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1 **Integration of rock joint roughness into the Mohr-Coulomb shear**  
2 **behaviour model – Application to dam safety analysis**

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9

## 10 **ABSTRACT**

11           Based on a previous experimental study that we conducted on granite joint replicas  
12           including thirty direct shear tests, this article proposes to integrate the joint  
13           roughness in the Mohr-Coulomb shear behaviour model (MC). The model  
14           integrates joint roughness into constitutive stress-displacement relationships  
15           describing mechanical behaviour of rock joint. The shear strength of rock joint is  
16           assessed by the MC shear criterion that takes into account the apparent cohesion.  
17           This MC model integrating rock joint roughness component is validated against  
18           other experimental results from the literature. It is able to accurately predict the  
19           peak shear strength of an unbounded rock joint with an average relative error of  
20           7.9%. It is finally used to assess the role of joint roughness on the shear behaviour  
21           of a gravity dam foundation.

22

## 23 **KEYWORDS**

24           Apparent cohesion, DEM, Mohr-Coulomb model, rock joint, roughness, shear  
25           behaviour

26

27           **Introduction**

28   The shear strength of unbounded (cohesionless) rock joints have been extensively studied  
29   since the 1950s. For such a joint, it is widely accepted that shear strength and behaviour  
30   are controlled by many parameters such as joint roughness (Patton 1966; Barton and  
31   Choubey 1977; Kulatilake et al. 1995), joint interlocking (Zhao 1997a, 1997b), rock  
32   weathering (Barton and Choubey 1977; Nouailletas et al. 2017), joint mechanical  
33   properties (Ghazvinian et al. 2010; Rullière et al. 2021) and scale (Bandis et al. 1981;  
34   Tatone and Grasselli 2013; Buzzi and Casagrande 2018).

35           Several failure criteria have been developed to predict joint peak shear strength.  
36   The simplest one is the linear Mohr-Coulomb criterion that links the joint peak shear  
37   strength ( $\tau_{\text{peak}}$ ) to the normal load ( $\sigma_N$ ) and joint surface characteristics ( $C$  and  $\phi$ ). Barton  
38   and Choubey (1977) extended the joint surface characteristic parameter  $\phi$  of the Mohr-  
39   Coulomb criterion to take into account the effects of joint roughness and joint weathering  
40   on peak shear strength. Barton-Choubey criterion is well known in rock mechanics  
41   (ISRM 2014). This criterion introduces the joint roughness parameter (JRC) determined  
42   by visual comparison of the joint with a two-dimensional abacus. Because this abacus  
43   cannot cover the wide spectrum of roughness that a joint can exhibit, some authors  
44   consider that the JRC value might be prone to subjectivity (Hsiung et al. 1993; Beer,  
45   Stead, and Coggan 2002; D. Sow et al. 2016; D. Sow et al. 2017).

46           Later, Grasselli and Egger (2003) proposed a more complex criterion which  
47   adopts a three-dimensional view of the rock joint to estimate its peak shear strength. The  
48   criterion proposed by Grasselli and Egger (2003) considers that only the asperities facing  
49   the shear direction contribute to joint peak shear strength. Nevertheless, it was  
50   demonstrated that this criterion tends to overestimate the peak shear strength of smooth  
51   joints ((Xia et al. 2014). More recently, Zhang et al. (2016) proposed a 2D criterion that  
52   considers only a proportion of the asperities facing the shear direction. Wang and Lin

53 (2018) proposed a shear criterion that describes either the sliding or shearing mechanism  
54 effects on peak shear strength as a function of the normal stress. Others relevant  
55 references in the field of our work have to be cited also (Chen et al. 2022; Wang et al.  
56 2022; Zhang et al. 2022a; Zhan et al 2022b; Barton and Shen 2017; Wang et al. 2020).

57 Besides assessing the shear strength of joints, it is necessary to evaluate their pre-  
58 failure shear behaviour. To this end, several shear behaviour models that analytically  
59 describe the interactions between stresses and relative displacements of a joint subjected  
60 to shearing have been developed. Historically, the work of Goodman, Taylor, and Brekke  
61 (1968) is considered as the first development of numerical laws to model the shear  
62 behaviour of rock joints. It suggests that stresses are connected to displacements through  
63 mathematical linear elastic laws using constant stiffness, the shear stress being bounded  
64 by a maximum value that can be estimated using one of the criteria mentioned above.  
65 Later, considerable research improved this model either by considering that stiffness is  
66 not systematically constant during shearing (Bandis et al. 1983) or by taking into account  
67 the effect of joint damage (Cundall and Lemos 1988). The continuous improvement of  
68 computers has led to the development of even more complex and comprehensive shear  
69 behaviour models in recent decades. Duriez et al. (2011) developed a shear behaviour  
70 model with incrementally non-linear constitutive relations to describe the mechanical  
71 behaviour of infilled rock joints along a variety of loading paths. Oh et al. (2015) and Li  
72 et al. (2016) proposed a model that considers together and independently the contributions  
73 of large-scale roughness (waviness) and small-scale roughness (unevenness) of a joint to  
74 its shear behaviour.

75 For engineering applications such as rock slope stability and dam foundation  
76 design, many engineering guidelines suggest using the Mohr-Coulomb criterion to assess  
77 the shear strength of a rock joint (USBR 1987; US Army Corps of Engineers 1995; CFBR

78 2012; ISRM 2014; Federal Energy Regulatory Commission 2016). Mohr-Coulomb input  
79 parameters, cohesion  $C$  and friction angle  $\phi$ , are determined through experimental shear  
80 tests on small joint samples. For an unbounded rock joint, the cohesion obtained  
81 corresponds to the apparent cohesion concept, which depicts the roughness and joint  
82 interlocking effect on the shear strength (Amitrano and Schmittbuhl 2002; Rullière et al.  
83 2020). However, guidelines recommend a safe practice considering a low or null value of  
84 apparent cohesion, since its extrapolation to the scale of the structure is delicate. But this  
85 safe practice is conservative since the resistance characteristics of the joint due to its  
86 morphology (roughness, interlocking) are neglected.

87         The aim of this article is to integrate rock joint roughness components into the  
88 Mohr-Coulomb shear behaviour model which is chosen for its wide usage in engineering  
89 practice strength (USBR 1987; US Army Corps of Engineers 1995; CFBR 2012; ISRM  
90 2014; Federal Energy Regulatory Commission 2016), in order to take into account the  
91 role of joint roughness on joint shear behaviour. This integration aims to take into account  
92 the role of joint roughness on joint shear behaviour and is built up on the base of thirty-  
93 five experimental tests (direct joint shear tests and joint compression tests) and links joint  
94 roughness to joint shear behaviour and strength. The rock joint shear behaviour is  
95 described through stress-displacement laws. Joint failure state is expressed by the Mohr-  
96 Coulomb criterion including the apparent cohesion concept, to stick to the current practice  
97 of rock engineering. At the end, the MC shear behaviour model integrating rock joint  
98 roughness is validated with experimental data coming from rock mechanics literature.

99         The paper is divided into four parts. The first part presents the materials and  
100 methods (Part 1). In the second part, the MC shear behaviour model integrating rock joint  
101 roughness is developed and calibrated (Part 2). It consists in the development of empirical  
102 laws correlating the joint roughness to the shear behaviour model input parameters. These

103 empirical laws are then integrated in the MC model, so that it takes into account the role  
104 of joint roughness on joint shear behaviour. In the third part, the developed model is  
105 validated in the third part: model blind-predictions are faced to direct shear test results  
106 from rock mechanics literature in order to evaluate the model's accuracy (Part 3). In the  
107 last part, the MC shear behaviour model integrating rock joint roughness is used to  
108 demonstrate the key role of joint roughness on the shear behaviour of a gravity dam  
109 foundation (Part 4). All the numerical calculations presented in this paper were performed  
110 using the Discrete Element Method (DEM).

## 111 **1. Materials and Methods**

112 This part presents: i) the experimental data used to develop the empirical equations  
113 linking the joint roughness to the shear behaviour model input parameters, ii) the shear  
114 behaviour model used in the study, and iii) the DEM code in which the model will be  
115 implemented to predict the shear behaviour at the laboratory and gravity dam scales.

### 116 ***1.1. Experimental data used to develop the shear behaviour model***

117 The experimental data come from a previous work (Rullière et al. 2020) and are  
118 summarized in this article. Thirty direct shear tests on granite joint replicas were carried  
119 out in CNL conditions and under 0.1, 0.2, 0.35 and 0.6 MPa normal stress. Additional  
120 joint compression tests were conducted to characterize the joint normal stiffness. The tests  
121 were specifically performed for the research work presented in this paper.

122 It is specified in this introduction to the experimental data that our work does not  
123 take into account the scale effect between the sample tested in the laboratory and the real  
124 scale model. Several authors such as (Bandis et al. 1981) have worked on the scale effect  
125 and have shown the influence of sample size on shear strength. This question of scale

126 effect between laboratory and field is, in general, a universal problem of this kind of study  
127 that our research has not taken into account.

### 128 *1.1.1. Joint roughness statistical indicator ( $Z_2$ )*

129 Many studies have been conducted to define the roughness concept because it is  
130 considered to be the main parameter governing the shear behavior of rock discontinuities.  
131 Barton and Choubey (1977) developed the joint roughness coefficient (JRC) to estimate  
132 joint roughness using a visual comparison of 10 standard profiles. With the advancement  
133 of non-contact methodologies used to scan rock joint surfaces, new parameters have been  
134 developed to assess roughness. In this context, the 2D directional parameter  $Z_2$  is widely  
135 accepted and used in the rock mechanics community as a non-subjective roughness  
136 indicator (Tse and Cruden 1979; Kulatilake et al. 1995; Tatone and Grasselli 2010;  
137 Magsipoc, Zhao, and Grasselli 2019; Ram and Basu 2019). Indeed, for a joint profile, the  
138 parameter  $Z_2$  is a 2D directional parameters that describes the local topographic slope of  
139 the joint profile.  $Z_2$  can be seen as a topographical slope and corresponds to the root-  
140 mean-square of the first derivative (Myers 1962).  $Z_2$  can be calculated as:

$$141 \quad Z_2 = \left[ \frac{1}{L} \sum_1^{n-1} \left( \frac{y_{i+1} - y_i}{x_{i+1} - x_i} \right)^2 \right]^{\frac{1}{2}} \quad (1)$$

142 Where  $L$  is the length of the joint profile,  $x_i$  and  $y_i$  are the coordinates of the  
143 discretized joint profile (0.5 mm sampling interval).

144 The calculation of  $Z_2$  requires only a topographic scan of the joint surface and, in  
145 comparison to other roughness indicators, it is rather simple and non-time-consuming to  
146 implement the calculation of  $Z_2$  in an algorithm.

147 Five granite joints (J0, J1, J2, J3, J4) with different roughness's ( $Z_2$  from 0 to  
148 0.373) were selected to produce the replicas. Prior to replica production, the granite joints  
149 were scanned with a non-contact laser profilometer (Figure 1). Data from the scans were



150 then implemented in a specifically designed algorithm linking the surface morphology to  
151 the roughness statistical parameter  $Z_2$  (Rullière et al. 2020). Table 1 shows the  $Z_2$  value  
152 for each of the five granite joints used to produce the replicas.

153 [FIGURE 1 HERE]

154 [TABLE 1 HERE]

### 155 1.1.2. Direct shear test

156 Following the production of the replica (six replicas per selected roughness), thirty direct  
157 shear tests were conducted in CNL conditions. Twenty direct shear test results, carried  
158 out on J0, J1, J2, J3 and J4 under 0.1, 0.2, 0.35 and 0.6-MPa normal stress come from the  
159 results of Rullière et al. (2020). For this work, ten additional direct shear tests were carried  
160 out for J0, J1, J2, J3 and J4, under 0.1 and 0.6-MPa. These new results provide more shear  
161 tests results over the 0.1 to 0.6-MPa interval and strengthen the Mohr-Coulomb failure  
162 envelope plot. The normal stress levels used in this study correspond to those observed  
163 in dam foundations or other civil engineering works (Sow et al. 2017). Each replica was  
164 used for a single direct shear test.

165 Figure 2 shows the shear strength versus shear displacement curves for all the  
166 direct shear tests used in this study. The key data are summarized in Table 2. It classically  
167 appears from the direct shear tests results that the rougher the joint, the higher the shear  
168 strength (peak or residual).

169 [FIGURE 2 HERE]

170 [TABLE 2 HERE]

171 Figure 3 depicts the Mohr-Coulomb failure envelopes plotted from the  
172 experimental direct shear test results for the peak and residual shear strengths. Apparent  
173 cohesion and friction angle values at both peak and residual stages are shown in Table 1.  
174 Again, it appears that the rougher the joint, the higher the apparent cohesion and friction

175 angle values. Since the results were obtained on an unbounded rock joint replica, it was  
176 considered that the apparent cohesion could be mobilized at peak stage only (EPRI 1990;  
177 Rullière et al. 2020). Therefore,  $C_{APP \text{ Residual}} = 0\text{-kPa}$ .

178 [FIGURE 3 HERE]

179 The shear stiffness  $K_s$  was calculated for shear stresses ranging from 25 to 75%  
180 of the peak shear strength (Kumar and Verma 2016). This precaution allows setting aside  
181 the curved areas of the shear strength-shear displacement curve (initial loading and peak  
182 phase). Table 1 presents the  $K_s$  mean value obtained for all the joints tested. It appears  
183 that the rougher the joint, the higher the shear stiffness.

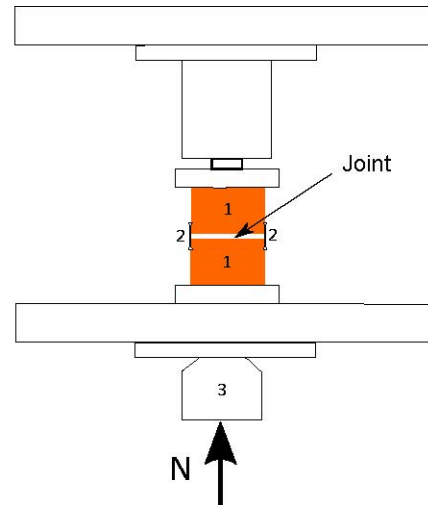
184 Figure 4 shows normal versus shear displacement curves for all the direct shear  
185 tests (Rullière et al. 2020). It is verified that the dilatant behaviour depends on both of the  
186 joint roughness and the normal stress imposed during the shear tests. As shown in Figure  
187 4, a rougher joint lead to larger normal displacement during a direct shear test in CNL  
188 conditions. The dilatancy angle ( $dN$ ) is calibrated from the maximal value of the  $\Delta v / \Delta u$   
189 ratio (usually around the peak shear displacement value). Table 1 shows the  $dN$  mean  
190 value obtained for all the joints tested. As expected, the rougher the joint, the higher the  
191 dilatancy angle.

192 [FIGURE 4 HERE]

### 193 1.1.3. Compression test results

194 Joint normal stiffness ( $K_N$ ) assessment requires specific experimental tests called joint  
195 compression test (Bandis et al. 1981; Fan, Cao, and Tang 2018). Such a test involves  
196 recording the relative normal displacements of a rock joint under a given normal load  
197 (Figure 5).

