices. A

* Corresponding author: joren.andries@mow.vlaanderen.be

cutting ring of diameter 70 mm and height 20 mm was used for the CRS samples and a cutting ring of diameter 63.5 mm and height 20 mm for the IL samples. Finally, the top and bottom surface of the specimens were trimmed. Trimmings near the sample were used to determine the initial water content (Table 1).

Table 1. Soil origin and properties

	M1	**M2**
location	Waasmunster (Onshore near the river Durme)	North Sea (Offshore near Zeebrugge)
depth	18.0 – 18.5 m (from ground surface)	16.0 – 16.5 m (from seabed)
level (with reference to Belgian low seawater level)	-13.8 – -14.3 mTAW (from Belgian vertical reference level)	-26.4 – -26.9 mTAW (from Belgian vertical reference level)
geologic formation	Maldegem Formation (Paleogene)	Maldegem Formation (Paleogene)
soil classification (USCS)	CH	CH
liquid limit (Casagrande method) (EN 17892-12)	123 %	107 %
plastic limit (EN 17892-12)	27 %	27 %
natural water content w_n (EN 17892-1)	34.4 – 35.5 %	30.4 – 32.5 %
initial void ratio e_0	0.88 – 0.91	0.78 – 0.85
particle density (helium pycnometer)	2.66 Mg/m³	2.68 Mg/m³

3 Procedures

On each sample two IL tests according to EN 17892-5 [1] and two CRS tests according to ASTM D4186-12 [2] were performed. Four subsamples (IL1, IL2, CRS1 and CRS2) were therefore taken from each sample (M1 and M2). An overview of both the CRS setup and the IL setup can be found in Fig. 1 and Fig. 2. All tests were carried out at a temperature of (20 ± 2) °C.

3.1 Saturation and prevention of swell

As OC clay tends to swell when it comes into contact with water, dry porous disks are used underneath and on top of the sample without the use of filter paper.

There are two notable differences between the IL and the CRS procedures when it comes to saturating the soil and avoiding any swelling:

1) While during IL testing the sample is loaded to the next load step in the prescribed sequence until no further swell is detected, during the saturation stage of a CRS test the height of the sample is kept constant and swell pressure is measured;

2) Unlike the IL test in which backpressure cannot be applied, during CRS testing a backpressure of 500 kPa was used to saturate the specimen and the system.

3.2 Loading and unloading the sample

In the load sequence of an IL test, the vertical stress is typically increased by a factor of two for each stage. In this study, each vertical stress level beyond the swell pressure was maintained for a period of 3 days. Based on the deformation readings, primary consolidation was always completed for all OC clays tested.

Loading and unloading during a CRS test is carried out at a constant rate of strain. This can be controlled using the displacement reading of the load frame (LF) or using the reading of an external displacement transducer (LVDT) attached to the load ram and mounted on the top of the consolidation cell. The soil is drained from the top of the clay sample and the (excess) pore pressure is measured at the undrained base of the sample. According to ASTM D 4186-12, a constant axial force should be maintained between stages of loading and unloading until base excess pressure has dissipated to nearly zero. This paper (§ 4.5) compares the constant load stage as described in the ASTM with a constant effective vertical stress stage.

Fig. 1. CRS setup

Fig. 2. IL setup

4 Results and discussion of CRS tests

4.1. Swell pressure and stiffness of the cell

After sample installation, water was introduced to the sample while maintaining constant height. Only after the load cell reached a constant reading (corresponding to the swell pressure), the cell was gradually pressurised to 500 kPa backpressure. Fig. 3 and Fig. 4 show two ways of doing so. During saturation of sample M1, the reading of the external LVDT was kept constant. During saturation of sample M2, the displacement reading of the load frame was kept constant.

