

Study and characterization of the roughness: the interfaces of gravity dams

Rousseau B., Ballivy B., Rivard P.

Rock Mechanics Laboratory, Université de Sherbrooke, 2500 University Bd, Sherbrooke, J1K 2R1, Quebec, Canada

Riss J., Marache A.

GHyMaC Laboratory, University of Bordeaux 1, Batiment B18, Avenue de la Faculté, 33405 Talence Cedex, France



Contact e-mail: Baptiste.Rousseau@USherbrooke.ca

ABSTRACT

This article deals with the shear behavior of concrete-rock fractures under low normal stress of two dams from Quebec. These dams are located in two different geological areas. Samples from these dams were sheared under constant normal load and at the same time, laser scans of the fractures surfaces were done with a laser profilometer before and after shear test. From these data, a geostatistical study of roughness parameters has been realised. The whole study underlines some common characteristics between samples from the two dams despite the differences of substratum (limestone and granite).

RÉSUMÉ

Cet article se propose d'étudier le comportement en cisaillement des discontinuités roche-béton de deux barrages poids en béton québécois. Pour cela, des essais de cisaillement sous faible contrainte normale ont été conduits. Ces essais ont été complétés par la numérisation, à l'aide d'un rugosimètre laser, des surfaces des discontinuités ante- et post-cisaillement ; à partir de ces numérisations, une étude géostatistique a été menée. Il en résulte la mise en évidence de certaines caractéristiques mécaniques communes aux interfaces d'un même barrage.

1 INTRODUCTION

The long term stability of concrete gravity dams with regards to hazards that had not been taken into account in their design and construction is a topical issue in Quebec. Indeed, the dams that are older than 50 years have not been built according to current standards, particularly regarding the seismic hazards (new standards have been applied since the 6.2-magnitude earthquake of at Saguenay in 1988). Models used to improve the stability of dams use some parameters such as the internal friction angle (ϕ), but actually, these parameters are given by indirect tests or tables of parameters. That is the reason why this study proposes a better evaluation of the intrinsic parameters of rock-concrete interfaces which are the most critical interface in gravity dams.

This study proposes to describe and analyse the behavior of rock-concrete interfaces under shear stress. The investigated discontinuities were collected with a 150 mm core drill from two gravity dams located in the Province of Quebec. The discontinuities, collected by coring, were then sheared by the direct shear device of the University of Sherbrooke (Quebec, Canada), under low normal stress which remained constant throughout the test. Before and after shearing, systematically scanning of the surface of the walls of each interface was performed with a laser profilometer.

A comparison between the results of mechanical tests and the results obtained from geostatistical processing of scans allows were used. This methodology was never used for the rock-concrete interfaces study and it allows to highlight classes of interfaces.

2 METHODOLOGY

2.1 Direct shear test

The concrete-rock discontinuities were obtained by coring two dams named in this paper Dam 1 and Dam 2. The concrete cast on the bedrock of the dam had mechanical and textural properties considered constant within a given dam. The samples were made *in situ* with a core sampler of 150 mm (6 inches) or 100 mm (4 inches). The selection of the sampling diameter depended of the coring place. Samples of the concrete-rock discontinuities were then sheared under constant normal stress on the direct shear test device made at the Sherbrooke University. This device (Figure 1) allows working with low levels of normal stresses as found in service dams.

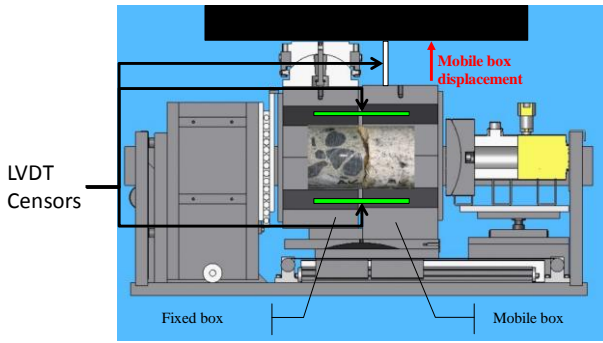


Figure 1: Direct shear device developed at Sherbrooke University

In order to calculate shear resistance parameters, three levels of normal stress σ_N were applied on samples during shear tests: 0.15, 0.30 or 0.45 MPa. The 0.30 MPa value is estimated to be the stress found at the rock-concrete interface, based on the dam's heights and the concrete density. Furthermore, the tangential speed of strain used was 0.15 mm/min and it has been maintained constant during the test. The maximum shear displacement was set up at 14 mm, according to Flamand (2000), Grasselli (2001), Marache (2002). This strain corresponds to nearly 10% of the diameter of the sheared interfaces (Brown, 1981).