198 [FIGURE 5 HERE]



199

200 In practice, several loading-unloading cycles are performed to determine the  
201 normal stiffness of a rock joint (Bandis et al. 1983; Marache 2002). The first cycle allows  
202 the joint to be fully interlocked, so that the deformations recorded during the following  
203 cycles correspond to the joint's deformations only and not to a change of joint  
204 interlocking.

205 Five joint compression tests were performed, for each granite joint. The joints  
206 were subjected to an increasing normal stress, up to 30% of the compressive strength  
207 matrix of the material, prior to a progressive discharge. The charge-discharge cycles were  
208 repeated three times and the  $K_N$  was measured along the last loading cycle. Figure 6  
209 shows the normal stress versus normal displacement curves obtained from the joint  
210 compression tests. The  $K_N$  value of each joint is presented in Table 1. If the  $K_N$  values  
211 are compared to joint roughness, it seems that the rougher a joint, the higher the  $K_N$ .

212

[FIGURE 6 HERE]

### 213 **1.2. Shear behaviour model used in the study**

214 The shear behaviour model used in this study follows an elastic-plastic formulation based  
215 on the work of Goodman, Taylor, and Brekke (1968). As a joint is subjected to a shear  
216 stress, the response of the joint is initially exclusively elastic. The displacements can be

217 correlated to the normal or shear stress variations through constant joint normal or shear  
218 stiffness and classical linear elastic laws:

$$219 \quad \begin{pmatrix} d\sigma_N \\ d\tau \end{pmatrix} = \begin{pmatrix} K_N & 0 \\ 0 & K_S \end{pmatrix} \cdot \begin{pmatrix} d\Delta_V \\ d\Delta_U \end{pmatrix} \quad (2)$$

220 Where  $d\sigma_N$  is the normal stress variations,  $d\tau$  is the shear stress variation,  $K_N$  is the joint  
221 normal stiffness,  $K_S$  is the joint shear stiffness,  $d\Delta_U$  is the shear displacement increment  
222 and  $d\Delta_V$  is the normal displacement increment.

223 A direct shear test conducted in Constant Normal Load (CNL) conditions is  
224 piloted by constant tangential displacement increments ( $d\Delta_U$ ). Therefore, the shear stress  
225 progressively increases according to (2). On the contrary, the normal stress increment is  
226 null. In this type of test, this means that the model assumes no normal displacement  
227 increments ( $d\Delta_V$ ) during the elastic stage.

228 The elastic stage extends until the shear stress  $\tau$  reaches the shear strength value  
229  $\tau_{\text{peak}}$  defined by the Mohr-Coulomb criterion which serves as both a (constant) yield  
230 surface and plastic limit condition in the  $(\tau; \sigma_N)$  plane, in the absence of elasto-plastic  
231 hardening. As rock joints are unbounded, the cohesion of the Mohr-Coulomb criterion  
232 corresponds to the apparent cohesion,  $C_{\text{app}}$ , as presented in the introduction.

$$233 \quad \tau = C_{\text{app}} + \sigma_N \cdot \tan(\varphi) \quad (3)$$

234 Where  $\tau$  is the shear stress,  $\sigma_N$  is the normal stress,  $C_{\text{app}}$  is the apparent cohesion and  $\varphi$  is  
235 the friction angle.

236 As soon as the peak shear stress is reached, the shear behaviour model considers  
237 that the joint is in a failure phase: the elastic stage is completed and the plastic stage  
238 begins. The shear stress decreases instantaneously to the residual and constant value  
239  $\tau_{\text{residual}}$ . This choice of a brittle behaviour, as opposed to a gradual softening, is chosen for  
240 simplicity, in line with engineering practices.

241 In the plastic stage, a flow rule governs the dilatancy whereby normal  
242 displacement variations are correlated to shear displacement variations and to the joint  
243 dilatancy angle:

$$244 \quad d\Delta_v = d\Delta_u \cdot \tan(dN) \quad (4)$$

245 Where  $d\Delta_u$  is the shear displacement increment,  $d\Delta_v$  is the normal displacement  
246 increment and  $dN$  the dilatancy angle.

### 247 *1.3. The Discrete Element Method*

248 A rock mass can be considered as a set of rocky blocks separated from each other by  
249 joints. For such a set, a mechanical stress will lead to displacements that are almost  
250 exclusively due to the displacements along the joints (the rocky block deformations are  
251 considered as insignificant). This consideration exactly matches the DEM hypothesis:  
252 blocks are considered as infinitely stiff and the displacements are exclusively due to block  
253 contacts.

254 The DEM method and UDEC software (Itasca 2019) were used in this paper to  
255 perform the different numerical calculations, either to simulate laboratory direct shear  
256 tests in CNL conditions or to assess the shear behaviour of a gravity dam founded on a  
257 complex jointed rock mass. In UDEC, the hydro-mechanical behaviour of a set of blocks  
258 and joints is simulated by adjusting the joint properties (mechanical properties and  
259 behaviour models).

260 **2. Calibration of the MC shear behaviour model integrating rock joint**  
261 **roughness**

262 ***2.1. Equations linking joint roughness to shear behaviour model input***  
263 ***parameters***

264 The aim of this section is to develop empirical equations that correlate the joint roughness  
265 to the input parameters of the MC shear behaviour model integrating rock joint roughness.  
266 These input parameters are  $C_{app\ peak}$ ,  $\phi_{peak}$ ,  $C_{app\ residual}$ ,  $\phi_{residual}$ ,  $dN$ ,  $K_N$  and  $K_S$  (see section  
267 1 for more details). The experimental data presented in Part 1 are used and three kinds of  
268 mathematical laws are studied: linear, power, and exponential laws. For each kind of law,  
269 the least square method was used to determine coefficients  $a$ ,  $b$  and  $c$  :

270 
$$P = a \cdot Z_2 + b \quad (5)$$

271 
$$P = a \cdot Z_2^b + c \quad (6)$$

272 
$$P = a \cdot e^{Z_2 \cdot x} + c \quad (7)$$

273

274 Where  $a$ ,  $b$  and  $c$  are empirical coefficients and  $P$  stands for any model parameters  
275 among  $C_{app\ peak}$ ,  $\phi_{peak}$ ,  $C_{app\ residual}$ ,  $\phi_{residual}$ ,  $dN$ ,  $K_N$  and  $K_S$ .

276 For the sake of concision, only the power equations are presented. Indeed, it  
277 appeared during the study that the power equations (6) were able to correlate the joint  
278 roughness to the shear behaviour input parameter with great precision. In comparison to  
279 linear and exponential equations, the power equations had the highest coefficient of  
280 determination ( $R^2$ ). However, the reader will find linear and exponential equations in  
281 Appendix A.

282 Based on the data presented in Table 1, the power law equations describing the  
283 shear behaviour and shear failure state of the joint are developed. Power law coefficients

284  $a$ ,  $b$  and  $c$ , which correlate the roughness to the peak apparent cohesion, peak friction  
285 angle, residual friction angle, dilatancy angle, and normal and shear stiffness, are  
286 presented in Table 3. Figures 7 to 9 compare the power law predictions to the  
287 experimental data.

288 The values of coefficients  $a$ ,  $b$  and  $c$  and the equations developed in this section  
289 are valid only for the following conditions:

- 290 • the rock joint roughness is described by the  $Z_2$  statistical parameter.  $Z_2$  ranges  
291 between 0 (smooth joint) and 0.373 (rough joint);
- 292 • the rock joint is subjected to a normal load ranging from 0.1 to 0.6-MPa.

293 [TABLE 3 HERE]

294 [FIGURE 7 HERE]

295 [FIGURE 8 HERE]

296 [FIGURE 9 HERE]

## 297 *2.2.Numerical direct shear tests using the MC shear behaviour model*

298 Once the shear behaviour model was established, thirteen numerical direct shear tests  
299 were performed under the exact same conditions of normal load and roughness as the  
300 experimental direct shear tests presented in the previous section. In other words, data  
301 provided by the numerical shear behaviour model were compared to the experimental  
302 data used to develop it, to control if the numerical results fit the experimental observations  
303 and if the calibration of the numerical model had been carried out well.

304 Using UDEC software (Itasca 2019), the rock joint is represented by the contact  
305 between two infinitely rigid blocks. While the lower block is fixed, the upper block is  
306 subjected to both a normal load and a constant horizontal displacement rate. In the DEM  
307 model, the lower block length is longer than the upper block to avoid any toppling of the

308 upper block. For the numerical direct shear tests, the normal load and horizontal  
309 displacement rate were set as equal to the experimental values applied during the  
310 experimental direct shear tests (normal load ranges from 0.1 to 0.6-MPa and the  
311 horizontal displacement rate is 0.1 mm/min to insure quasi-staticity). The numerical  
312 model also required the roughness of the joint (depicted by  $Z_2$ ).

313 Figure 10 and Figure 11 show the shear stress versus horizontal displacement  
314 curves and normal displacement versus horizontal displacement curves. The dotted lines  
315 correspond to the experimental data, while the full lines correspond to the numerical  
316 results.

317 From Figures 10 and 11, it appears that despite its bilinear aspect, the numerical  
318 model manages to describe the rock joint shear behaviour efficiently. This is particularly  
319 true during the peak phase, and to a lesser extent for the residual phase (end of test).  
320 Nonetheless, as shown in Figure 10, the post-peak joint behaviour is not realistically  
321 described by the shear behaviour model since as soon as the peak shear strength is  
322 reached, the release of the shear stress is instantaneous, while the experimental tests show  
323 a progressive decrease of the shear stress towards a residual shear stress value. Figure 11  
324 shows that the shear behaviour model satisfactorily reproduces the dilatant behaviour of  
325 a rock joint, although there is no contraction stage and for the shear behaviour model, the  
326 dilatancy starts as soon as the shear stress reaches its peak (these are related to the  
327 equations that govern the MC shear behaviour model, see Part 1). However, it should be  
328 noted that in rock-mechanics engineering practice, the post-peak shear behaviour has not  
329 been extensively studied and the MC model approximations could be sufficient for a vast  
330 majority of applications.

331 Figure 12 displays the model peak and residual shear strength and contrasts it with  
332 the experimental recorded values. For the peak stage, the results predicted by the model



333 are very close to the experimentally recorded values (generally, the difference is less than  
334 10%). For the residual phase, the differences are slightly larger, but can be explained by  
335 the numerous jumps present on the experimental curves (Figure 2).

336

337 [FIGURE 10 HERE]

338 [FIGURE 11 HERE]

339 [FIGURE 12 HERE]

340 The shear behaviour model can be considered as calibrated, as the numerical shear  
341 test results matches the experimental results.

342 The next step consists of a validation in which we aim to assess the ability of the  
343 shear behaviour model to predict the rock joint shear behaviour of all rock joints. To this  
344 end, the aim of the next section is to compare the model blind-predictions to other  
345 published experimental direct shear test results that were not used to develop the  
346 numerical model.

### 347 **3. Validation of the MC shear behaviour model with literature data**

#### 348 ***3.1. Published data used for the validation step***

349 We consider four articles in the rock mechanics literature that present experimental direct  
350 shear tests carried out in the validity conditions of the shear behaviour model developed  
351 in this study. In these four studies, the joint roughness indicator ( $Z_2$ ) ranges between 0  
352 and 0.373 and joints were subjected to normal load ranging from 0.1 to 0.6-MPa during  
353 the direct shear tests (Lee, Park, and Song 2014; Jang and Jang 2015; Li et al. 2018; Ram  
354 and Basu 2019). Please note that the data from these studies were not used to develop the  
355 numerical model previously described.

356 Ram and Basu (2019) studied the shear behaviour of unfilled natural rock joints  
357 with reference to the weathering grade of the joint. Due to the different weathering grade  
358 used by the authors, only seven direct shear tests results could be used (Table 4). These  
359 shear tests were carried out on smooth rock joints ( $Z_2$  ranges from 0.057 to 0.130) under  
360 a normal stress of about 0.2-MPa.

361 Jang and Jang (2015) conducted more than 180 direct shear tests on rock joint  
362 replicas. Forty direct shear test results could be used directly in this research paper:  $Z_2$   
363 ranges from 0.082 to 0.351 and the normal stresses levels are 0.2 and 0.53-MPa MPa  
364 (Table 4).

365 From Lee, Park, and Song (2014), six direct shear tests results were extracted:  $Z_2$   
366 ranges from 0.142 to 0.253 and the normal stress levels range from 0.1 to 0.5-MPa (Table  
367 4).

368 Lastly, in Li et al. (2018), only one direct shear test was conducted in the  
369 conditions presented above ( $Z_2 < 0.373$  and the normal load from 0.1 to 0.6-MPa).

370 In brief, this amounts to fifty-four direct shear results selected from several  
371 published rock mechanics papers that were used to validate the shear behaviour model  
372 developed in this paper. The selected direct shear tests were carried out at various normal  
373 stress levels (from 0.1 to 0.53-MPa) on natural rock joints or on rock joint replicas that  
374 exhibited several levels of roughness ( $Z_2$  ranging from 0.057 to 0.351). All of the selected  
375 papers gave the peak shear strength, whereas only eleven residual shear strengths were  
376 available.

### 377 **3.2.Results**

378 Table 4 and Figure 13 compare the experimental direct shear tests results from literature  
379 to the shear behaviour model blind-predictions. In Table 4, the relative difference between  
380 experimental results and model blind-predictions are also given.



406 with the ISRM (1978) description. In Rullière et al. (2020), the granite joints exhibited  
407 mainly sliding mechanisms during the direct shear tests. On the other hand, in the study  
408 of Jang and Jang (2015), the rock joints came from different rock types including granite  
409 and schist joints, which are described being “*very rough and stepped*” in accordance with  
410 the ISRM (1978). Very rough or stepped asperities will tend to gather the shear stresses  
411 during a direct shear test, and schist joints will mainly be damaged by shearing  
412 mechanisms. The reader should note that the direct shear tests conducted in (Lee, Park,  
413 and Song 2014; Li et al. 2018; Ram and Basu 2019) were carried out on granite and gneiss  
414 joints. In Jang and Jang (2015), as the rock type was not linked to the joint identification  
415 code, it is therefore impossible to know which rock joint replica came from granite or  
416 schists joints. Nevertheless, we attributed the large difference between the experimental  
417 data and the model blind-predictions to the rock type (schist), as explained above. If the  
418 specific large differences of Jang and Jang (2015) are excluded (see dashed circles in  
419 Figure 13), the mean relative error between the experimental data from the rock  
420 mechanics literature and the model blind-predictions is 7.9%.

421       Regarding the residual shear strength ( $\tau_r$ ), a mean relative difference of 23% was  
422 obtained between the experimental results and the model blind-predictions (Table 4).  
423 Although this mean relative difference is based on a few experimental results, as not all  
424 the selected papers gave the residual shear strength, this gap between the experimental  
425 data and the model blind-prediction can be explained by the following reasons:

- 426       • The model approach to predicting the residual shear strength is very conservative.  
427       The shear stress decreases suddenly to its residual value once the peak is reached  
428       whereas during an experimental direct shear test, the shear stress decreases  
429       gradually to its residual value,

430 • Also, during this progressive shear stress decrease observed in the experimental  
431 tests, some shear stress jumps could be observed, very likely related to sheared  
432 material that interfered between the joint walls.