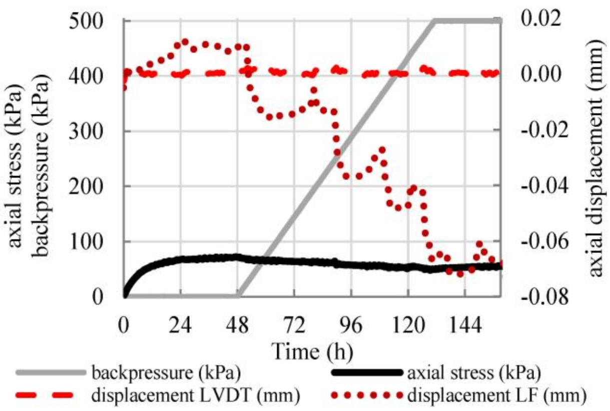

Fig. 3. Saturation of M1_CRS2

Since the consolidation cell does not have an unlimited stiffness, the load frame moved down almost 0.1 mm (0.5% axial strain) during the saturation of M1_CRS2 in order to keep the displacement of the LVDT zero when the cell was pressurised to 500 kPa. This is undesirable because it allows partial swelling of the sample as the sample is unloaded. As a result, the CRS test started at a slightly higher initial void ratio and a corresponding lower axial stress (55 instead of 70 kPa).

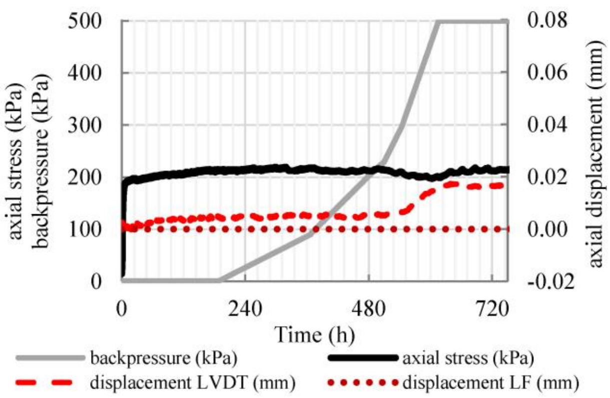

Fig. 4. Saturation of M2_CRS2

When the displacement of the load frame is kept zero (as for sample M2), the LVDT measures the deformation of the cell. In fact, the height of the sample did not change and the displacement reading of the LVDT can be zeroed after this stage. In the setup described in this paper, the authors recommend to keep the load frame position constant when increasing the backpressure.

There were a couple more differences between the saturation procedures of M1 and M2. M1 was saturated from both sides of the sample, while M2 was only saturated from the top in order to be able to measure the excess pore water pressure at the bottom of the sample (which always remained smaller than 10 kPa). The saturation of M1 was performed in 1 week, as it was done from two sides (but with less control over the distribution of the pore water pressure in the sample). The saturation of M2 happened (over-) cautiously slow and consequently took 1 month.

4.2. Stiffness of the load frame

When loading or unloading the sample, it must also be taken into account that the loading system itself does not have an unlimited stiffness. It is therefore better to use the reading of the external LVDT (mounted on the load ram) to control the rate of deformation.

Fig. 5 shows the rate of deformation during a CRS loading stage on M2. The LVDT reading is used to control the velocity of the load frame. Although there might be slightly more fluctuations in the velocity of the load frame when using an LVDT as external control (than is the case when the velocity of the load frame is set to a constant value), the average rate of deformation corresponds better to the actual deformation of the sample. During each loading stage, the rate of strain based on the LVDT reading remained very well within the acceptable deviation of 10% from the target rate of strain (according to ASTM D4186-12 cyclic variations should be smaller than ± 10%).

