Short and long term compressibility of rock particle assemblages

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ABSTRACT

During construction and first impoundment of rockfill dams, significant deformations can occur due to crushing of highly stressed rock particles. In order to improve the general understanding of this phenomenon, short term and long term compressibility of assemblages of rock particles was studied with one-dimensional compression tests in a large oedometer cell. The main results from the test program revealed that short term compressibility tends to be more important for samples with a wide grain size distribution curve, a low density and when the samples were wetted or saturated prior to loading. With respect to long term compressibility, it appears that the ratio between the time-dependent compressibility index and the compressibility index, $C_{\epsilon,t}/C_{\epsilon,\sigma}$, tends toward a constant of about 0.02 for dry rock particles and 0.03 for saturated samples. Furthermore, the compressibility was successfully related to a breakage factor, highlighting the importance of particle breakage on deformations of rockfill.

RÉSUMÉ

La compressibilité à court terne et à long terme d'assemblages de particules de roche granitique a été étudiée lors d'essais de compression uniaxiale à l'intérieur d'une nouvelle cellule oedométrique de grand diamètre. Il a été démontré que la compressibilité à court terme est plus importante pour des assemblages de particules d'étalement granulométrique plus étalé, de faible masse volumique et pour des assemblages préalablement saturés ou humidifiés. Il a également été démontré que le ratio entre l'indice de fluage et l'indice de compressibilité, C_{e,t}/C_{e,o}, est de 0,02 pour les échantillons secs et de 0,03 pour les échantillons humides. Finalement l'analyse des distributions granulométriques des échantillons testés a confirmé la relation entre la fragmentation des particules et la compressibilité de l'assemblage.

1 INTRODUCTION

Since the 1960's, rockfill dams are commonly used as a water barrier structure. The most common types are the concrete face rockfill dam (CFRD), the earth core rockfill dam (ECRD) and the asphaltic core rockfill dams (ACRD). These types of design consist of an impervious material and a rockfill structure providing the stability of the dams. For the La Romaine hydroelectric project, located on the north coast of Quebec, rockfill dams are an economical choice when rocks of good quality can be found in the vicinity of the project.

As for the mechanical behavior of such structures, close monitoring of certain rockfill dams over the last 50 years revealed that significant deformations may arise from the first impoundment and additional deformations often cumulate during the lifetime of the dam. In order to improve the comprehension of the physical mechanisms behind such deformations, several large scale testing program were carried out on different rockfill types (Marsal, 1967; Fumagalli, 1969; Marachi, 1969; Becker, K. et al., 1972; Marsal, 1973). They revealed that the crushing of highly stressed rocks was the main mechanism behind rockfill deformation.

Testing rockfill materials is technically difficult since large laboratory equipments are needed. A more common and practical way to determine rockfill properties is to test assemblages of smaller rock particles of the same materials and with the same properties (density, grain size distribution, water content, etc.). Even though the scale effect has a certain influence on the mechanical behavior observed in laboratory, this approach allows determination of rockfill properties at a relatively low cost.

This paper presents the results of an experimental program consisting of multiple uniaxial compression tests on rock particle assemblages in a new large oedometer cell (Cormier & Konrad, 2013).The main objective is to study the effect of certain characteristics of rockfill on its compressibility. More specifically, the testing program is aimed at the study of the influence of grain size distribution, initial density and water state of rock particle assemblages on short term and long term compressibility.

2 BACKGROUND

2.1 Short term compressibility

The behavior of granular materials under one-dimensional compression has traditionally been represented by plotting the change in void ratio against the logarithm of the vertical stress. In this representation, the granular media present a phase of low compressibility, under relatively low stresses, characterized by particle rearrangement and a small degree of particle crushing. Once the applied stress is sufficient to produce significant particle crushing, the vertical deformations are enhanced and a clear curvature in the void ratio-log stress is usually noticeable, a deformation mechanism called clastic yielding by McDowell and Bolton (1998). However, the stress range of interest in the study of rockfill dams is generally below the yielding point marking the onset of significant crushing. Furthermore, under one-dimensional compression, rock particle assemblages still exhibit a certain amount of particle crushing, and thus, significant vertical deformation at those lower values of vertical stresses (Marsal, 1973; Oldecop & Alonso, 2001; Ovalle, Dano et al., 2014). If no yielding point is attained during an oedometer test, the interlocking of the particles is particle particle breakage and improved by rearrangement. Consequently, the sample tested becomes less compressible and the constrained modulus, or oedometer modulus, increases as the vertical stress increases (Pestana & Whittle, 1995). Therefore, the stress-strain curve can accurately be approximated by a power law, as defined by equation 1:

$$\varepsilon (\%) = \mathsf{B}(\sigma/\sigma_r)^m$$
^[1]

where *m* is the slope of the compression curve in a log-log plot, *B* is the vertical strain for a vertical stress ratio, σ/σ_r , equal to unity and σ_r is a reference stress. The reinterpretation by Parkin (1977) of the oedometer test on granular materials, performed by (Kjaernsli & Sande, 1963) implies values of the gradient "m" of about 0,75 for crushed materials. A similar approach was used by Fumagalli (1969) in the interpretation of one-dimensional compression test performed on crushed limestone rocks in a large oedometer cell. The latter found that for the material tested, the value of the exponent "m" varied between 0,4 to 0,6 for well graded materials. Finally, uniaxial compression test performed on Pancrudo Slates by Oldecop and Alonso (2001) revealed that the stressstrain relationships of the specimen tested were mostly linear.

2.2 Long term compressibility

It is well known that granular materials during secondary compression present deformations proportional to the logarithmic of time, as expressed by equation 2:

$$\varepsilon = C_{\varepsilon, t} \log(t/t_0)$$
[2]

where $C_{e,t}$ is the slope of the deformation-log time relationship and to is the time from which strain are measured. This behavior is consistent with the deformation observed in rockfill dams. The secondary compression of highly stessed granular materials and rocks is mostly due to the subcritical crack growth phenomenon (Atkinson, 1984). A phenomenon by which flaws and cracks propagates within the rocks under a constant stress and at a certain velocity until failure of the particle. Although it was well established that the subcritical crack growth was enhanced by the presence of water (Michalske & Freiman, 1982), recent development successfully related the relative humidity surrounding rock particles to the collapse of rockfill and to the timedependant deformation observed in rockfill dams (Oldecop & Alonso, 2001; Oldecop & Alonso, 2007).

Since particle breakage is a predominant phenomenon in both short term and long term compressibility, a proportionality between the deformations in both stages is usually noticeable in laboratory experiments (Mesri & Castro, 1987; Oldecop & Alonso, 2007; Mesri & Vardhanabhuti, 2009).

3 LABORATORY EQUIPEMENT

The special apparatus, depicted in Figure 1, was designed to carry out uniaxial compression tests in a stainless steel oedometer cell on rock particles assemblages. The cell is a fixed ring type and has an inside diameter of 320 mm. The vertical stress applied to the sample is produced by increasing the air pressure in an air spring. As it inflates, the mobile head moves downward and generate a vertical force which is applied to the sample through the piston. The piston is guided through a sleeve of lubricated stainless steel bearings. The air spring can withstand pressure of up to 700 kPa and produce a vertical load of 150 kN. The applied vertical force is measured through a load cell located between the air spring and the piston.





As indicated in Figure 1, another load cell is placed at the base of the oedometer cell and a 25,4 millimeters thick stainless steel disk transfer the vertical load from the bottom of the sample to the load cell. The total vertical force of friction can thus be calculated by equation 3:

$$F_s = P - Q$$
 [3]

where F_s is the total vertical force of friction, P is the applied vertical force and Q is the vertical force measured at the base of the sample. The average vertical stress along the sample height can then be calculated with the equation 4, proposed by D.W Taylor (Taylor, 1942) :

$$\sigma_{\alpha} = \frac{F_{s}}{-\ln(1 - \frac{F_{s}}{P}) \cdot A}$$
[4]

where "A" is the surface area of the tested sample. Although the average vertical stress can be calculated, the friction must be reduced to obtain a fairly uniform vertical stress distribution inside the sample tested. To this effect, three thin layers of polystyrene bands intertwined with silicon grease are placed between the sample and the wall of the oedometer cell. The vertical displacement are measured with a LVDT (Linear variable differential transformer) fixed to the piston, outside of the oedometer cell. The deformations due to the contraction of the piston bellow the LVDT support are corrected to account only on the deformations of the sample. The oedometer cell is designed to withstand water pressure of 1 MPa (Cormier & Konrad, 2013).