433 Finally, it appeared that the MC shear behaviour model integrating rock joint  
434 roughness was able to accurately predict the experimental peak shear strength of various  
435 direct shear tests published in the rock mechanics literature. For some specific cases, the  
436 blind-predictions were quite far from the experimental results, but in our opinion these  
437 differences can be explained by the rock type. Indeed, the shear behaviour model was  
438 developed using granite joints while some of the data presented were related to schist  
439 joints.

#### 440 **4. Application of the MC shear behaviour model to a gravity dam**

441 Gravity dams are mainly exposed to shear mechanisms: under the combination of  
442 different actions, shearing can take place at the rock-concrete interface or at rock mass  
443 foundation joints. International gravity dam design guidelines suggest using the Mohr-  
444 Coulomb criterion to assess the shear strength of a rock joint of such structures (USBR  
445 1987; US Army Corps of Engineers 1995; CFBR 2012; Federal Energy Regulatory  
446 Commission 2016).

447 Part 3 showed that the shear strength and behaviour of a rough rock joint subjected  
448 to low normal stresses could be accurately estimated by the numerical shear behaviour  
449 model developed, which takes into account joint roughness (developed in Part 2). In this  
450 Part 4, we wish to highlight: i) that the shear behaviour model developed previously could  
451 be used at the dam scale, and ii) the influence of the rock joint roughness on the shear  
452 behaviour of a gravity dam, subjected to different load cases.

453 ***4.1.Presentation of the numerical gravity dam model and hypothesis***

454 The gravity dam studied is founded on a granitic rock mass from Canadian shield,  
455 intersected by three sets of joints that are prone to discontinuities in the displacement  
456 field, i.e. finite relative displacements across themselves. Among these sets, one is sub-  
457 horizontal and two sub-vertical, presenting dips of  $16^\circ$  N  $0^\circ$ ,  $80^\circ$  N  $0^\circ$  and  $82^\circ$  N  $180^\circ$ ,  
458 respectively. The joints of the same set are spaced according to a uniform distribution,  
459 whose standard deviations and mean values are given in Table 5. The persistence of the  
460 joints was considered as fully persistent (default preset). This agrees with the observations  
461 of the drillings carried out on the dam site, in the context of a dam founded on the  
462 Canadian shield where the persistence is generally very high.

463

464 All the joints exhibited the same roughness characteristics, described by a  $Z_2$  value. In  
465 the following calculations, the  $Z_2$  value was increased from zero (depicting a smooth  
466 joint) to 0.373 (depicting a very rough rock joint). It was also assumed that the rock mass  
467 joints are very long compared with the dam size and they can be considered as persistent  
468 at the numerical study scale. In other words, the joints we that the joints cut the entire  
469 rock foundation mass. There is therefore no increase in the length of the joints. On the  
470 other hand, the joints are free to open. The granite density was set to  $2500 \text{ kg/m}^3$ .

471 [TABLE 5 HERE]

472 The specifications of the case study gravity dam are:

- 473
- 474 • Height of 45m and an operating level of 41 m;
  - 475 • The width at the head of the structure is 4 m;
  - 476 • The upstream face is vertical, while the downstream batter (H/V) is 0.65;
  - The width at the toe is 32 m.

477 The dam is fully made of concrete (density of 2350 kg/m<sup>3</sup>) and it is assumed that  
478 its construction was of good quality: rock-foundation and construction interfaces were  
479 treated. In other words, excellent adhesion can be considered for concrete-concrete  
480 construction joints and rock-concrete interfaces. Therefore, relative displacements can  
481 only occur in the rock mass, along the rock joints described previously. For the sake of  
482 convenience, it was also assumed that the gravity dam was not equipped with any  
483 drainage system. Figure 14 shows a schematic view of the gravity dam modelled on its  
484 granitic rock mass foundation.

485 [FIGURE 14 HERE]

486 The numerical model is built using UDEC software (Itasca 2019) with a Discrete  
487 Element Method (DEM) approach. The DEM makes it straightforward to describe the  
488 discontinuity of displacements across joints, while a continuous-based FEM approach  
489 would require to adopt a more complex model, typically XFEM (Moës et al., 1999). In  
490 this DEM model, the particles are blocks of different sizes and shapes that come from  
491 three sets of discontinuities that cut the granitic massif. Numerical damping was used  
492 with a 0.8 coefficient and the time step was fixed at 30  $\mu$ s to avoid divergence of the  
493 explicit dynamic scheme. The boundaries of the numerical model were fixed in all  
494 directions and located at a distance of at least five times the structure height to avoid any  
495 interferences.

496 Different load cases were considered:

- 497 (1) Normal Operating Conditions: the reservoir is filled;  
498 (2) Extreme Conditions: the reservoir is filled and the dam is subjected to a seismic  
499 solicitation.

500 In Normal Operating Conditions, the dam is only subjected to mechanical actions  
501 related to the presence of water upstream of the structure (water pressure on the upstream

502 face of the dam, uplift). The reservoir is instantly filled with water from the natural  
503 foundation ground to the normal operating level (41 m of water upstream, 0 m of water  
504 downstream). The hydrostatic pressure field is imposed on the gravity dam rock  
505 foundation joints and assumed to be constant in time.

506 The seismic calculation is performed using a simple pseudo-static method, as  
507 suggested by French regulations. The values of the horizontal ( $a_h$ ) and vertical  
508 acceleration ( $a_v$ ) are:  $a_h = 2.0 \text{ m/s}^2$  and  $a_v = 1.8 \text{ m/s}^2$ .

509 For the two loading cases considered, the analysis of the behaviour of the structure  
510 is carried out based on horizontal and vertical displacements of: i) the dam downstream  
511 face crest, and ii) the dam foundation at a depth of 3 m below the upstream toe.

#### 512 ***4.2. Results and discussion***

513 Table 6 shows the displacements recorded during the numerical modelling of a gravity  
514 dam subjected to load levels. From Table 6, it appears that the rock joint roughness has  
515 an important effect on the horizontal displacements recorded at the dam crest or in its  
516 foundation. However, the effect of rock joint roughness on the vertical displacements is  
517 not obvious.

518 For smooth rock foundation joints ( $Z_2 = 0$ ), when the dam is filled and without  
519 seismic solicitation, the horizontal and vertical displacements recorded at the dam crest  
520 reach respectively a value of 17.2 mm and -0.74 mm. When seismic solicitation is applied,  
521 the displacements at the dam crest increase and reach the values of 24.5 mm for the  
522 horizontal displacement and 2.21 mm for the vertical displacement (Table 6).

523 For the rough rock foundation joints ( $Z_2 = 0.373$ ), the horizontal displacements  
524 observed at the crest under normal operations and seismic conditions are reduced in  
525 comparison to smooth joints. Under normal operating conditions, the horizontal dam crest  
526 displacements reach the value of 11.00 mm maximum versus 17.2 mm for smooth joints,





551 constitutive stress-displacement laws. The model shear failure corresponded to the Mohr-  
552 Coulomb criterion including apparent cohesion.

553 The shear behaviour model was developed based on more than thirty-five  
554 experimental tests (direct shear tests and compression tests). Power laws that correlated  
555 the joint roughness parameter  $Z_2$  to the shear behaviour input parameter were developed  
556 (calibration step, Part 2). The model was then validated by comparing its blind-  
557 predictions to published experimental data (validation step, Part 3). The results indicated  
558 that the MC shear behaviour model integrating rock joint roughness was able to predict  
559 the peak shear strength of an unbounded rock joint with an average relative error of 7.9%).  
560 The shear behaviour model developed was found to be strongly influenced by the rock  
561 type.

562 Lastly, the shear behaviour model developed was applied at full scale (gravity  
563 dam) to highlight the role of rock joint roughness in dam shear behaviour. This practical  
564 application exhibited that the higher the rock joint roughness ( $Z_2$  value), the smaller the  
565 displacements (either at the dam crest or in its foundations).

566 Regarding the perspectives of this work, it is important to recall that the MC shear  
567 behaviour model integrating rock joint roughness is valid for specific conditions: a normal  
568 load from 0.1 to 0.6-MPa and a roughness indicator  $Z_2$  ranging from 0 to 0.373. Moreover,  
569 the shear behaviour model developed takes into account the effects of joint roughness on  
570 its shear strength and behaviour. However, the reader should know that other parameters  
571 could also have strong effects (rock properties, interlocking, contact properties, the  
572 presence of infilled materials in joints, etc.).

573 Future work will attempt to incorporate other shear behaviour / strength influence  
574 parameters in the shear behaviour model developed. However, this task requires much  
575 more data and implies launching new experimental and exhaustive studies. Finally, it is

576 important to note that the last part of this paper is intended to be only explanatory since  
577 the tests were conducted at the laboratory scale. Thus, more work is required to use the  
578 shear behaviour model developed accurately at a larger scale.

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772

773 **Appendix A**

774 Linear laws correlating the  $Z_2$  value to the shear behaviour model input parameters.

775 
$$K_N = 31.22 \cdot x + 68.14 \quad R^2 = 0.96 \quad (8)$$

776 
$$K_S = 4.91 \cdot x + 1.96 \quad R^2 = 0.77 \quad (9)$$

777 
$$C_{app\ peak} = 821.9 \cdot x - 28.5 \quad R^2 = 0.88 \quad (10)$$

778 
$$\phi_{peak} = 118.5 \cdot x + 28.06 \quad R^2 = 0.96 \quad (11)$$

779  $\varphi_{\text{residual}} = 89.6 \cdot x + 25.6$   $R^2 = 0.98$  (12)

780  $dN = 79.55 \cdot x + 0.26$   $R^2 = 0.99$  (13)

781 Exponential laws correlating the  $Z_2$  value to the shear behaviour model input parameters.

782  $K_N = 15.87 \cdot e^{1.49 \cdot x} + 52.61$   $R^2 = 0.97$  (14)

783  $K_S = 0.257 \cdot e^{5.77 \cdot x} + 1.90$   $R^2 = 0.94$  (15)

784  $C_{\text{app peak}} = 56.64 \cdot e^{5.09 \cdot x} - 55.01$   $R^2 = 0.99$  (16)

785  $\varphi_{\text{peak}} = 1.164 \times 10^4 \cdot e^{0.011 \cdot x} - 1.164 \times 10^4$   $R^2 = 0.96$  (17)

786  $\varphi_{\text{residual}} = 1.081 \times 10^4 \cdot e^{0.006 \cdot x} - 1.081 \times 10^4$   $R^2 = 0.96$  (18)

787  $dN = 1.809 \times 10^4 \cdot e^{0.0041 \cdot x} - 1.809 \times 10^4$   $R^2 = 0.98$  (19)

Detailed answers to the reviewers

*Integration of rock joint roughness into the Mohr-Coulomb shear behaviour model – Application to dam safety analysis*

Authors: Adrien RULLIERE, Laurent PEYRAS, Jérôme DURIEZ, Patrice RIVARD and Pierre BREUL

European Journal of Environmental and Civil Engineering

| N° remark                             | Remark  | Answer   |
|---------------------------------------|---|--|
| <b>Reviewer 1</b>                     |   |  |
| 1                                     | The quality of figures is still bad and some figures are even blurry  | We are very sorry about the quality of the figures that was lost during the generation of the PDF file. The figures have been taken out of the text and attached in vector PDF format. The quality is much better now. Again, we are sorry for this problem. |
| <b>Reviewer 2</b>                     |   |  |
| 1                                     | In Section "Material and Methods", the experimental results should be provided firstly, then introduce the MC model and DEM.  | Done   |
| 2                                     | The references cited in introduction should be up to date. Please refer to the relevant literature 10.1007/s10064-019-01536-9 ; 10.1016/j.jrmge.2022.04.006 10.1016/j.engfracmech.2022.108427 ; 10.1016/j.jrmge.2016.11.004 | Done. These references have been added.  |
| 3                                     | Some figures are not in high quality, for instance, the fonts in Figs 2, 6, 10, 11 and 13 ... are too small to difficult to identify. It is kindly suggested that improving the quality.                                    | Done. See the answer to the question n°1 – reviewer n°1  |
| 4                                     | Eq (3). The explanation of symbols should be in an academic style.  | Done, see reviewer 3 question 3.   |
| 5                                     | Eq (7) "Z2" should be subscript.  | Done   |
| 6                                     | Some of the subtitles of subsections should be more concise and clearer.  | Done. Several subtitles have been changed and shortened.   |
| 7                                     | Please unify the term "rock joint" and "shear strength of rock joint"   | Done. We have change all the “rock joint shear strength” by “shear strength of rock joint”   |
| Corrections suggested on the PDF File |   | We thank the reviewer for his careful proofreading which improves the article All the corrections suggested have been taken into account.  |

**Reviewer 3**

|   |   |  |
|---|---|--|
| 1 | The figure quality is extremely bad   | See the answer to the question n°1 – reviewer n°1  |
| 2 | Do you think your model could apply to the kilometer scale? If so, how? If not, why?  | <p>We do not think that the model could apply to the kilometer scale, due to the scale effect. The model considers the roughness provided by the joint asperities (micro and macroscale), but it does not take into account the waviness of the joint.</p> <p>In the introduction of § 1.1, we say on this subject: <i>“It is specified in this introduction to the experimental data that our work does not take into account the scale effect between the sample tested in the laboratory and the real scale model. Several authors such as (Bandis et al. 1981) have worked on the scale effect and have shown the influence of sample size on shear strength. This question of scale effect between laboratory and field is, in general, a universal problem of this kind of study that our research has not taken into account.”</i></p>  |
| 3 | Please learn how to embed a mathematical equation in Word professional. I have attached a file for you to learn (together with high-resolution figures).  | Thanks for the comment. We modified the equations in the manuscript and the explanation of symbols are now in an academic style.   |
| 4 | <p>Why do you prefer the Mohr-Coulomb model to the Drucker-Prager model? Have you considered strain-softening in your analysis?</p> <p>Please refer to the following papers, discuss them, and include them in your reference list.<br/>DOI: 10.1016/j.compgeo.2022.104728<br/>DOI: 10.1016/j.cma.2022.115666</p> | <p>We use the Mohr-Coulomb model because it is the model that is most widely used in the world in dam engineering to assess the stability of dams. We mentioned this in the introduction: <i>“ The aim of this article is to integrate rock joint roughness components into the Mohr-Coulomb shear behaviour model which is chosen for its wide usage in engineering practice strength (USBR 1987; US Army Corps of Engineers 1995; CFBR 2012; ISRM 2014; Federal Energy Regulatory Commission 2016), in order to take into account the role of joint roughness on joint shear behaviour.”</i></p> <p>The Drucker-Prager model is not so used in dam engineering.</p> <p>We considered a brittle deformation-damping in our MC model since, during the numerical shear test, the shear stress reaches a maximum value before a sudden drop to a residual value.</p> <p>Done. These references have been added.</p> |
| 5 | Do you consider the propagation (dynamics) of joints?   | <p>The joints were considered persistent at the scale of the model, which means that the joints cut the whole rock mass foundation. There is therefore no increase in the length of the joints. On the other hand, the joints are free to open.</p> <p>These details were already into the paper: please see lines 459-462 and 466-468. We added some details to be more understandable. Please see lines 468-470.</p>   |

Table 1. Experimental data obtained for each joint studied.