During the tests, several parameters were recorded (in brackets is the error on the sensor measurements given by manufacturers): the normal force (250 N), the tangential force (25 N), the normal displacement (3 μm) and tangential displacement (75 μm).

Before each test, twice the normal stress used during the direct shear test was held on discontinuities for one minute, and then progressively decreased to reach the desired normal stress, in order to ensure a correct placement of the walls of the discontinuity (Gentier *et al.*, 2000). This part of shear test ensures a good fitting of the discontinuity.

2.2 Roughness measurements

Roughness is the relief of the two walls of a discontinuity. It was measured using a laser profilometer (Figure 2).

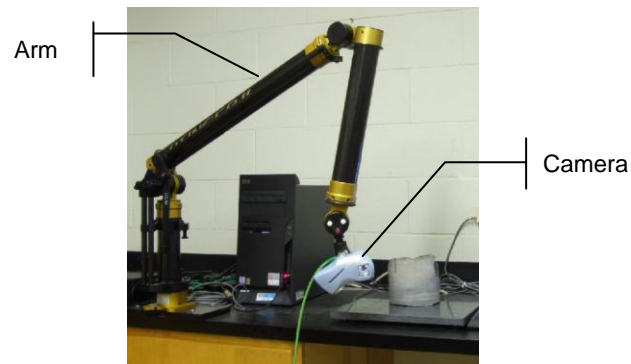


Figure 2: Photography of the laser profilometer

The laser profilometer used has a resolution of 80 μm in (x ; y) plan and 16 μm in the z axis. Once the scan of the fracture walls has been done, several roughness parameters were calculated and variographic study can be performed on the heights of the surface (section 4).

3 DIRECT SHEAR TESTS RESULTS

3.1 Presentation of Dam 1

Dam 1 is located in a geological province consisting of limestones from Cambro-Ordovician age. The limestone is gray, with crystal sizes ranging from micrometer to millimeter. Centimeter thick shale beds intersperse the limestone rock mass, they are very soft, will trot out easily and grow in a plane perpendicular to the core extension. Moreover, they are generally at the surface of discontinuities, but thick enough to infer that the topography of the walls of the fracture is the one of the underlying limestone.

To characterize the mechanical behavior of the rock-concrete interface of the dam, three tests were conducted on rock-concrete cores of 150 mm in diameter, two on unbounded discontinuities without initial cohesion that have been repositioned (1-2 and 1-3, σ_N respectively 0.50 and 0.17 MPa), and one on a bounded discontinuity with bridges of material between the upper and lower (1-1, $\sigma_N = 0.31$ MPa). The maximum tangential displacement obtained during the tests was 14 mm. Normal stresses were considered to be equal to the normal in situ stress applied at the level of the sample.

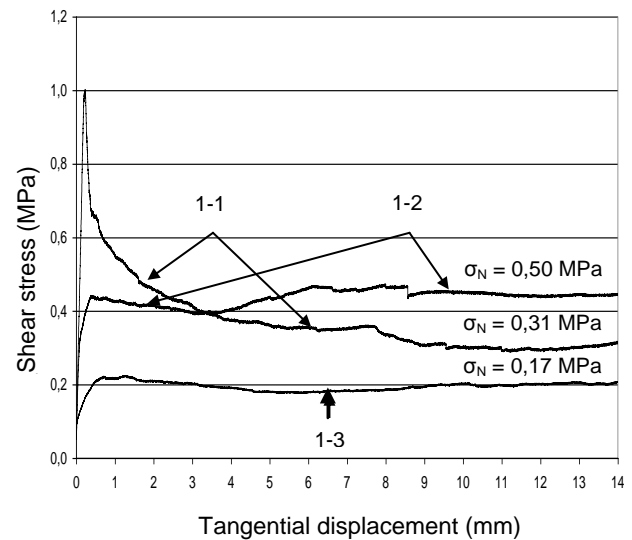


Figure 3: Curves of tangential displacement as a function of shear stress of concrete-rock contact Dam 1

Figure 3 shows the two shear curves obtained from unbounded samples prior to testing (1-2 and 1-3). No defined peak can be found; there is rather a plateau reached at a displacement of about 0.4 mm for 1-2 and of about 0.6 mm for 1-3 of. These values were obtained for shear stress of 0.42 MPa and 0.21 MPa, respectively. The value of the plateau is barely the same than that of normal stress.