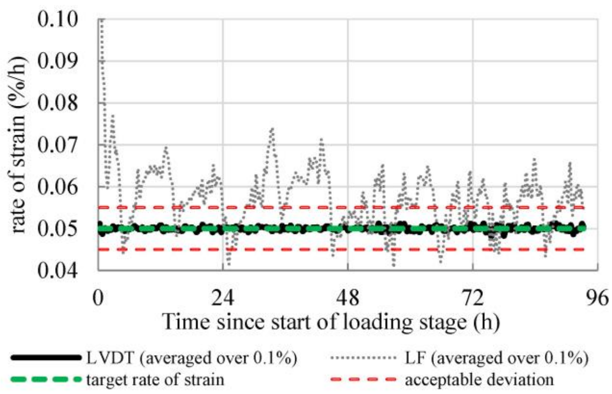

Fig. 5. Strain rate during the loading stage of M2

4.3. Estimating an appropriate rate of strain

ASTM D4186-12 [2] suggests a starting value for the strain rate based on the Unified Soil Classification System. Both soils that are subject of this paper can be classified as CH material and therefore 0.10%/h could be a good starting value if an extra test to find the most appropriate rate can be carried out. As only a limited amount of undisturbed homogeneous material of each soil was available in the laboratory and because of the very high plasticity of the soil, an additional strain rate estimation based on the liquid limit [3] was made to limit the base excess pressure ratio to 20%. Based on the values in Table 2 a rate of strain of 0.05 %/h or 0.10 %/h was chosen for the loading stage. The rate of strain during unloading was selected as half the loading strain rate.

Table 2. Estimating an appropriate rate of strain

	M1	M2
ASTM D4186 (2012) [2]	0.10 %/h (CH)	0.10 %/h (CH)
Gorman (1981) [3]	0.025 %/h (LL = 123%)	0.05 %/h (LL = 107%)
selected rate of strain during loading	CRS1: 0.10 %/h CRS2: 0.05 %/h	CRS1: 0.05 %/h CRS2: 0.05 %/h
selected rate of strain during unloading	CRS1: -0.05 %/h CRS2: -0.025 %/h	CRS1: -0.025 %/h CRS2: -0.025 %/h

4.4. Strain rate evaluation

According to ASTM D4186-12 [2] the selected strain rate should cause a base excess pressure ratio Δu_{base} / σ_v between 3% and 15%. European standards such as NS 8018 [6] (Norwegian standard) and SS 27126 [7] (Swedish standard) mention that the base excess pressure ratio should not exceed 10%, however it might be acceptable that the ratio is higher in some parts of the test if it remains under 20%.

Fig. 6 shows the increase in base excess pore water pressure divided by the total vertical stress as a function of the average effective vertical stress $\sigma'_{v,avg}$ during loading and unloading of each CRS test. It can be concluded that the selected strain rates are still fairly high for this type of soil, but not excessively high. Strain rate selection based on the liquid limit of an OC clay can help to estimate the correct order of magnitude for the strain rate. Although, in the case of sample M1, it significantly underestimated the strain rate due to a slightly higher liquid limit.

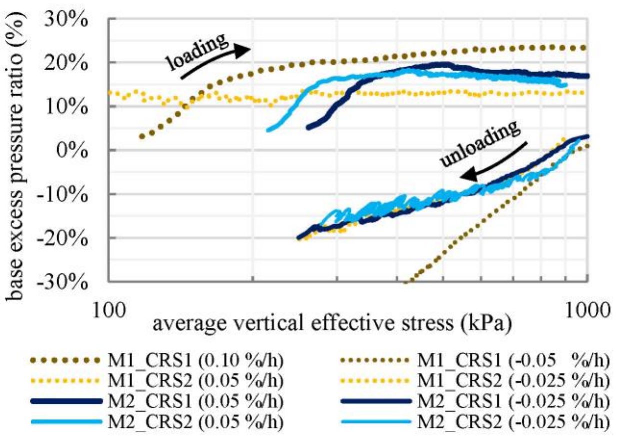

Fig. 6. Base excess pore pressure ratio as a function of average vertical effective stress during loading and unloading

The normalised strain rate β as defined by Lee [8] always remained well below 0.1. Consequently, the small strain theory can be applied as in this case its solution is not too sensitive to the selected strain rate.

4.5. Constant stress stage

The average vertical effective stress $\sigma'_{v,avg}$ during a CRS test is often calculated assuming that the distribution of the excess pore water pressure over the height of the sample is parabolic, using equation (1).

$$\sigma'_{v,avg} = \sigma_v - (2/3)\Delta u_{base} \tag{1}$$

In this paper, the same equation is used between stages of loading and unloading while maintaining a constant $\sigma'_{v,avg}$ and while excess pore water pressure dissipates. In order to illustrate the difference with the procedure described in ASTM D4186-12, the constant stress stage in the first CRS test on sample M2 (CRS1) is performed with constant effective stress while a second sample of the same OC clay is tested with a constant load stage (CRS2) as described by the ASTM.