4 METHODOLOGY

4.1 Material tested

For this study, a granitic aggregate which consists of about 66 % of granitic gneiss, 24 % of biotite and 9% of hornblend was selected from a quarry located in the Quebec city area. Samples of four different grain size distribution (GSD) curves were tested. All four GSD curves, depicted in Figure 2, have a maximum grain size of 56 millimeters and the minimum grain size varies from fine particle (<0.08 mm) for grain size distribution curves G1, G2 and G3 to 30 millimeters for G4.



Figure 2 : Grain-size gradation curves of the tested samples

The grading characteristics are summarized in Table 1. The samples tested were placed in three equal layers of about 6 millimeters high. Each layer was compacted with an electrical vibrating hammer. Careful visual inspection of each layers surface did not reveal grain breakage after compaction. The maximum dry densities obtained by this method of compaction for each GSD are presented in Table 1.

	D _{max}	D _{min}	D ₆₀	D ₁₀	D ₆₀ / D ₁₀	ρd
	mm	mm	mm	mm		kg/m³
G1	56	< 0.08	16	0,2	90	2250
G2	56	< 0.08	22	0,5	45	2120
G3	56	< 0.08	28	2,3	14	2000
G4	56	30	42	32	1.3	1490

4.2 Testing procedure

Each oedometer test can be divided into four distinct stages. In Stage 1, the sample is first loaded up to a given vertical stress, 1,2 MPa for most tests. The vertical stress is ramped up at a constant rate of 0.2 MPa/hour. In Stage 2, time-dependent deformations are measured for more than 1000 minutes under constant vertical stress. In Stage 3, the dry samples are flooded by filling the oedometer cell with water and the time-dependent deformations are measured for a similar period of time. Finally in Stage 4, the specimen is unloaded and a sieve analysis is performed after drying the sample. The stressstrain relationship of the test performed will be presented in a double logarithmic plot since the stress-strain relationship of assemblages of rock particles are best approximated by a power law. Since a stress reference of 1 MPa (σ_1) was chosen (see equation1), for an applied average vertical stress of 1 MPa, $\sigma_a/\sigma_1=1$. The time dependent vertical deformation will be plotted against the logarithm of the elapsed time for stages 2 and 3 for each test. For specimens which were initially wet or saturated, time-dependent deformations were measured for only one phase of secondary compression.

4.3 Testing program

The testing program consists of 8 oedometer tests. The influence of three characteristics of rock particles assemblages on compressibility was assessed: grain size distribution, initial density and water state (humid, dry and saturated). The testing program was divided in two test series. Series A was aimed at the effect of the GSD on compressibility and Series B was devoted to the effect of initial density and water content. The main characteristics of each test Series are presented in Table 2.

Table 2 : Characteristics of tested sample
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Sample	ρ _d (kg/m³)	Water condition
G1-D100-D	2249	Dry
G1-D99-H	2233	Humid (W _i =6 %)
G2-D100-D	2116	Dry
G3-D100-D	1991	Dry
G3-D100-S	1991	Saturated ¹
G3-D94-D	1874	Dry
G3-D94-S	1874	Saturated ¹
G4-D100-D	1490	Dry

1: The samples were saturated at a constant vertical average stress of 50 $\ensuremath{\mathsf{KPa}}$.

5	RESU	ILTS

5.1 Test Series A

The four samples tested were compacted to the maximal dry density that could be obtained with the compaction method described earlier.



Figure 3 : (left) Stress-strain relationship of samples G1-D100-D and G2-D100-D (Right) Strain records before and after flooding (under constant stress) of samples G1-D100-D and G2-D100-D.



Figure 4 : (left) Stress-strain relationship of samples G3-D100-D and G4-D100-D (Right) Strain records before and after flooding (under constant stress) of samples G3-D100-D and G4-D100-D.