The curve of the sample 1-2, shows a slight increase in shear stress after 4 mm of tangential displacement; it moves from 0.40 MPa to 0.48 MPa. This increase is observed only for this sample. It may correspond to the presence of an asperity in the fracture surface influencing the mechanical behavior for such a move (Lee *et al.*, 2001).

The curve of test 1-1 (normal stress of 0.31 MPa) has a well defined peak at 1 MPa, this stress being equal to the force required for breaking the bridges of material between the walls (Tsang & Witherspoon, 1981). Finally, these curves show an initial tangent modulus of 5 MPa/mm and an increase in the value of residual stress with increasing normal stress σ_N , which is typically observed in the case of rock-rock discontinuities (Patton, 1966).

Figure 4 shows the three dilation curves. A slight contraction (0.015 mm) identical in all three samples is found within the first 0.2 mm. This equal phase of contraction may come from the homogeneity of the rock and of the concrete that the interfaces are made of (Buzzi *et al.*, 2007).

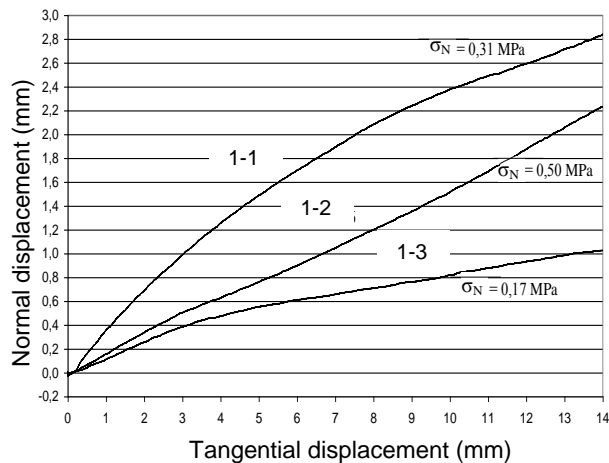


Figure 4: Dilatancy curves of tests on rock-concrete contacts Dam Hull 2.

The curve of the test performed on the sample 1-2 (Figure 4) has an initial slope increasing at 4 mm of tangential displacement: this is related to the increased shear stress for the same tangential displacement (Figure 3). Beyond 10.5 mm, the slope increases again, indicating the presence of an asperity on the surface topography of the sample. This asperity influences the shear behavior of the whole sample (Lopez, 2000).

Table 1: Synthesis of dilatancy parameters of Dam 1

Interface	Peak dilatancy (°)	Residual dilatancy (°)	Maximum dilatancy (mm)
1-1	21	6	2.8
1-2	10	7	2.2
1-3	8	3	1.0

The dilatancy curve of test 1-1, following the breakdown of a discontinuity with cohesion, has the highest dilatancy angle (dilatancy values are given in Table 1). Then comes the curve of the test 1-2 and finally the 1-3 test. The maximum dilatancy is highly variable from one sample to another; it is 1 mm for testing 1-3, 2.2 mm for 1-2 and 2.8 mm for 1-1. The maximum dilatancy angle is observed for the bounded specimen and the angles are quite smaller for both unbounded samples. This may be correlated with the general behavior of these three samples.

After describing the mechanical results of shear tests realized on samples from Dam 1, the results for Dam 2 are outlined below.

3.2 Presentation of Dam 2

Dam 2 is located in the region of Abitibi-Témiscamingue, in north-western Quebec, in the geologic province of Canadian Shield. Commonly referred as gneiss, the granitic rock composing the foundation presents a granular texture and shows a heterogeneous low marked foliation and quartz veinlets. It is composed of quartz, feldspar and some dark mica; the minerals are smaller than 1 mm

Six shear tests were performed (Table 2), four samples of 150 mm in diameter (numbered 1 to 4) and two samples of 100 mm in diameter (numbered 5 and 6) were tested. All samples were unbounded (no cohesion).

The maximum tangential displacement reached for the 150 mm samples was 14 mm and it was 10 mm for the 100 mm samples. However, during the test on sample 4, a tangential displacement of only 10.8 mm was reached, because of a tangential displacement sensor problem.