| Joint | Joint roughness statistical indicator $Z_2$ | Apparent cohesion at peak shear strength $C_{APP\_Peak}$ (kPa) | Friction angle at peak shear strength $\varphi_{Peak}$ (°) | Apparent cohesion at residual shear strength $C_{APP\_Residual}$ (kPa) | Friction angle at residual shear strength $\varphi_{residual}$ (°) | Dilatancy angle $dN$ (°) | Joint normal stiffness $K_N$ (GPa/m) | Joint shear stiffness $K_S$ (GPa/m) |
|-------|---|--|--|--|--|--------------------------|--------------------------------------|-------------------------------------|
| J0    | 0   | 0  | 25.6   | 0  | 25.6   | 0                        | 68.5                                 | 2.2                                 |
| J1    | 0.186                                       | 95   | 54.0   | 0  | 40.7   | 15.3                     | 73.8                                 | 2.4                                 |
| J2    | 0.241                                       | 140  | 58.7   | 0  | 49.1   | 20.6                     | 74.4                                 | 3.2                                 |
| J3    | 0.302                                       | 202  | 62.8   | 0  | 54.6   | 22.9                     | 78.4                                 | 3.3                                 |
| J4    | 0.373                                       | 326  | 69.9   | 0  | 57.5   | 30.2                     | 80.0                                 | 4.1                                 |

Table 2. Direct shear test results

| Joint | $\sigma_N$ (MPa) | $\tau_{peak}$ (MPa) | $\tau_{residual}$ (MPa) |
|-------|------------------|---------------------|-------------------------|
| J0    | 0.10             | 0.04                | 0.04                    |
|       | 0.20             | 0.09                | 0.09                    |
|       | 0.35             | 0.17                | 0.17                    |
|       | 0.60             | 0.32                | 0.32                    |
| J1    | 0.10             | 0.24                | 0.11                    |
|       | 0.10             | 0.26                | 0.07                    |
|       | 0.20             | 0.41                | 0.20                    |
|       | 0.35             | 0.52                | 0.38                    |
|       | 0.60             | 0.94                | 0.59                    |
|       | 0.60             | 1.02                | 0.47                    |
| J2    | 0.10             | 0.31                | 0.11                    |
|       | 0.10             | 0.32                | 0.14                    |
|       | 0.20             | 0.52                | 0.22                    |
|       | 0.35             | 0.68                | 0.43                    |
|       | 0.60             | 1.34                | 0.73                    |
|       | 0.60             | 1.05                | 0.75                    |
| J3    | 0.10             | 0.39                | 0.22                    |
|       | 0.20             | 0.60                | 0.37                    |
|       | 0.35             | 1.00                | 0.61                    |
|       | 0.60             | 1.42                | 0.96                    |
|       | 0.60             | 1.41                | 0.76                    |
| J4    | 0.10             | 0.61                | 0.21                    |
|       | 0.10             | 0.58                | 0.15                    |
|       | 0.20             | 1.02                | 0.43                    |
|       | 0.35             | 1.22                | 0.51                    |
|       | 0.60             | 1.93                | 0.94                    |
|       | 0.60             | 2.20                | 1.06                    |

Table 3. Power law coefficients a, b and c values according to the input parameter of interest.

| Shear behaviour model input parameter | <i>a</i> | <i>b</i> | <i>c</i> |
|---------------------------------------|----------|----------|----------|
| $C_{app\ peak}$                       | 2120.0   | 1.93     | 2.81     |
| $\Phi_{peak}$                         | 82.17    | 0.64     | 25.62    |
| $C_{app\ residual}$                   | 0.0      | 0.0      | 0.0      |
| $\Phi_{residual}$                     | 87.39    | 0.98     | 25.45    |
| dN                                    | 75.0     | 0.94     | 0.02     |
| $K_N$                                 | 39.82    | 1.24     | 58.50    |
| $K_S$                                 | 15.55    | 2.12     | 2.17     |

Table 4: Comparison of experimental direct shear test results from the rock mechanics literature (Lee, Park, and Song 2014; Jang and Jang 2015; Li et al. 2018; Ram and Basu 2019) and the numerical blind-predictions of the shear behaviour model developed in this paper - all values are in MPa. The results in italics have been left aside for future work.

| Reference                   | Sample ID | $Z_2$ | $\sigma_N$ | Experimental data |          | Shear behaviour model blind-predictions |            | %             |             |
|-----------------------------|-----------|-------|------------|-------------------|----------|---|------------|---------------|-------------|
|                             |           |       |            | $\tau_{peak}$     | $\tau_R$ | $\tau_{peak}$                           | $\tau_R$   | $\tau_{peak}$ | $\tau_R$    |
| <b>Ram and Basu (2019)</b>  | JS-15     | 0.057 | 0.23       | 0.22              | 0.20     | 0.20                                    | 0.14       | <b>8.5</b>    | <b>31.4</b> |
|                             | JS-19     | 0.130 | 0.23       | 0.49              | 0.28     | 0.31                                    | 0.18       | <b>36.5</b>   | <b>37.2</b> |
|                             | JS-21     | 0.057 | 0.23       | 0.19              | 0.15     | 0.20                                    | 0.14       | <b>5.7</b>    | <b>8.7</b>  |
|                             | JS-26     | 0.047 | 0.24       | 0.19              | 0.15     | 0.20                                    | 0.14       | <b>3.1</b>    | <b>8.0</b>  |
|                             | JS-30     | 0.130 | 0.23       | 0.34              | 0.24     | 0.31                                    | 0.18       | <b>8.6</b>    | <b>26.8</b> |
|                             | JS-33     | 0.084 | 0.23       | 0.23              | 0.20     | 0.24                                    | 0.15       | <b>3.8</b>    | <b>24.6</b> |
|                             | JS-34     | 0.061 | 0.22       | 0.23              | 0.20     | 0.20                                    | 0.13       | <b>13.9</b>   | <b>33.5</b> |
| <b>Li et al. (2018)</b>     | J3        | 0.235 | 0.27       | 0.66              | 0.50     | 0.60                                    | 0.29       | <b>8.9</b>    | <b>42.8</b> |
| <b>Jang and Jang (2015)</b> | N2000-J1  | 0.082 | 0.20       | 0.23              | -        | 0.21                                    |            | <b>10.1</b>   | -           |
|                             |           |       | 0.53       | 0.56              | -        | 0.52                                    |            | <b>8.0</b>    | -           |
|                             | N2000-J2  | 0.118 | 0.20       | 0.26              | -        | 0.26                                    |            | <b>1.0</b>    | -           |
|                             |           |       | 0.53       | 0.62              | -        | 0.62                                    |            | <b>0.1</b>    | -           |
|                             | N2000-J3  | 0.127 | 0.20       | 0.33              | -        | 0.27                                    |            | <b>16.8</b>   | -           |
|                             |           |       | 0.53       | 0.73              | -        | 0.65                                    |            | <b>10.7</b>   | -           |
|                             | N2000-J4  | 0.126 | 0.20       | 0.27              | -        | 0.27                                    |            | <b>1.5</b>    | -           |
|                             |           |       | 0.53       | 0.64              | -        | 0.65                                    |            | <b>1.0</b>    | -           |
|                             | N2000-J5  | 0.132 | 0.20       | 0.28              | -        | 0.28                                    |            | <b>0.3</b>    | -           |
|                             |           |       | 0.53       | 0.65              | -        | 0.66                                    |            | <b>3.0</b>    | -           |
|                             | N2000-J6  | 0.139 | 0.20       | 0.35              | -        | 0.29                                    |            | <b>17.3</b>   | -           |
|                             |           |       | 0.53       | 0.79              | -        | 0.69                                    |            | <b>12.5</b>   | -           |
|                             | N2000-J7  | 0.147 | 0.20       | 0.33              | -        | 0.30                                    |            | <b>8.3</b>    | -           |
|                             |           |       | 0.53       | 0.76              | -        | 0.71                                    |            | <b>6.5</b>    | -           |
| N2000-J8                    | 0.161     | 0.20  | 0.35       | -                 | 0.33     |   | <b>4.7</b> | -             |             |



|                                  |           |       |      |      |      |      |            |             |             |
|----------------------------------|-----------|-------|------|------|------|------|------------|-------------|-------------|
|                                  |           |       | 0.53 | 0.79 | -    | 0.76 |            | <b>3.9</b>  | -           |
|                                  | N2000-J9  | 0.154 | 0.20 | 0.32 | -    | 0.32 |            | <b>0.4</b>  | -           |
|                                  |           |       | 0.53 | 0.75 | -    | 0.74 |            | <b>1.7</b>  | -           |
|                                  | N2000-J10 | 0.178 | 0.20 | 0.50 | -    | 0.36 |            | <b>28.0</b> | -           |
|                                  |           |       | 0.53 | 1.03 | -    | 0.82 |            | <b>20.4</b> | -           |
|                                  | N2000-J11 | 0.179 | 0.20 | 0.35 | -    | 0.36 |            | <b>4.5</b>  | -           |
|                                  |           |       | 0.53 | 0.78 | -    | 0.82 |            | <b>5.6</b>  | -           |
|                                  | N2000-J12 | 0.189 | 0.20 | 0.36 | -    | 0.38 |            | <b>7.1</b>  | -           |
|                                  |           |       | 0.53 | 0.80 | -    | 0.86 |            | <b>7.8</b>  | -           |
|                                  | N2000-J13 | 0.210 | 0.20 | 0.41 | -    | 0.42 |            | <b>2.7</b>  | -           |
|                                  |           |       | 0.53 | 0.89 | -    | 0.94 |            | <b>5.7</b>  | -           |
|                                  | N2000-J14 | 0.221 | 0.20 | 0.42 | -    | 0.45 |            | <b>6.7</b>  | -           |
|                                  |           |       | 0.53 | 0.91 | -    | 0.99 |            | <b>8.7</b>  | -           |
|                                  | N2000-J15 | 0.218 | 0.20 | 0.38 | -    | 0.44 |            | <b>15.4</b> | -           |
|                                  |           |       | 0.53 | 0.84 | -    | 0.98 |            | <b>15.9</b> | -           |
|                                  | N2000-J16 | 0.224 | 0.20 | 0.43 | -    | 0.46 |            | <b>7.0</b>  | -           |
|                                  |           |       | 0.53 | 0.90 | -    | 1.01 |            | <b>12.0</b> | -           |
|                                  | N2000-J17 | 0.241 | 0.20 | 0.44 | -    | 0.50 |            | <b>11.8</b> | -           |
|                                  |           |       | 0.53 | 0.98 | -    | 1.08 |            | <b>11.1</b> | -           |
|                                  | N2000-J18 | 0.247 | 0.20 | 0.43 | -    | 0.51 |            | <b>18.7</b> | -           |
|                                  |           |       | 0.53 | 0.93 | -    | 1.11 |            | <b>19.2</b> | -           |
|                                  | N2000-J19 | 0.327 | 0.20 | 0.56 | -    | 0.75 |            | <b>34.1</b> | -           |
|                                  |           |       | 0.53 | 1.15 | -    | 1.58 |            | <b>36.7</b> | -           |
|                                  | N2000-J20 | 0.350 | 0.20 | 0.57 | -    | 0.84 |            | <b>48.0</b> | -           |
|                                  |           |       | 0.53 | 1.19 | -    | 1.75 |            | <b>47.3</b> | -           |
| <b>Lee, Park and Song (2014)</b> | R-463     | 0.142 | 0.20 | 0.28 | -    | 0.29 |            | <b>5.3</b>  | -           |
|                                  | R-1130    | 0.227 | 0.10 | 0.29 | 0.10 | 0.29 | 0.10       | <b>0.6</b>  | <b>1.6</b>  |
|                                  |           |       | 0.20 | 0.35 | -    | 0.46 |            | <b>32.1</b> | -           |
|                                  | R-1233    | 0.244 | 0.20 | 0.42 | 0.29 | 0.50 | 0.22       | <b>19.9</b> | <b>25.2</b> |
|                                  |           |       | 0.50 | 0.96 | 0.50 | 1.04 | 0.55       | <b>9.3</b>  | <b>9.3</b>  |
| R-1284                           | 0.253     | 0.20  | 0.57 | -    | 0.53 |      | <b>7.6</b> | -           |             |

Table 5. Geological data for the joints of the gravity dam rock foundation.

| Joint set | Dip angle  | Mean spacing (m) | Standard deviation on spacing (m) |
|-----------|------------|------------------|-----------------------------------|
| 1         | 16° N 0°   | 0.95             | 0.37                              |
| 2         | 80° N 0°   | 1.52             | 0.82                              |
| 3         | 82° N 180° | 2.33             | 1.77                              |

Table 6. Displacement values recorded during the numerical application at the gravity dam scale of the shear behaviour model developed. Values are in mm.

| Z <sub>2</sub> | Normal operating conditions |                     |                     |                   | Seismic conditions    |                     |                     |                   |
|----------------|-----------------------------|---------------------|---------------------|-------------------|-----------------------|---------------------|---------------------|-------------------|
|                | Crest horizontal disp       | Toe horizontal disp | Crest vertical disp | Toe vertical disp | Crest horizontal disp | Toe horizontal disp | Crest vertical disp | Toe vertical disp |
| 0              | 17.2                        | 13.8                | -0.7                | -2.4              | 24.5                  | 18.1                | 2.2                 | -2.2              |
| 0.05           | 17.9                        | 8.8                 | 1.5                 | -2.4              | 29.2                  | 12.8                | 4.6                 | -2.5              |
| 0.1            | 19.3                        | 8.1                 | -2.2                | -2.2              | 19.8                  | 8.3                 | 2.6                 | -2.2              |
| 0.15           | 16.5                        | 6.9                 | 1.7                 | -2.3              | 28.2                  | 10.1                | 5.5                 | -2.5              |
| 0.2            | 15.1                        | 6.8                 | 1.4                 | -2.4              | 25.4                  | 9.0                 | 4.8                 | -2.4              |
| 0.25           | 13.8                        | 7.5                 | 1.0                 | -2.1              | 23.1                  | 8.9                 | 4.0                 | -2.0              |
| 0.3            | 12.9                        | 6.6                 | 0.8                 | -2.6              | 21.3                  | 7.9                 | 3.6                 | -2.4              |
| 0.373          | 11.0                        | 5.9                 | 0.3                 | -2.8              | 19.0                  | 7.2                 | 2.8                 | -2.6              |

Figure 1: 3D view of the granite joints used in this study (Rullière et al. 2020).

Figure 2: Shear stress versus shear displacement curves from Rullière et al. (2020).

Figure 3: Mohr-Coulomb linear regression at peak and residual state for the five granite joints used in this study.

Figure 4: normal displacement versus shear displacement curves from Rullière et al. (2020).

Figure 5: Photograph (Jacobsson, Flansbjerg, and Andersson 2016) and schematic view of the hydraulic press during the compression tests. The rock joint sample (1) is set in a uniaxial press controlled for displacements. During the joint compression test, extensometers (2) and a load cell (3) record the normal relative displacements and the normal force (N).

Figure 6: Compression test curves for all the granite joints used in this study.

Figure 7: Comparison between the experimental data and empirical power law predictions for apparent cohesion, peak friction angle and residual friction angle.

Figure 8: Comparison between the experimental data and empirical power law predictions for the dilatancy angle.

Figure 9: Comparison between the experimental data and empirical power law predictions for normal and shear stiffness.

Figure 10: Joint shear stress versus shear displacement curves according to: i) in solid lines the model predictions, and ii) in dotted lines the experimental data from Rullière et al. (2020).