Stress-strain relationships and time-dependant deformation for samples G1-D100-D (Cu=90) and G2-D100-D (C_u=45) are presented in Figure 3. Both of these samples were initially more compressible up to a stress ratio of 0.06 and 0.09 respectively, which may be explained by the reorientation and the rearrangement of the particles. Above these values of vertical stresses, the samples were less compressible. The stress-strain relationship of both samples is best represented by a straight line in a double logarithmic plot. The slopes of the stress-strain curves are similar for both tests and the deformation recorded at an average vertical stress of 1,2 similar as well. The time-dependent MPa are deformations of both samples increased when flooded. After unloading, the final recorded vertical deformations were guite similar, 1,65 % for G2-D100-D and 1,70 % for G1-D100-D. Although the GSD of these samples are different, they present a similar behavior.

The stress-strain relationship and time-dependant deformations of samples G3-D100-D (C_u =14) and G4-D100-D (C_u =1,3) are presented in Figure 4. An increased

compressibility at low stress cannot be observed for those two specimens. For sample G3-D100-D, no clear shift of behavior is noticeable and the stress-strain curve can be well approximated by a power law for the whole stress range applied. As for sample G4-D100-D, the compressibility is enhanced above a vertical average stress of 0,4 MPa ($\sigma_a/\sigma_r=0,4$). At larger stresses, clear crushing noise of the tested materials could be heard, indicating the onset of particle breakage. The time-dependant deformations for this sample can hardly be approximated by a straight line in semi-log scale for the time duration observed. This sample presents more vertical deformations than the other specimens.

5.2 Test Series B

In this series of test, samples of similar density and GSD but with different water content were compared. The stress-strain curves and time-dependant deformations obtained in these tests are presented in Figure 5 to Figure 7.



Figure 5 : (left) Stress-strain relationship of samples G1-D100-D and G1-D100-S (Right) Strain records before and after flooding (under constant stress) of samples G1-D100-D and G1-D100-S.







Figure 7 : (left) Stress-strain relationship of samples G3-D94-D and G4-D94-S (Right) Strain records before and after flooding (under constant stress) of samples G3-D94-D and G4-D94-S.

It can be seen that in every case, the humid or saturated samples are more compressible than the dry ones. A similar observation can be made for time-dependant deformation. The slope of the strain-log time curve is steeper for humid and saturated sample than that observed in dry samples. Furthermore, the slope of these curves increases once the specimen is flooded.

6 DISCUSSION

6.1 Short-term compressibility

The oedometer modulus, or constrained modulus, is frequently used to characterize the short term compressibility of granular specimens. However, this parameter is impractical since it varies with the applied stress. A more convenient way to assess stress-strain compressibility of rock particles assemblage is through a power law, defined above by equation 1. Although two parameters (m and B) are needed to define the stressstrain curves, both are constant for the whole stress range applied on the sample. Values of exponent *m* for every test are plotted against the initial density in Figure 8. It appears that *m* decreases with increasing density and that the water state of the sample does not seem to affect significantly its value. Values of m range from 0,88 for sample G4-D100-S to 0,4 for sample G3-D100-S. Parameter B represents the vertical deformation for a stress ratio (σ_a/σ_1) value of 1. Figure 10 presents the value of parameter B for all the sample tested and indicates that the parameter B decreases with increasing density. Since humid and saturated samples present generally more vertical deformations than dry samples, the recorded vertical deformations for a stress ratio (σ_a/σ_1) of 1 (parameter B) are more important for humid and saturated samples, as indicated in Figure .

6.2 Long-term compressibility

For each test performed, time-dependent deformations were measured before and after flooding of the samples. In addition, test G1-D100-S, G3-D100-S AND G3-D94-S were initially humid or saturated and only time-dependent



Figure 8 : Exponent m vs. initial density

deformation in wet condition could be observed. Let us first define a time-dependent compressibility index, expressed by equation 5 :

$$C_{\varepsilon,t} = \Delta \varepsilon / \log(t/t_0)$$
 [5]

Figure 9 presents the time-dependent compressibility index computed for the six test performed in Series B :



Figure 9 : Time-dependant compressibility index for samples of different water states and different densities

For samples which were initially dry, time-dependent compressibility index before and after the saturation of the samples are indicated as well. This figure first indicates that the time-dependant compressibility index is enhanced when an initially dry sample is saturated. Although the creep index after saturation of an initially dry simple is not exactly similar to the time-dependent compressibility index of an initially saturated sample, it tends toward a similar value. Figure 9 also indicates that the time-dependent compressibility index decreases with increasing density, as previously observed for parameters *B* and *m*.