Table 2: Normal stresses (σ_N) applied during testing

Sample	Diameter (mm)	Normal stress (MPa)
1	150 mm	0.33 MPa
2	150 mm	0.36 MPa
3	150 mm	0.50 MPa
4	150 mm	0.52 MPa
5	100 mm	0.42 MPa
6	100 mm	0.22 MPa

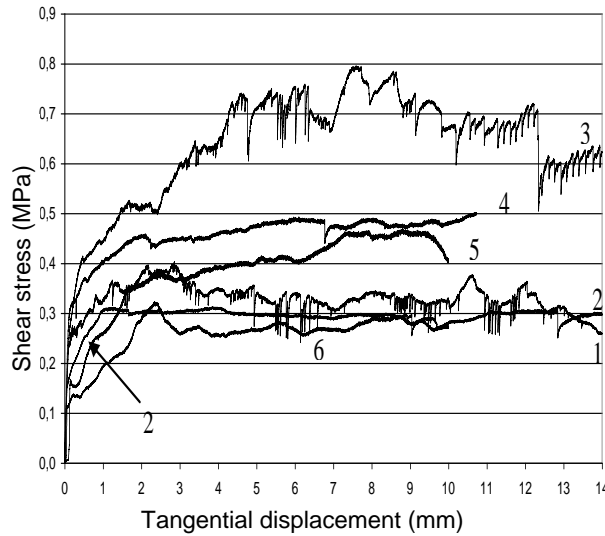


Figure 5: Curve of shear stress vs. Tangential displacement for rock-concrete contacts of Dam 2

The shear curves do not show any significant peak and present a similar initial behavior (Figure 5). Samples 1 and 2 show a residual strain of about the same value (0.30 and 0.33 MPa, respectively). The different tangential displacements (3.7 and 1 mm) can be linked with variations of roughness between the two samples. The tangential residual stress of sample 4 is 0.47 MPa for sample 3, it is more difficult to estimate and it is about 0.7 MPa. Although normal stresses are almost identical (0.50 and 0.52 MPa), the stress at the peak of sample 3 is higher than that of sample 4 due to a greater roughness. As for the curves of samples 5 and 6, they are, as expected, due to normal stresses used. Sudden decay of the shear stress occurs on the curve of test 2 at a tangential displacement of 9 and 13 mm, like for sample 4 with a 7 mm displacement (spikes in test 1 and 3 are associated with sensor problem). Finally, like the results of Dam 1, five of the six curves show a decrease of residual stress with decreasing normal stress.

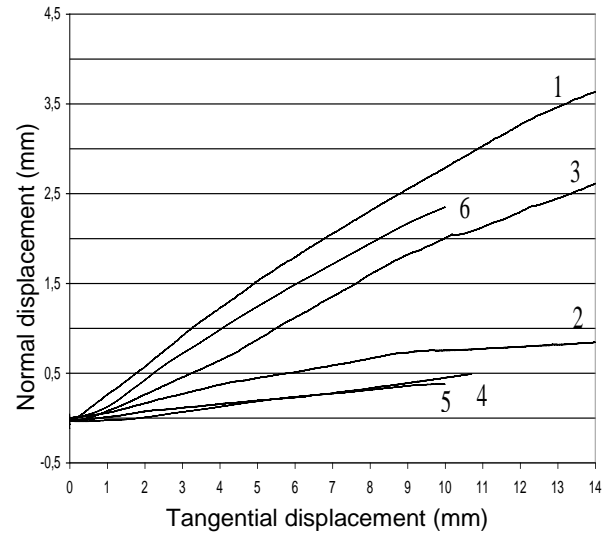


Figure 6: Dilatancy curve of the tests on rock-concrete contacts for Dam 2

Dilatancy curves (Figure 6) show a very heterogeneous behavior of concrete-rock discontinuities. The dilatancy angles in the first millimeter of displacement lie between 17° and 3° and those corresponding to the residual stress lie between 10° and 2° . The observed maximum dilatancy was 3.6 mm and the minimum dilation of 0.4 mm. It may be inferred that there is no general behavior for the rock-concrete discontinuities of this dam. As stated above, the rock is quite heterogeneous due to the slight foliation found in the rock mass. In such rock type, the locations of the discontinuities yield different behaviors with regards to shear testing.

We have seen the results from mechanical tests realized on Dam 2, we have previously seen from those tests of Dam 1; now look if some commonalities can emerge.