Figure 11: Joint normal displacement versus shear displacement curves according to: i) in solid lines the model predictions, and ii) in dotted lines the experimental data from Rullière et al. (2020).

Figure 12: Comparison of experimental direct shear test results and the numerical results of the shear behaviour model developed in this paper

Figure 13: Comparison between the model blind-predictions for peak shear strength and experimental data. Arrows show the largest differences between the model and experimental data.

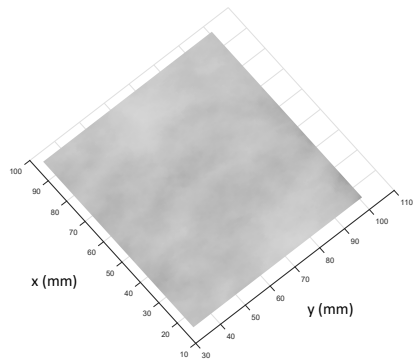
Figure 14: schematic view of the gravity dam model.

Figure 15: Horizontal displacements recorded at the dam crest for each  $Z_2$  value used during the numerical calculations. The normal operation conditions are shown in blue, the extreme condition in red.

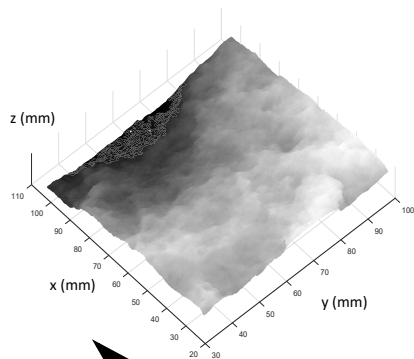
Figure 16: Global displacements (in m) recorded on the gravity dam under seismic conditions for  $Z_2 = 0.1$ .

Figure 17: horizontal displacements recorded at the upstream toe of the dam for each  $Z_2$  value used during the numerical calculations. The normal operation conditions are shown in blue, the extreme condition in red.

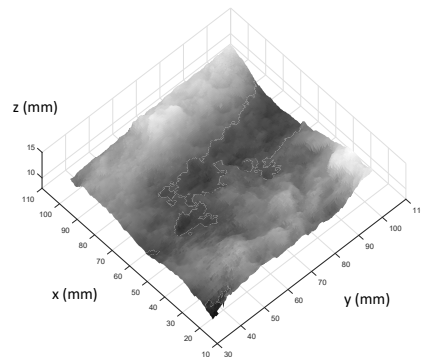
# J0



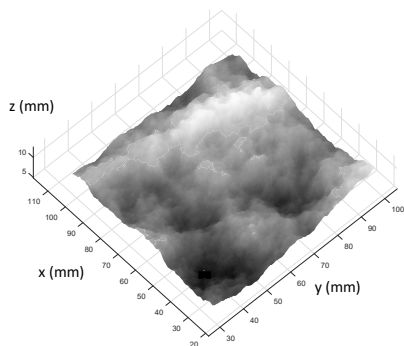
# J1



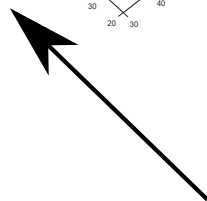
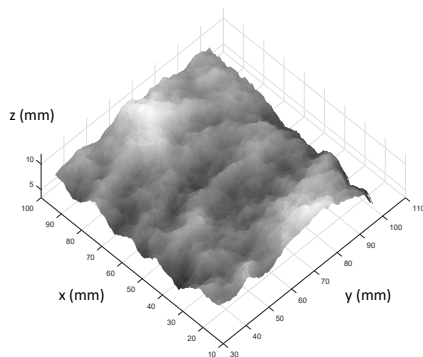
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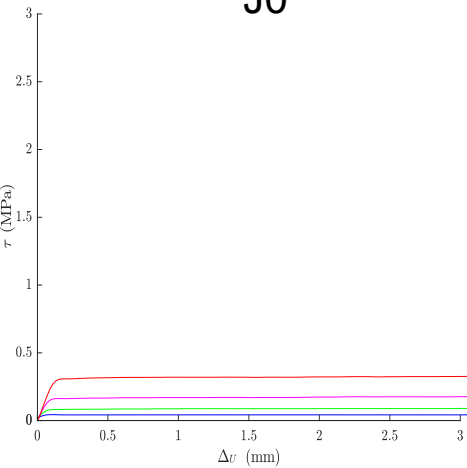
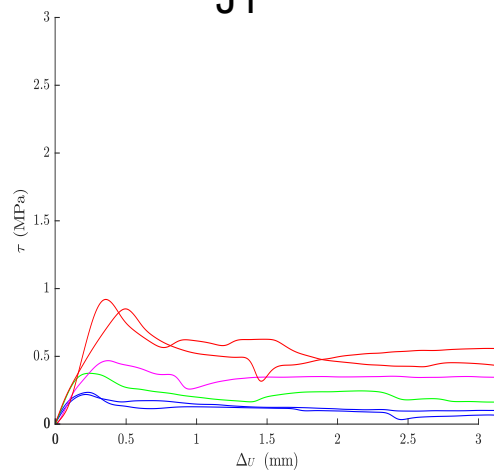
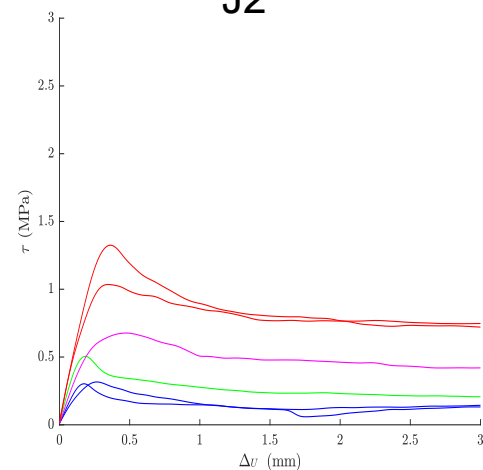
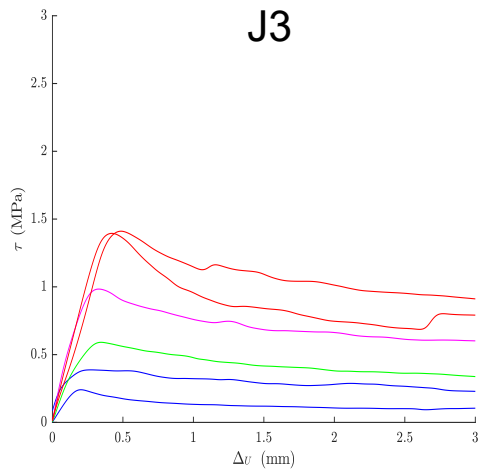
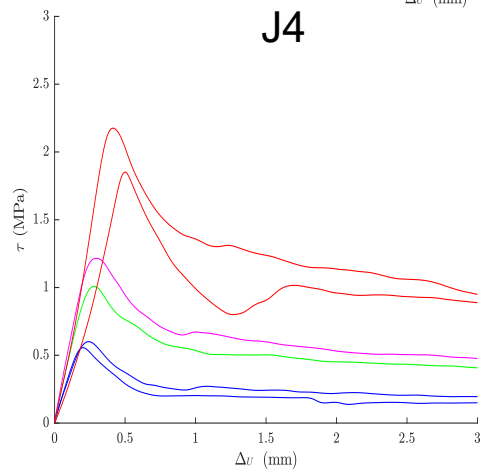


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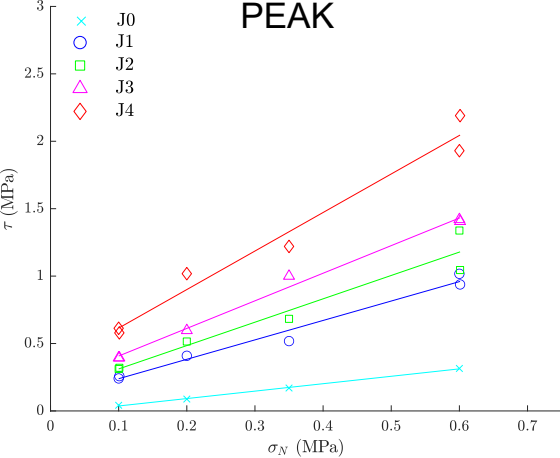
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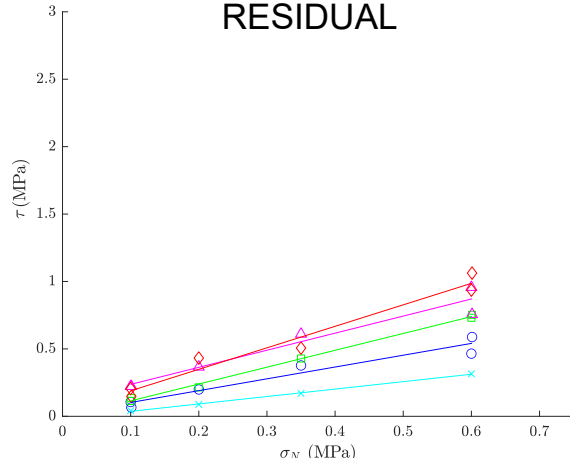
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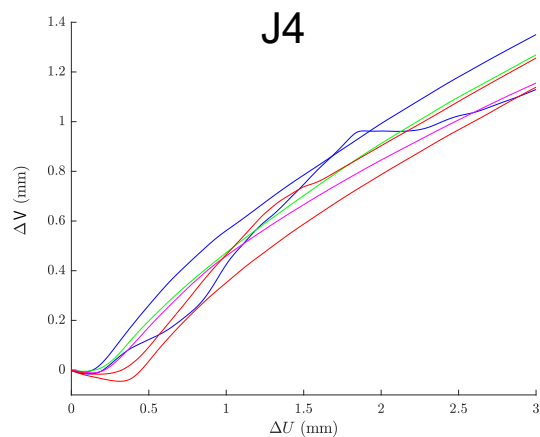
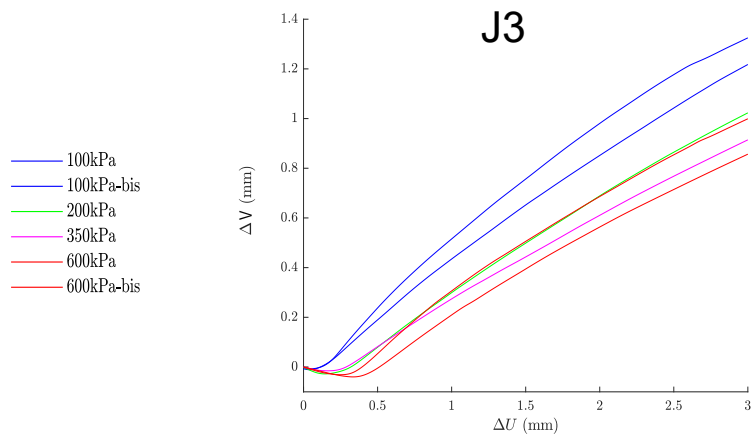
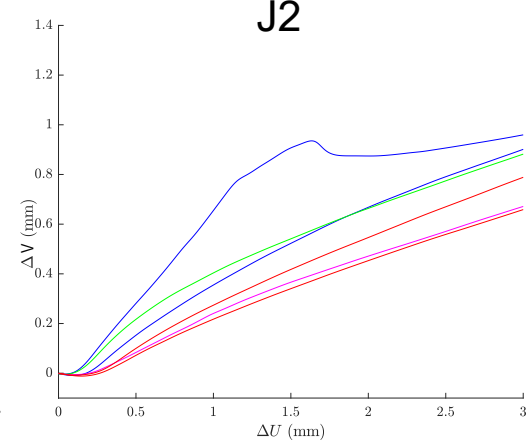
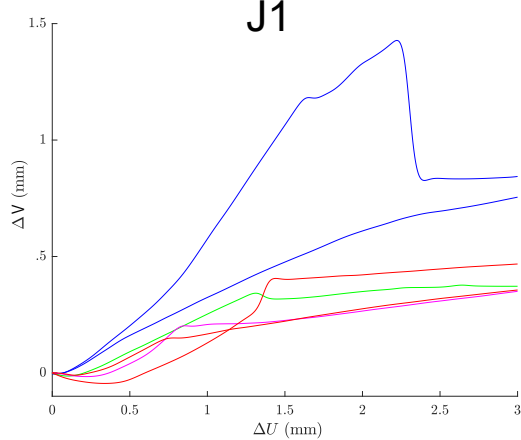
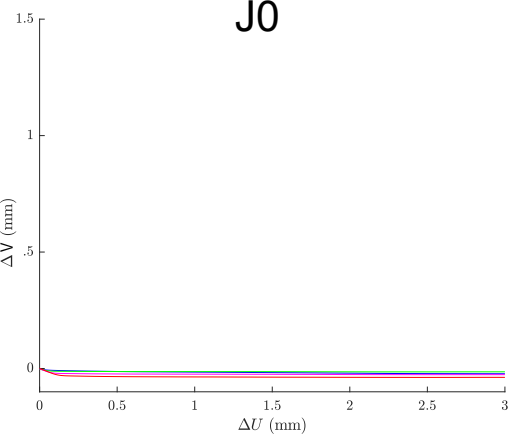
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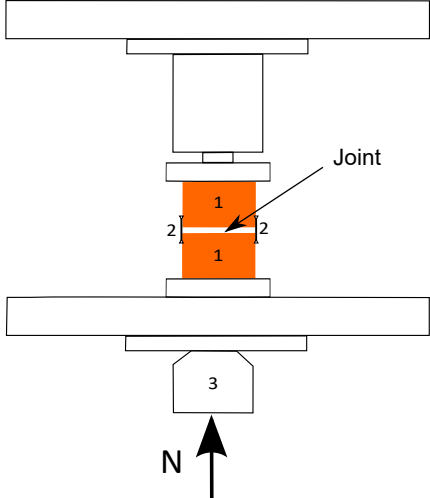
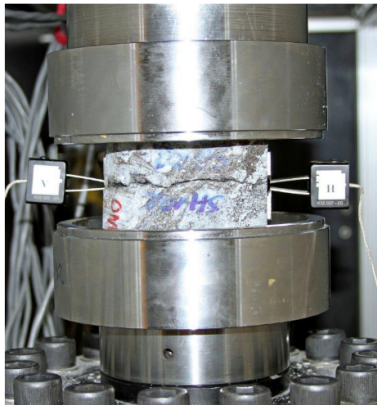
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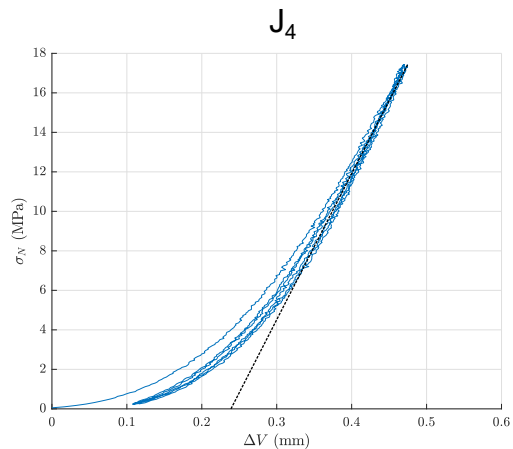
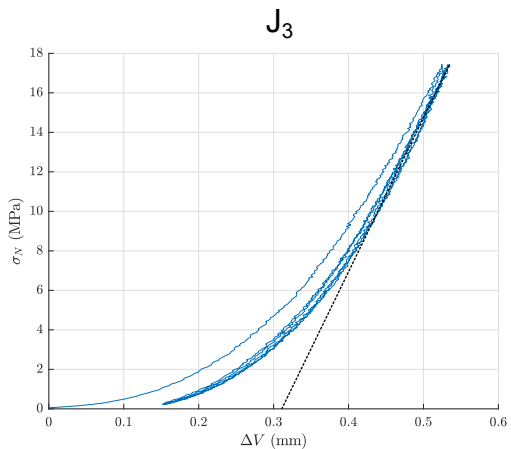
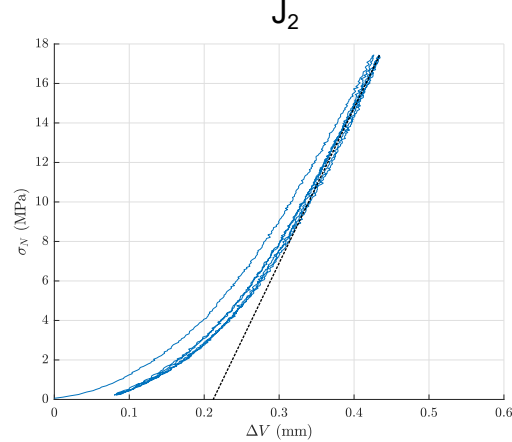
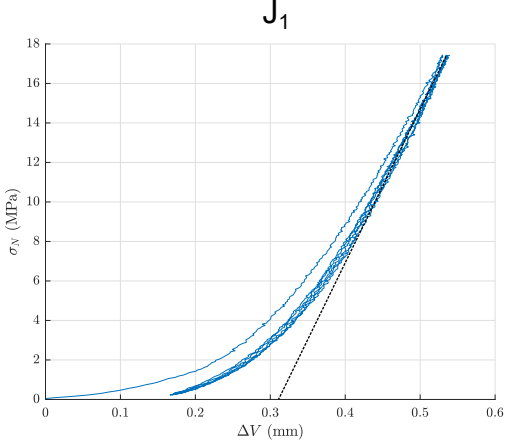
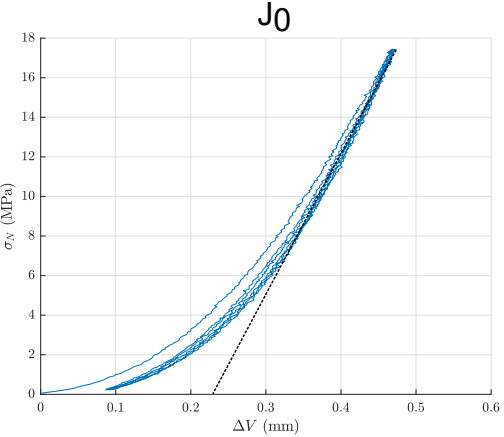
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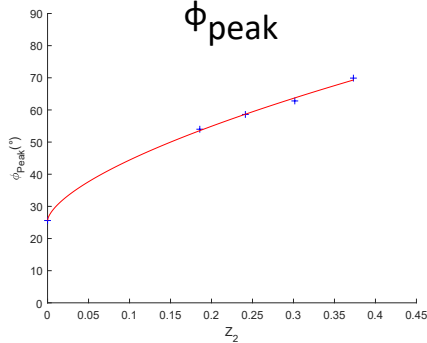
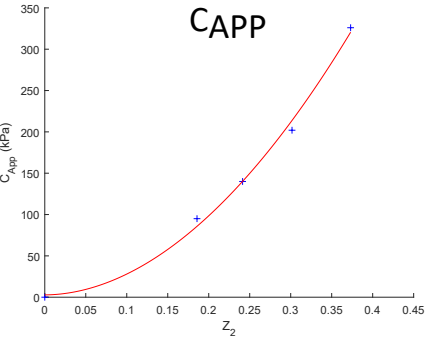