Figure 10 : Parameter B vs. initial density

6.3 Relationship between short and long term compressibility

Mesri et al. (2009) have shown that, for granular materials including rockfill, the secondary compression index, C_a , is proportional to the compression index, C_c . For such materials, the ratio C_a/C_c varies between 0,01-0,03. A similar approach was used in this paper to verify the link between short and long term compressibility. The time-dependent compressibility index was defined by equation 5 and the compressibility index used in this paper is defined by equation 6 :

$$C_{\sigma,\sigma} = \Delta \varepsilon / \log(\sigma/\sigma_0)$$
[6]

This compressibility index is equivalent to the tangent modulus of the strain-log stress curve. Since the stressstrain curves can be approximated by a power law, the compressibility index can be computed as the logarythm of the applied vertical stress, as expressed by equation 7:

$$C_{\varepsilon,\sigma} = \frac{\varepsilon}{d \log \sigma_a} = 2,3 \cdot B \cdot m \cdot \left(\frac{\sigma}{\sigma_1}\right)^m$$
[7]

For all tests performed, the time-dependent compressibility index, $C_{\epsilon,t}$, is plotted against the compressibility index, $C_{\sigma,t}$ in Figure 10. Results of oedometer tests performed on quartzite sand sample, from Ovalle, Dano et al. (2014), for different water states, are presented as well.



Figure 10. Relationship between time-dependent compressibility index and compressibility index

As observed by Mesri and Castro (1987), the timedependent compressibility index is proportional to the compressibility index for the tested material. However, results presented in Figure 10 indicate that the ratio between those two parameters is higher for wetted or saturated sample than for dry samples. Reinterpretation of results presented by Ovalle & al. indicate a clear influence of the water content on the ratio $C_{\rm e,t}/C_{\sigma,t}$ as well, which tend toward a constant value of about 0,012 for dry samples and 0,025 for saturated samples. Oldecop and Alonso (2007) also observed that dry samples presented a lower ratio between time-dependent compressibility index and compressibility index than wet or saturated samples. The ratios $C_{\epsilon,t}/C_{\sigma,t}$, which is equivalent to the ratio C_{α}/C_{c} , fits well with the experimental observations made by Mesri et al. (2009) for granular materials.

6.4 Particle Breakage

As mentioned above, an important mechanism in rockfill deformations is related to the breakage of highly stressed rock particles and the subsequent rearrangement of the crushed particles. This mechanism is predominant in both compression phase and during secondary the compression (creep) of a one-dimensional compression test on rock particle assemblages. The total deformation should therefore be closely related to a breakage index. As described in the test procedure, a sieve analysis was performed on each sample once the test was completed and the breakage ratio, B_g, developed by Marsal (1973), was calculated. In Figure 11, the breakage ratio is plotted against the plastic work, obtained by integrating the stress-strain curves of each test. Only the work during the compression phase that could be approximated by a power law is considered since the deformation at low stress is due to particle rearrangement rather than grain crushing. Figure 11 indicates a clear relationship between the plastic work and the breakage index. Well graded samples present lower plastic work and less breakage than samples characterised by a more uniform grain size distribution. Also, for a same amount of plastic work, test G4-D100-S presents significantly more breakage than the other tests. However, additional test of similar grain size distribution (G4) and different amount of plastic work would be necessary to evaluate properly the relation between breakage and plastic work for such specimens.



Figure 11. Correlation between plastic work and breakage index

7 CONCLUSIONS

In this paper, short and long term compressibility of rock particle assemblages in one dimensional compression test have been studied. The influence of grain size distribution, initial density and water state on the compressibility has been studied. The stress-strain compression curves were fitted using a power law characterized by a stress exponent m and a deformation parameter B. From the experimental observations reported in this paper, the following conclusions can be made:

- Rock particle assemblages are more compressible when humid or saturated compared to dry conditions;
- Well graded samples exhibit less vertical deformations compared to more uniform gradations;
- Exponent *m* decreases with increasing density;
- Parameter *B* decreases with increasing density and is strongly affected by the water state of the sample;
- Ratio C_{ε,t}/C_{σ,t} is constant and equal to about 0,12 for dry samples and 0,25 for humid and saturated samples of a granitic rock;
- The breakage index increases with increasing plastic work.

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