3.3 Comparison of the results for both dams

The curve of the test being apart 1-1 (single sample with a cohesion), we see that concrete-rock tests on the two dams do not present well defined peak shear. The shear behavior of treated samples is reached for values more or less important of tangential displacement. The shear stress remains constant or almost constant throughout the relative movement of the walls. This behavior can be explained by the low surface roughness used, that is to say, they present only bumps that form a small angle to the mean plane of discontinuity, and by low values of normal stresses applied (Jafari et al., 2003).

Dilatancy curves spread over a larger range of values for tests conducted on samples from Dam 2 as those from Dam 1 (0.4 to 3.6 mm for the former and a 2.8 mm for the second). The dilatancy is strongly influenced by the angle formed by the rough with the mean plane of discontinuity (*ie* colatitude; Verreault, 2000), so there is greater

diversity in interfaces colatitude of Dam 2 than in those of Dam 1 that is to say samples from Dam 2 show a drift topographic indeed this was observed during their development in the shear box for samples 3 and 1, and paradoxically not by 6, as we might have. Samples 2, 4 and 5 of Dam 2, meanwhile, seem at once less rough and without drift. All these assumptions have yet to be verified by calculating various statistical parameters of roughness characteristics and by the geostatistical treatment scans.

4 STUDY OF SPATIAL CORRELATIONS

The variographic study of heights z allows to highlight the spatial correlation distances and thus to understand the overall spatial structure (Chilès and Delfiner, 1999). Equation 1 is used for calculating the experimental variogram:

$$\gamma_e(\mathbf{h}) = \frac{1}{2N_h} \sum_{x_j - x_i \approx \mathbf{h}} [z(x_j) - z(x_i)]^2 \quad [1]$$

Where: h is the distance between two measurement points x_i and x_j
 N is the number of pairs of points separated by h

Statistically, the variogram have meaning only if it is calculated on the half-diameter of the sample. That is the reason why, in the following, h varies from 0 to 6.5 cm in increments of 0.5 mm for samples of 150 mm and varies from 0-5 cm in increments of 0.5 mm for 100 mm. In addition, a 5° angular tolerance and a tolerance of 1 mm side were used to calculate variograms.

The variograms were calculated on the heights of the walls fixed (F = rock wall) and mobile (M = concrete wall) for each sample from Dam 2, and this, in the shear direction (Figure 7) help to clarify the behavior of concrete-rock interfaces. If we look at the dilatancy curves from runs 1 and 2 (Figure 6), we note that for a normal stress of the same order of magnitude, their behavior in terms of dilatancy is different. If we now look at the variogram from the digitization of the walls of these samples, those of test 2 show a variance of heights lower than that of test 1. It is the same for samples 3 and 4: sample 4 is less rough than 3, which explains why the dilatancy is lower for the test 4. Moreover, we note that some initial variograms have common behaviors (3M and 3F, 5F, 5M, 1M and 4F, 1F and 6F), indicating that changes in the heights, the distances h , where the curves are indistinguishable, are of the same order of magnitude. For samples 3 and 5, this is a sign of a roughness similar on both walls (Grasselli *et al.*, 2002). In addition, higher-order structures in the size of the sheared samples can be identified and must be taken into account in an overall analysis of the behavior of dams (0.03 m (6M), 0.04 m (6F) and 0.06 m (1F)).

Furthermore, some curves have a hyperbolic behavior which is a clue that the samples present a topographic drift that hide the structures the variogram could underline. So, it will be interesting to compute the

variogram results not on the heights, z , but on the residual values of these heights to avoid the influence of the topographic drift.

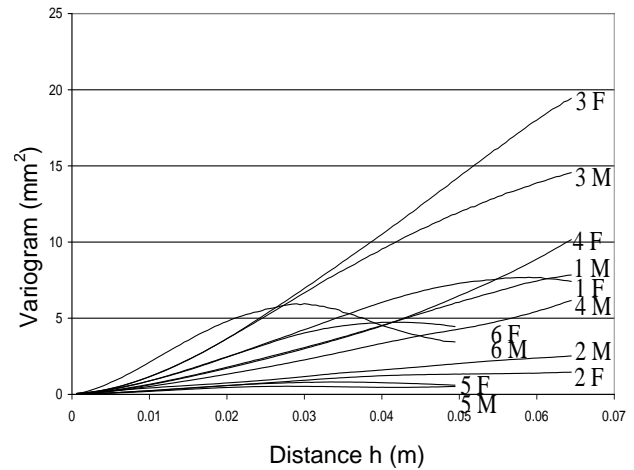


Figure 7: Variograms heights of rock-concrete contacts Dam Rapide7.