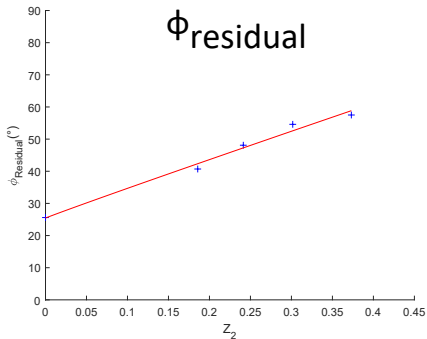


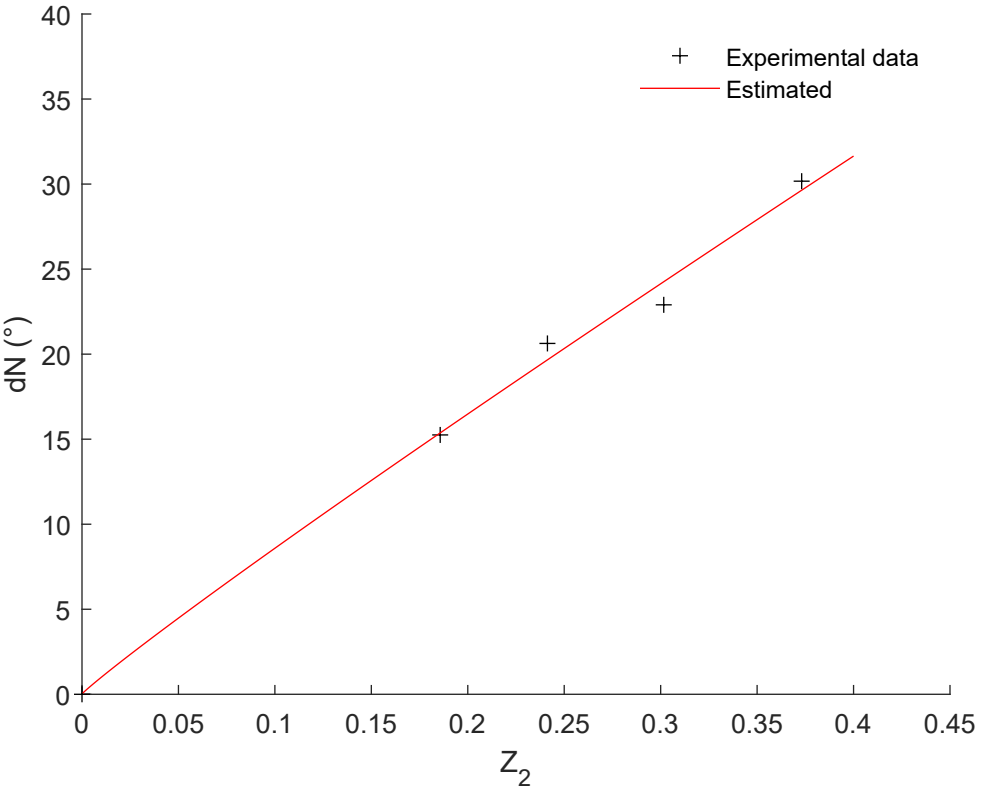


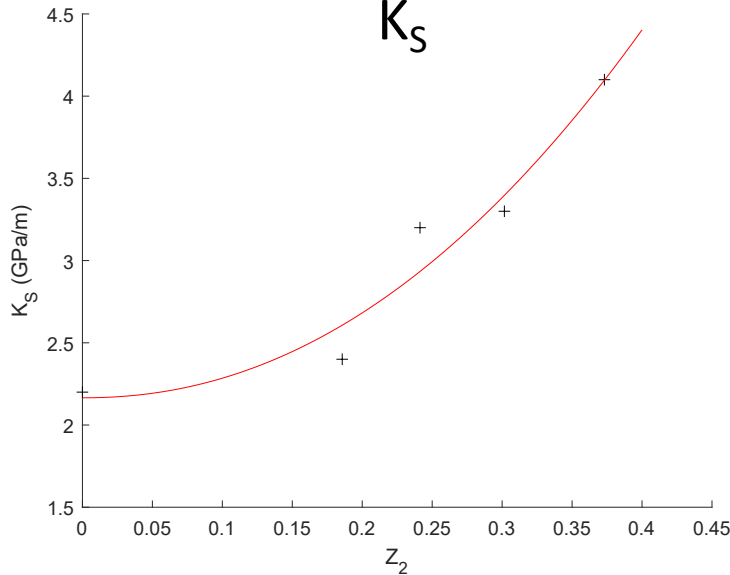
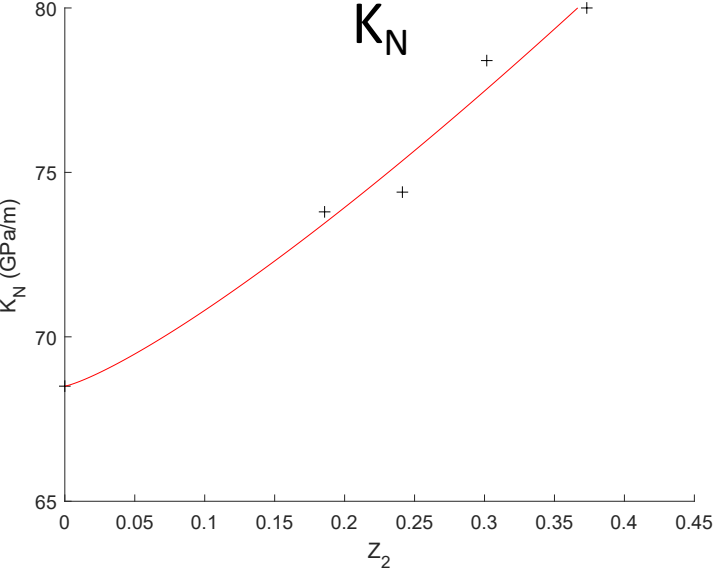




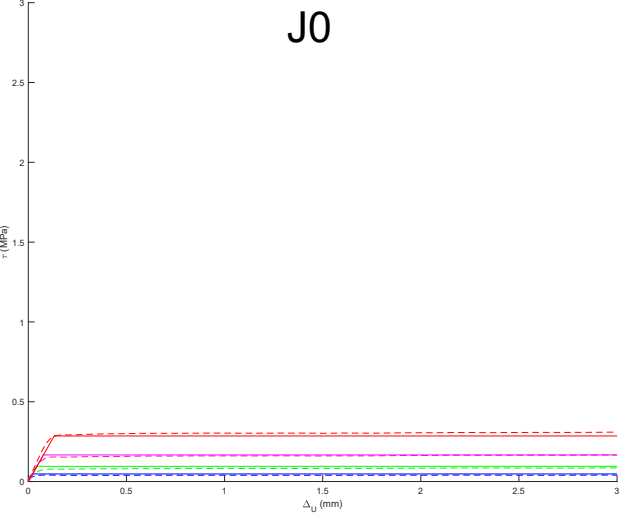
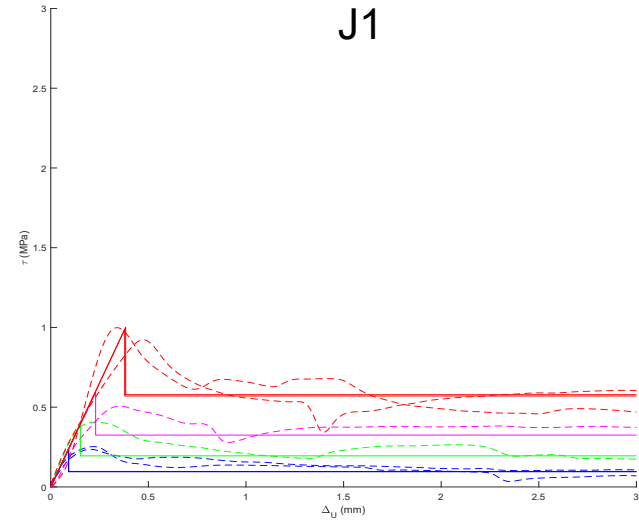
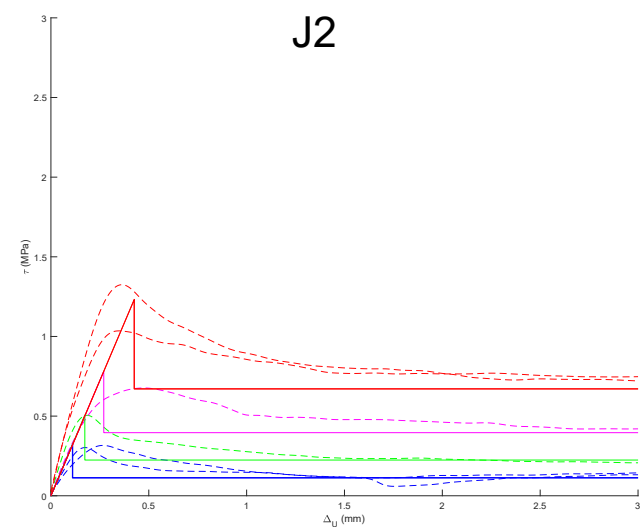
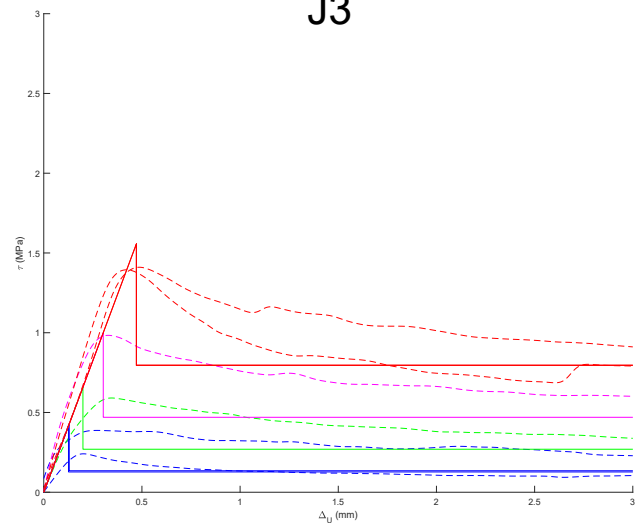
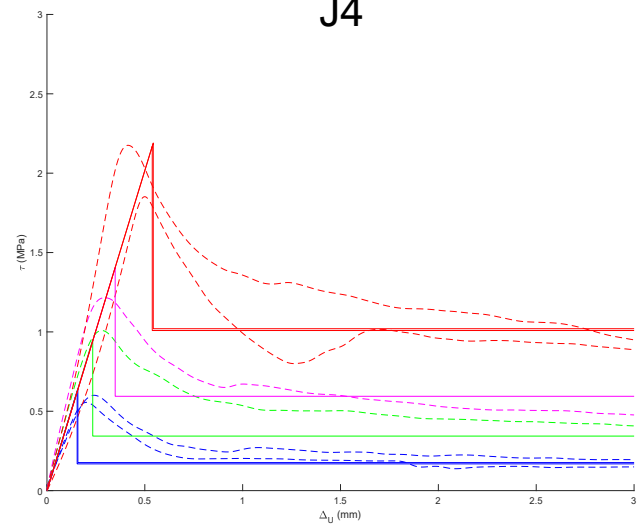
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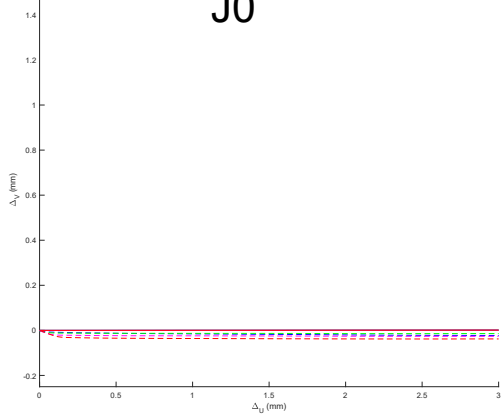
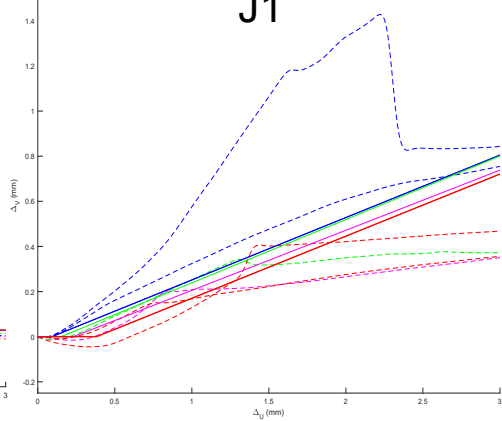
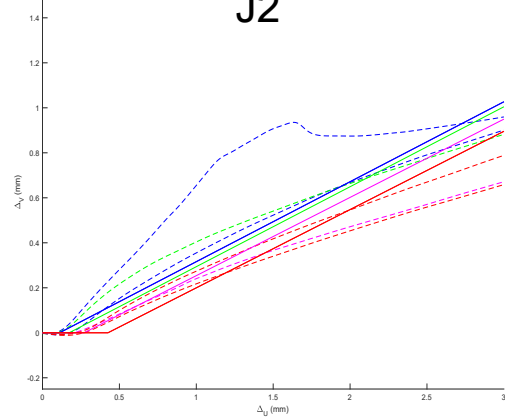
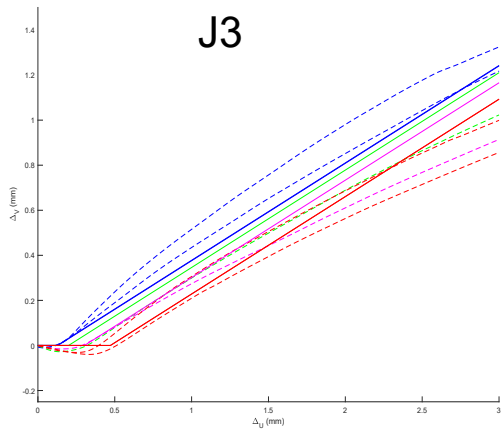
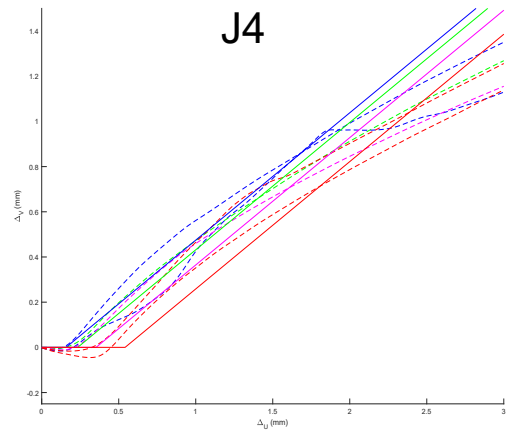