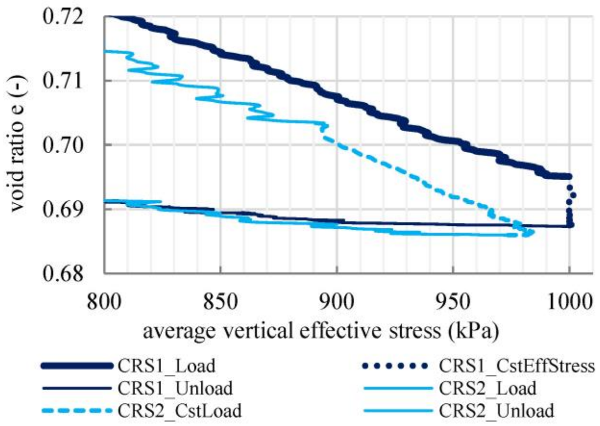

Fig. 7. Constant stress stage between loading and unloading

The loading stage of CRS1 in Fig. 7 (which is a detail of Fig. 9) runs up to the set value of 1000 kPa effective stress with a constant rate of strain and then maintains a constant effective stress while excess pore pressure dissipates. The loading stage of CRS2 on the other hand, is aborted at a total stress of 1000 kPa and then maintains a constant load (or total stress). While the excess pore pressure dissipates, both effective stress and void ratio change in an uncontrolled way. The slope of CRS2 in the stress range between 900 kPa and 1000 kPa may look similar to the slope of CRS1, but this part of the test was not carried out at a constant rate of strain when the axial load is maintained constant.

The authors are aware of the fact that equation (1) may not exactly match the real average effective stress during the constant stress stage. Nevertheless, they prefer and suggest maintaining a constant effective stress over maintaining constant load to allow the excess porewater pressure to dissipate. In this way the loading stage with a constant rate of strain is prolonged and the constant stress stage itself requires less time.

5 Comparison of CRS with IL results

Fig. 8 and Fig. 9 show the settlement data from both CRS and IL tests from the 1D-consolidation of respectively M1 and M2.

5.1. Swell pressure

In order to prevent swell during an IL test, the sample is loaded beyond the swell pressure. By doing so, the void ratio already slightly decreases during the first (swell) stages of the IL test.

The main reason why the swell pressure of M1_CRS2 is smaller than the swell pressure of M1_CRS1 is because the CRS test on the second subsample of M1 was only performed after the CRS test on the first subsample was completed and M1_CRS2 might already have been somewhat disturbed. The difference in swell pressure and the slightly larger initial void ratio can be an indicator thereof. It is advisable to install the sample in the apparatus immediately after extruding it from the tube, which was not possible for M1_CRS2. All four subsamples of M2 were tested at the same time immediately after extruding the thin walled tube.

5.2. Pre-consolidation pressure

The pre-consolidation pressure is determined in Table 3 and Table 4 using Casagrande's procedure [9]. Compared with the discrete results of IL tests, it is easier to find the point of minimum radius using continuous CRS test results. Especially because an OC clay sample can build up a swell pressure close to the pre-consolidation pressure and as a result only a few points of an IL test before the pre-consolidation pressure can be used. The uncertainty while determining the pre-consolidation pressure of an OC clay can be reduced when a CRS test is performed. One way of decreasing the uncertainty of the pre-consolidation pressure with IL testing would be to use a loading sequence in which the vertical stress is increased by a factor that is much smaller than 2, in which case the CRS test would be a quicker alternative.

5.3. Compressibility

The compression index C_c and recompression index C_r are calculated in Table 3 for M1 and in Table 4 for M2. C_c was calculated based on the data during the loading stage in the effective stress range 500 kPa - 1000 kPa. C_r was calculated based on the data during the unloading stage in the effective stress range 500 kPa - 250 kPa. These common stress intervals were chosen simply to be able to compare CRS and IL test results. An advantage of the CRS test is that it imposes less constraints on the choice of stress range.