5 CONCLUSION

This study has highlighted that for the same dam, the shear behavior of concrete-rock discontinuities is in general comparable. For Dam 1, we have only three samples, but the mechanical behavior of them is similar or quite similar. But for Dam 2, we have seen that the heterogeneity of the rock produces some disturbances in the general behavior of the discontinuities. This is underlined by the variographic study of the heights of the walls of these samples. There is no asperity which is common to all these interfaces. To complete this study two ways are possible: the first supposes the calculation of the variograms on the residual values of the heights to avoid the topographic drift influence. The second way consists to calculate the roughness parameters on each part of the interface used in this paper. In addition, rock-concrete interfaces are just one type of discontinuities that could lead to the ruin of the dam. It remains to characterize the concrete-concrete interfaces (joints casting) and rock-rock (stratification below the dam) of the two dams. In the longer term, all these study will help to model the shear behavior of the overall dam.

ACKNOWLEDGEMENTS

The authors would like to thanks Geroges Lalonde for his help for conducting the direct shear tests and Clermont Gravel for the scientific advices. Financial support has been provided by the Natural Science and Engineering Research Council of Canada and by Hydro-Quebec. Cores were supplied by Hdro-Quebec.

REFERENCES

- Brown E.T. 1981. Rock characterization, testing and monitoring. *ISRM suggested methods*, Pergamon Press.
- Buzzi O., Hans J., Boulon M., Deleruyelle F., Besnus F. 2007. Hydromechanical study of rock-mortar interfaces. *Physics and Chemistry of Earth*, 32 pp 820-831
- Chiles J.P., Delfiner P. 1999. Geostatistics, Modelling spatial uncertainty. *Wiley Series in Probability*. John Wiley and Sons, Inc. New York., p 695.
- Flamand R. 2000. Validation d'une loi de comportement mécanique pour les fractures rocheuses en cisaillement. *Thèse de doctorat, université du Québec à Chicoutimi (Canada)*, 470 p.
- Gentier S., Riss J., Archambault R., Flamand R., Hopkins D. 2000. Influence of fracture geometry on the shear behavior. *International Journal of Rock Mechanics and Mining Sciences & Geomechanics Abstracts*, 37, pp. 161-174.
- Grasselli G. 2001. Shear strength of rock joints based on quantified surface description. *Thèse de l'École Polytechnique Fédérale de Lausanne*, 124 pages.
- Grasselli G., Wirth J., Egger P. 2002. Quantitative three-dimensional description of a rough surface and parameter evolution with shearing. *International Journal of Rock Mechanics and Mining Sciences*, Volume 39, Issue 6, pp. 789-800
- Jafari M.K., Amini Hosseini K., Pellet F., Boulon M., Buzzi O. 2003. Evaluation of shear strength of rock joints subjected to cyclic loading. *Soil Dynamics and Earthquake Engineering* 23 pp.619–630
- Marache A. 2002. Comportement mécanique d'une fracture rocheuse sous contraintes normale et tangentielle. *Thèse pour l'obtention du grade de docteur, École Centrale des Arts et Manufacture, Paris*, 364p.
- Lee H.S., Park Y.J., Cho T.F., You K.H. 2001. Influence of asperity degradation in mechanical behavior of rough rock joints under cycling shear loading. *International Journal of Rock Mechanics & Mining Sciences* 38 pp. 967–980
- Lopez P. 2000. Comportement mécanique d'une fracture en cisaillement : analyse par plan d'expériences des données mécaniques et morphologiques connues sur une fracture. *Thèse de l'Université Bordeaux 1 et Ph.D. Thesis de l'Université du Québec à Chicoutimi*, 401p.
- Patton F.D. 1966. Multiple modes of shear failure in rock. *1er Congrès de la Société Internationale de Mécanique des Roches*, Lisbonne, pp. 509-513.
- Tsang Y.W. & Witherspoon P.A. 1981. Hydromechanical behaviour of a deformable rock fracture subject to normal stress. *Journal of Geophysical Research*, vol. 29 (7) pp. 9297-9298
- Verreault N. 2000. Elaboration d'une méthode de caractérisation de la dilatance d'une fracture rugueuse en fonction de son déplacement en cisaillement – *Mémoire de maîtrise de l'Université du Québec à Chicoutimi*, 196p