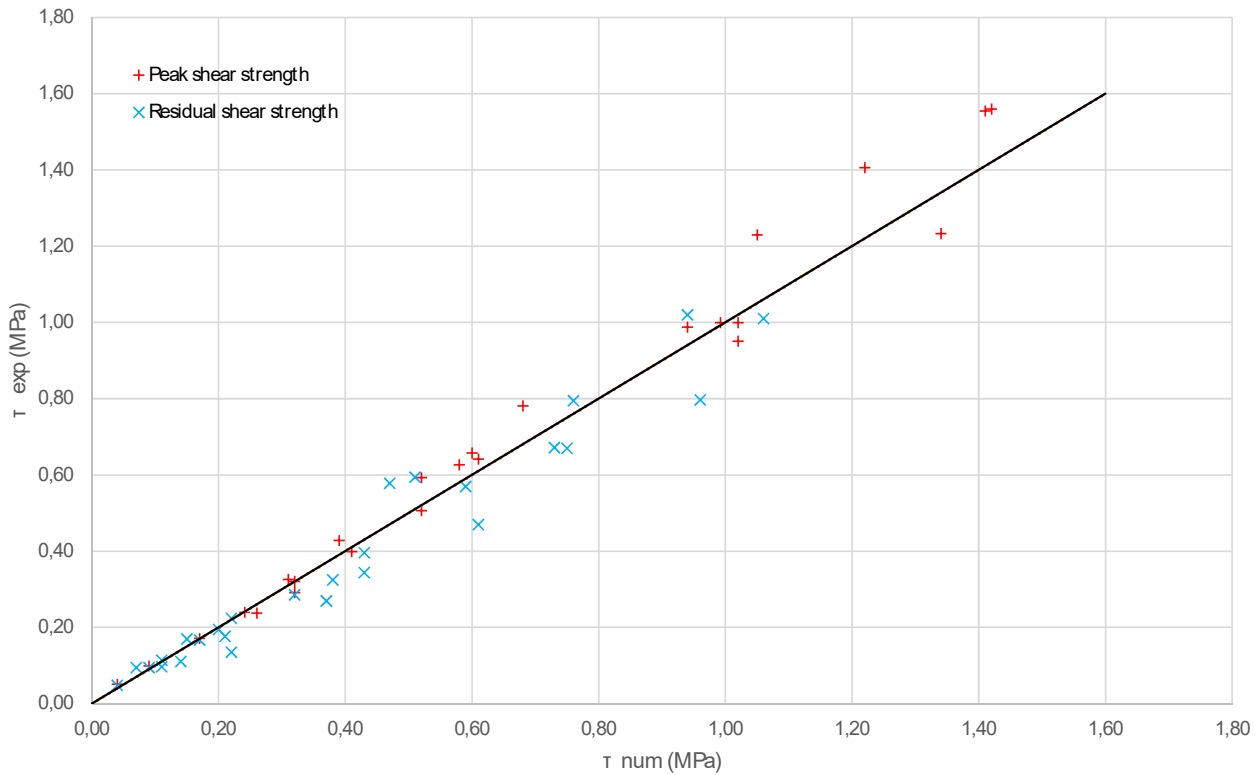
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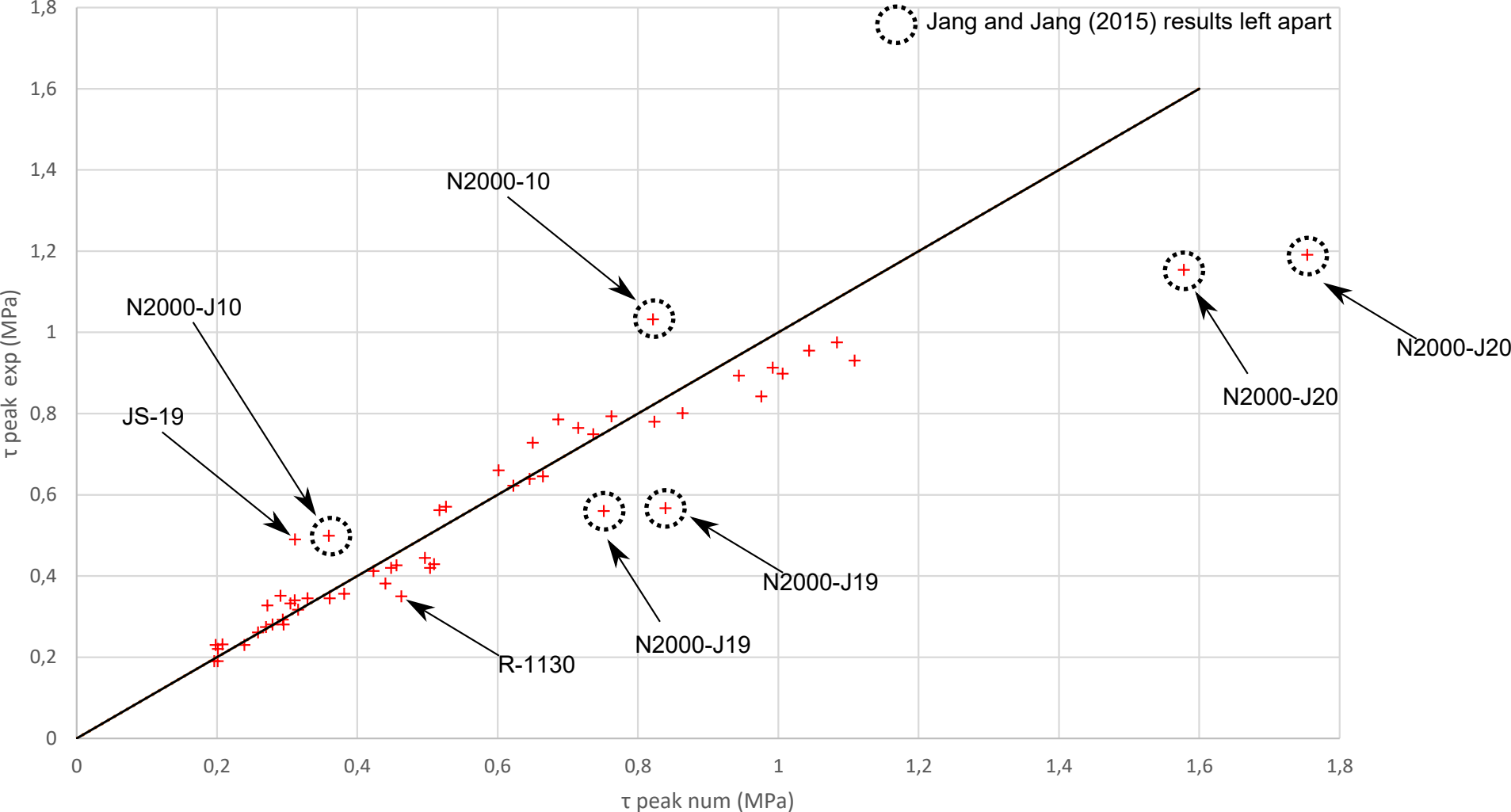
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- 100kPa-bis
- 200kPa
- 350kPa
- 600kPa
- 600kPa-bis