There is good agreement between the results obtained from the CRS test compared with the results from the conventional IL test. The difference between CRS results and IL results is of the same order of magnitude as the difference between two IL tests. As the loading stage itself (without the constant stress stage) takes less time during the CRS test, the values for C_c obtained from the CRS test are slightly smaller because less creep is measured (also see § 5.5).

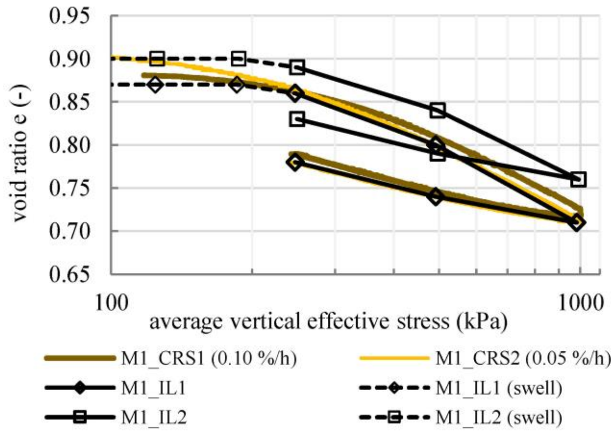

Fig. 8. CRS and IL results on M1

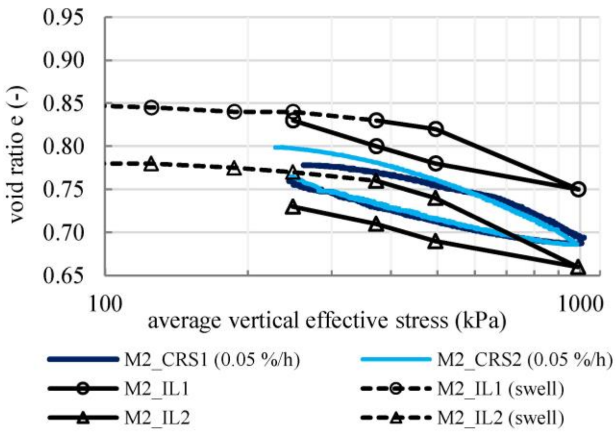

Fig. 9. CRS and IL results on M2

Table 3. Pre-consolidation pressure and compressibility of M1

	CRS1 *(0.10%/h)*	CRS2 *(0.05%/h)*	IL1	IL2
pre-consolidation pressure σ'_c (kPa)	360	310	not enough datapoints to accurately determine σ'_c	
compression index C_c (-)	0.27	0.28	0.30	0.27
recompression index C_r (-)	0.14	0.14	0.13	0.13

Table 4. Pre-consolidation pressure and compressibility of M2

	CRS1 *(0.05%/h)*	CRS2 *(0.05%/h)*	IL1	IL2
pre-consolidation pressure σ'_c (kPa)	500	450	not enough datapoints to accurately determine σ'_c	
compression index C_c (-)	0.20	0.23	0.23	0.27
recompression index C_r (-)	0.14	0.13	0.17	0.13

5.4. Hydraulic conductivity

Fig. 10 shows the calculated hydraulic conductivity k during the loading stage of both IL and CRS tests on M1 and M2. Results for the hydraulic conductivity in the normally consolidated range match reasonably well. Differences might be caused by samples being not perfectly saturated during an IL test and by the measurement uncertainty of the void ratio during both IL and CRS testing. As c_v is prone to change in the OC range, while the theory of Wissa [5] assumes a constant c_v, the hydraulic conductivity in the beginning of the CRS test might be overestimated. [10, 11]

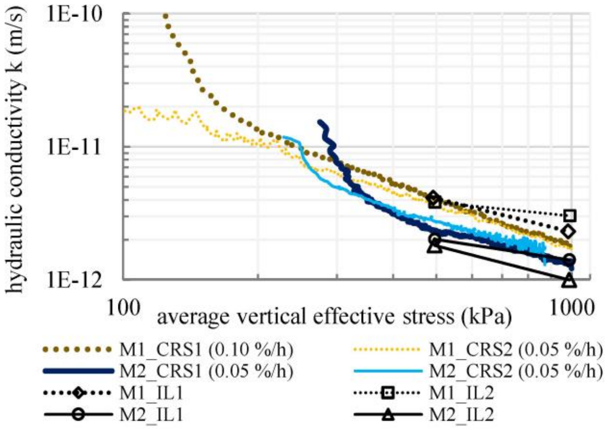

Fig. 10. Hydraulic conductivity of M1 and M2 during loading

5.5. Duration of the test

As the first seven steps of the prescribed IL sequence were quickly finished (due to swelling), only the last two loading steps could be completed on the OC clay samples. In the followed procedure, an IL test on OC clay with a large swelling pressure therefore does not take tremendously long. It could however be considered to decrease the load increment ratio of the load sequence to be able to determine σ'_c more accurately. The CRS test covered a slightly larger stress range. Moreover, the CRS samples were saturated very slowly, which also requires a substantial amount of time for a clay with a low hydraulic conductivity. Table 5 summarises the time needed for the (un)loading stages of the CRS and IL tests reported in this paper.

Table 5. Duration of the different test stages (days)

	M1				M2			
	CRS1	CRS2	IL1	IL2	CRS1	CRS2	IL1	IL2
loading	4	8	7	7	4	4	7	7
constant stress	3	1			3	4		
unloading	3	6	6	6	6	6	9	9
constant stress	2	1			3	4		
total time (days)	**12**	**16**	**13**	**13**	**16**	**18**	**16**	**16**

6 Conclusions

Following conclusions are drawn from the CRS and IL consolidation tests on OC clay performed in this study:

- Care needs to be taken not to register a deformation of the equipment as a deformation of the sample. In the CRS setup described in this paper, this was prevented by keeping the load frame displacement constant when pressurising the cell (saturation phase) and by controlling the external LVDT while loading and unloading the sample.
- The liquid limit of an OC clay can help to estimate a proper strain rate. Especially for clay with a very low permeability, the starting value mentioned in ASTM D 4186-12 would result in very high excess pore water pressures that compromise the interpretation of the results.
- Between stages of loading and unloading a stage with constant average vertical effective stress was used to allow excess pore water pressure to dissipate as an alternative for a stage with constant axial force.
- Similar values for compressibility and hydraulic conductivity were found using CRS and IL testing. Moreover, the uncertainty in estimating the pre-consolidation pressure and swell pressure of an OC clay was smaller using continuous CRS test results.
- As the duration of a CRS test on a soil with low hydraulic conductivity can also take up to a few weeks, the time saving aspect of the test was, in this specific case, found to be limited.

The authors would like to thank Sofie Dooms, Nathalie Moerman and Miranda Seigneur for their help in conducting the laboratory tests.

References

1 CEN, EN ISO 17892-5 (2017)

2 ASTM Int'l, ASTM D4186/D4186M-12 (2012)

3 C. T. Gorman, Kentucky Transportation Center Research Report **UKTRP-81-7** (1981)

4 A. T. Özer, E. C. Lawton, S. F. Bartlett, Can. Geotech. J. **49**: 18-26 (2012)

5 A. E. Z. Wissa, J. T. Christian, E. H. Davis, S. Heiberg, ASCE J. of the Soil Mechanics and Foundations Division. **97**:1393-1413 (1971)

6 Standards Norway, NS 8018 (1993)

7 Swedish Standards Institute, SS 27126 (1991)

8 K. Lee, Géotechnique, **31**: 215-229 (1981)

9 A. Casagrande, 1st Int. Conf. on Soil Mechanics and Foundation Engineering, **3**: 60-64 (1936)

10 T. C. Sheahan, P. J. Watters, ASCE J. Geotech. Geoenviron. Eng. **123** : 430-437 (1997)

11 P. J. Fox, H. P. Pu, J. T. Christian, ASCE J. Geotech. Geoenviron. Eng. **140**: 04014020 (2014)