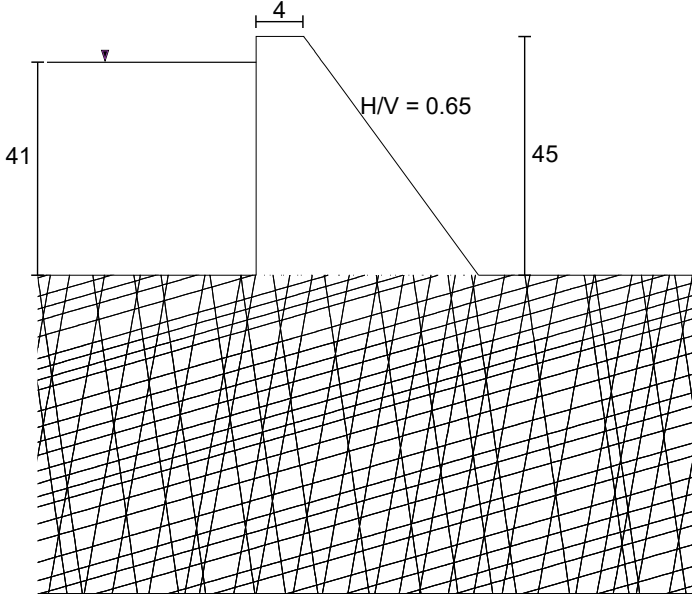
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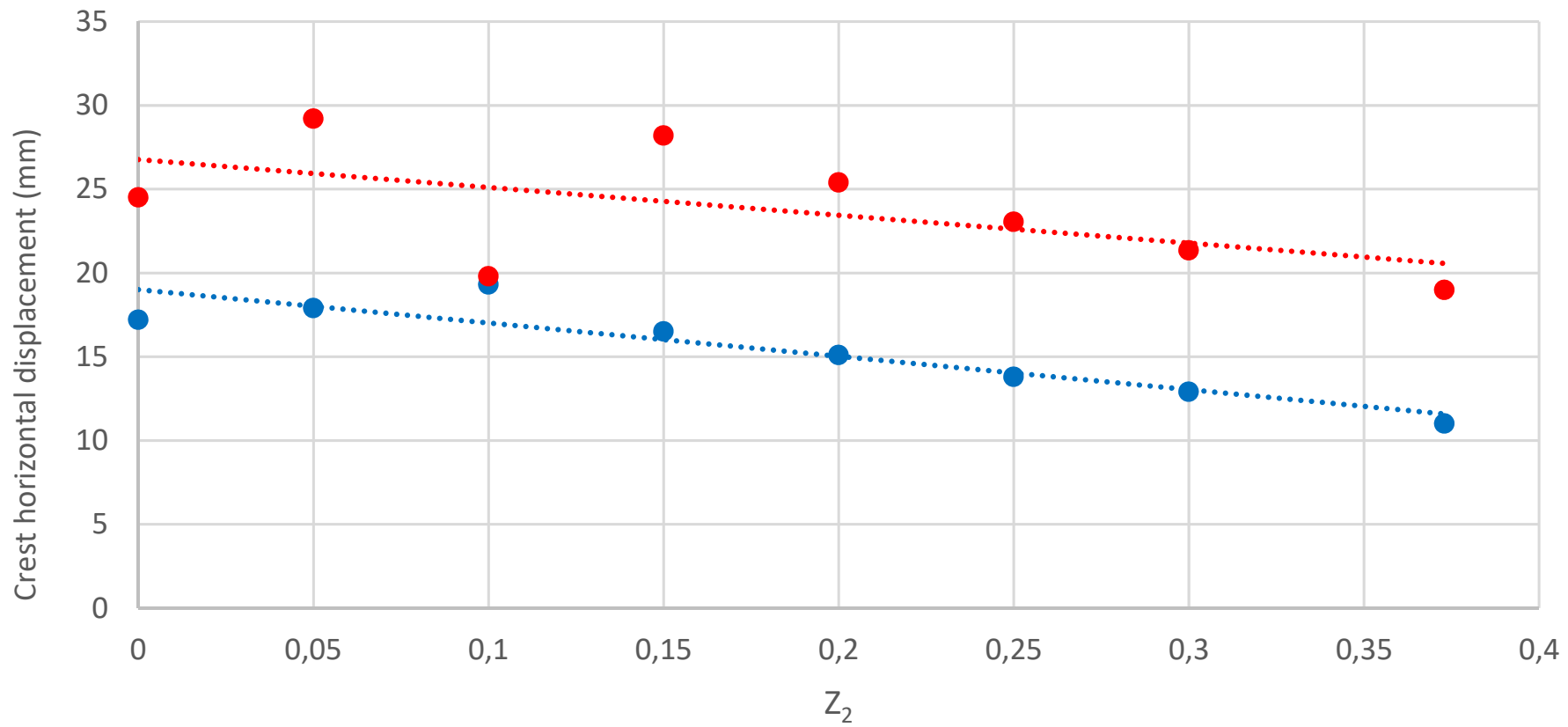
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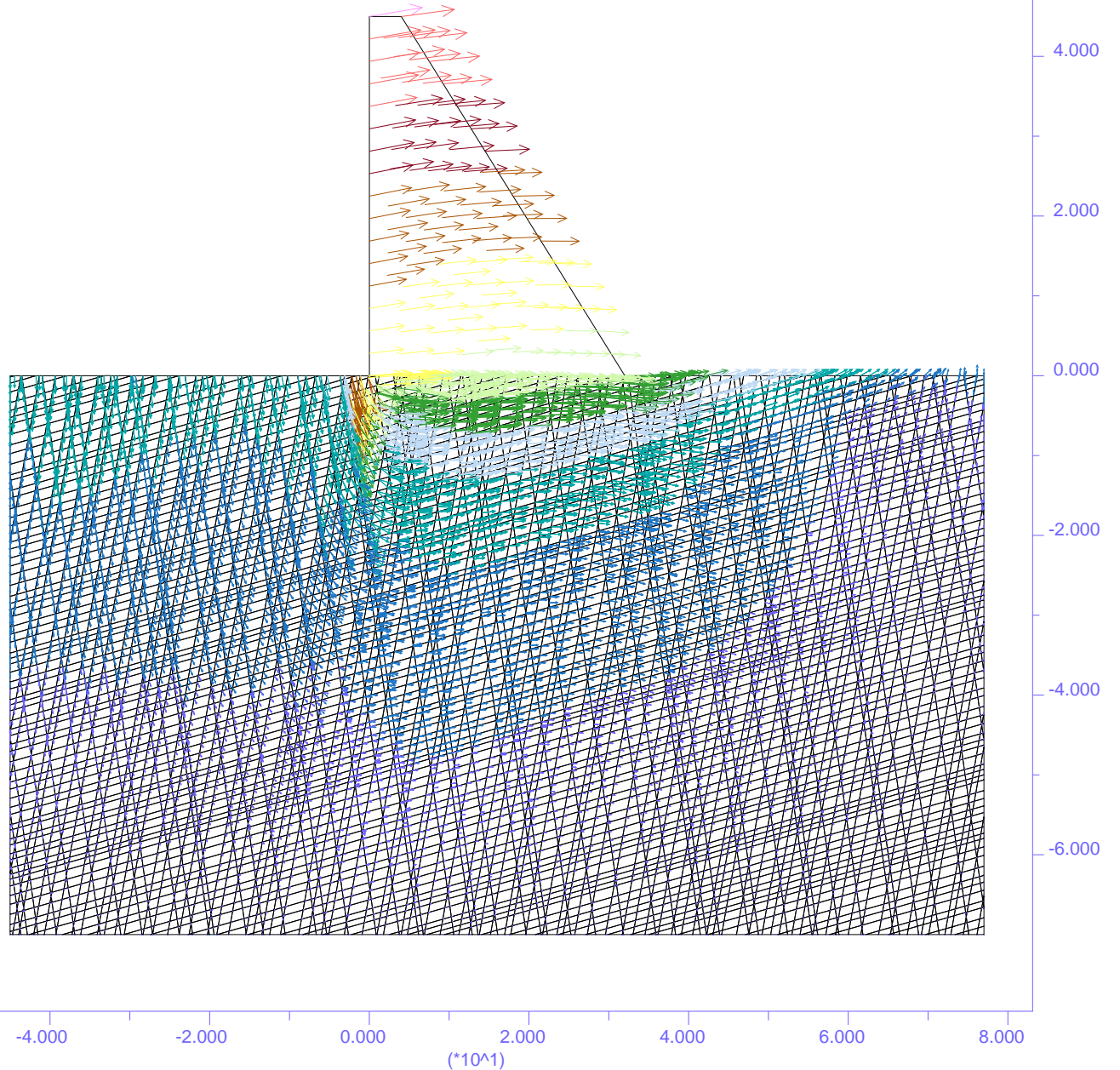
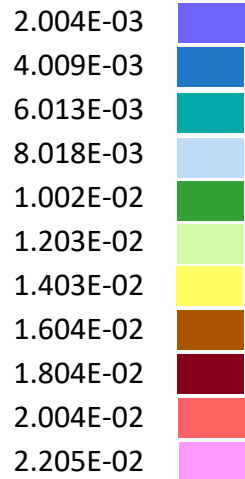
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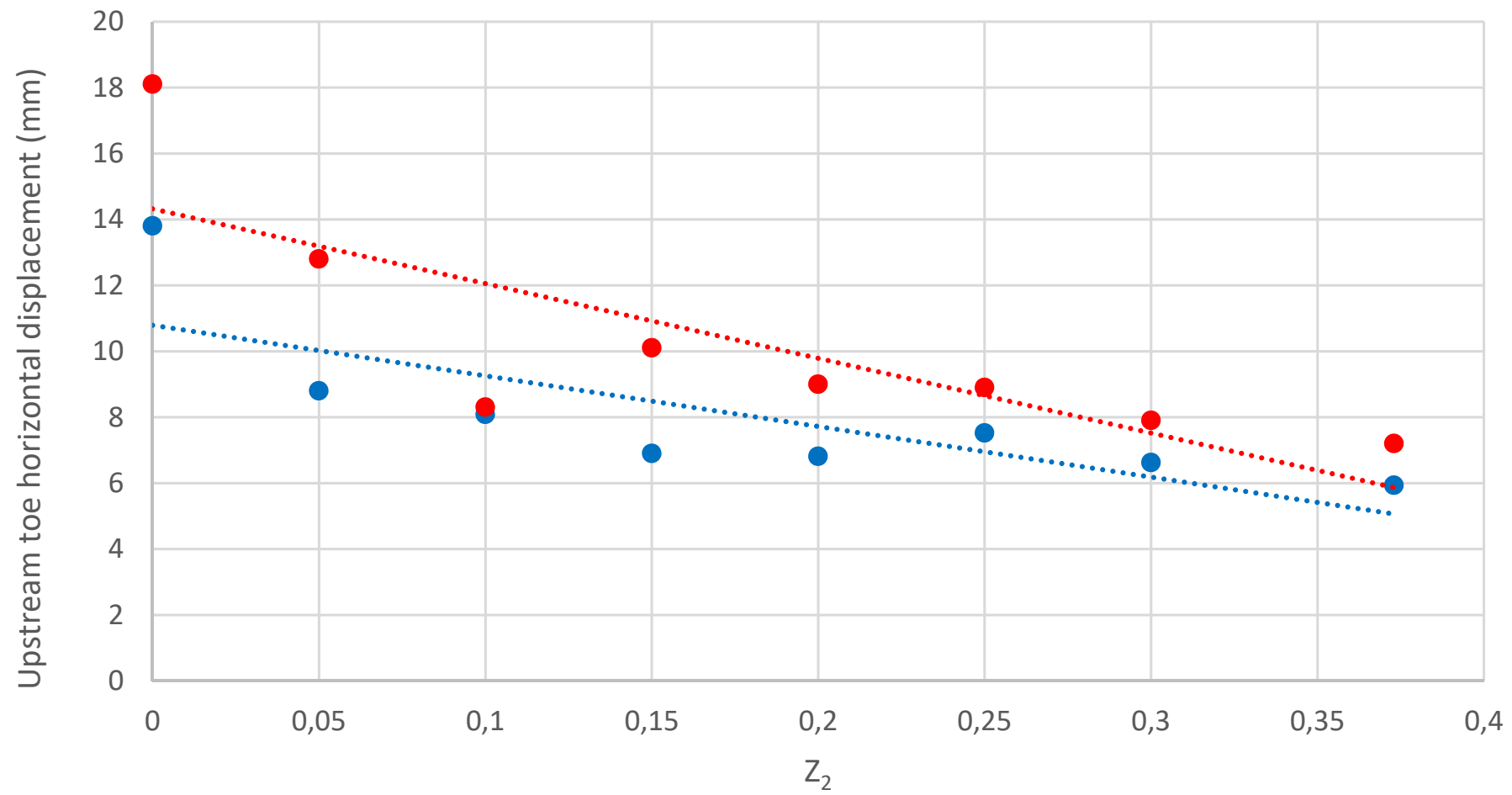
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Itasca Consulting Group, Inc.  
Minneapolis, Minnesota USA



1 **Integration of rock joint roughness into the Mohr-Coulomb shear**  
2 **behaviour model – Application to dam safety analysis**

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9

## 10 **ABSTRACT**

11           Based on a previous experimental study that we conducted on granite joint replicas  
12           including thirty direct shear tests, this article proposes to integrate the joint  
13           roughness in the Mohr-Coulomb shear behaviour model (MC). The model  
14           integrates joint roughness into constitutive stress-displacement relationships  
15           describing mechanical behaviour of rock joint. The shear strength of rock joint is  
16           assessed by the MC shear criterion that takes into account the apparent cohesion.  
17           This MC model integrating rock joint roughness component is validated against  
18           other experimental results from the literature. It is able to accurately predict the  
19           peak shear strength of an unbounded rock joint with an average relative error of  
20           7.9%. It is finally used to assess the role of joint roughness on the shear behaviour  
21           of a gravity dam foundation.

22

## 23 **KEYWORDS**

24           Apparent cohesion, DEM, Mohr-Coulomb model, rock joint, roughness, shear  
25           behaviour

26

27           **Introduction**

28   The shear strength of unbounded (cohesionless) rock joints have been extensively studied  
29   since the 1950s. For such a joint, it is widely accepted that shear strength and behaviour  
30   are controlled by many parameters such as joint roughness (Patton 1966; Barton and  
31   Choubey 1977; Kulatilake et al. 1995), joint interlocking (Zhao 1997a, 1997b), rock  
32   weathering (Barton and Choubey 1977; Nouailletas et al. 2017), joint mechanical  
33   properties (Ghazvinian et al. 2010; Rullière et al. 2021) and scale (Bandis et al. 1981;  
34   Tatone and Grasselli 2013; Buzzi and Casagrande 2018).

35           Several failure criteria have been developed to predict joint peak shear strength.  
36   The simplest one is the linear Mohr-Coulomb criterion that links the joint peak shear  
37   strength ( $\tau_{\text{peak}}$ ) to the normal load ( $\sigma_N$ ) and joint surface characteristics ( $C$  and  $\phi$ ). Barton  
38   and Choubey (1977) extended the joint surface characteristic parameter  $\phi$  of the Mohr-  
39   Coulomb criterion to take into account the effects of joint roughness and joint weathering  
40   on peak shear strength. Barton-Choubey criterion is well known in rock mechanics  
41   (ISRM 2014). This criterion introduces the joint roughness parameter (JRC) determined  
42   by visual comparison of the joint with a two-dimensional abacus. Because this abacus  
43   cannot cover the wide spectrum of roughness that a joint can exhibit, some authors  
44   consider that the JRC value might be prone to subjectivity (Hsiung et al. 1993; Beer,  
45   Stead, and Coggan 2002; D. Sow et al. 2016; D. Sow et al. 2017).

46           Later, Grasselli and Egger (2003) proposed a more complex criterion which  
47   adopts a three-dimensional view of the rock joint to estimate its peak shear strength. The  
48   criterion proposed by Grasselli and Egger (2003) considers that only the asperities facing  
49   the shear direction contribute to joint peak shear strength. Nevertheless, it was  
50   demonstrated that this criterion tends to overestimate the peak shear strength of smooth  
51   joints ((Xia et al. 2014). More recently, Zhang et al. (2016) proposed a 2D criterion that  
52   considers only a proportion of the asperities facing the shear direction. Wang and Lin

53 (2018) proposed a shear criterion that describes either the sliding or shearing mechanism  
54 effects on peak shear strength as a function of the normal stress. Others relevant  
55 references in the field of our work have to be cited also (Chen et al. 2022; Wang et al.  
56 2022; Zhang et al. 2022a; Zhan et al 2022b; Barton and Shen 2017; Wang et al. 2020).

57 Besides assessing the shear strength of joints, it is necessary to evaluate their pre-  
58 failure shear behaviour. To this end, several shear behaviour models that analytically  
59 describe the interactions between stresses and relative displacements of a joint subjected  
60 to shearing have been developed. Historically, the work of Goodman, Taylor, and Brekke  
61 (1968) is considered as the first development of numerical laws to model the shear  
62 behaviour of rock joints. It suggests that stresses are connected to displacements through  
63 mathematical linear elastic laws using constant stiffness, the shear stress being bounded  
64 by a maximum value that can be estimated using one of the criteria mentioned above.  
65 Later, considerable research improved this model either by considering that stiffness is  
66 not systematically constant during shearing (Bandis et al. 1983) or by taking into account  
67 the effect of joint damage (Cundall and Lemos 1988). The continuous improvement of  
68 computers has led to the development of even more complex and comprehensive shear  
69 behaviour models in recent decades. Duriez et al. (2011) developed a shear behaviour  
70 model with incrementally non-linear constitutive relations to describe the mechanical  
71 behaviour of infilled rock joints along a variety of loading paths. Oh et al. (2015) and Li  
72 et al. (2016) proposed a model that considers together and independently the contributions  
73 of large-scale roughness (waviness) and small-scale roughness (unevenness) of a joint to  
74 its shear behaviour.

75 For engineering applications such as rock slope stability and dam foundation  
76 design, many engineering guidelines suggest using the Mohr-Coulomb criterion to assess  
77 the shear strength of a rock joint (USBR 1987; US Army Corps of Engineers 1995; CFBR



78 2012; ISRM 2014; Federal Energy Regulatory Commission 2016). Mohr-Coulomb input  
79 parameters, cohesion  $C$  and friction angle  $\phi$ , are determined through experimental shear  
80 tests on small joint samples. For an unbounded rock joint, the cohesion obtained  
81 corresponds to the apparent cohesion concept, which depicts the roughness and joint  
82 interlocking effect on the shear strength (Amitrano and Schmittbuhl 2002; Rullière et al.  
83 2020). However, guidelines recommend a safe practice considering a low or null value of  
84 apparent cohesion, since its extrapolation to the scale of the structure is delicate. But this  
85 safe practice is conservative since the resistance characteristics of the joint due to its  
86 morphology (roughness, interlocking) are neglected.

87         The aim of this article is to integrate rock joint roughness components into the  
88 Mohr-Coulomb shear behaviour model which is chosen for its wide usage in engineering  
89 practice strength (USBR 1987; US Army Corps of Engineers 1995; CFBR 2012; ISRM  
90 2014; Federal Energy Regulatory Commission 2016), in order to take into account the  
91 role of joint roughness on joint shear behaviour. This integration aims to take into account  
92 the role of joint roughness on joint shear behaviour and is built up on the base of thirty-  
93 five experimental tests (direct joint shear tests and joint compression tests) and links joint  
94 roughness to joint shear behaviour and strength. The rock joint shear behaviour is  
95 described through stress-displacement laws. Joint failure state is expressed by the Mohr-  
96 Coulomb criterion including the apparent cohesion concept, to stick to the current practice  
97 of rock engineering. At the end, the MC shear behaviour model integrating rock joint  
98 roughness is validated with experimental data coming from rock mechanics literature.

99         The paper is divided into four parts. The first part presents the materials and  
100 methods (Part 1). In the second part, the MC shear behaviour model integrating rock joint  
101 roughness is developed and calibrated (Part 2). It consists in the development of empirical  
102 laws correlating the joint roughness to the shear behaviour model input parameters. These

103 empirical laws are then integrated in the MC model, so that it takes into account the role  
104 of joint roughness on joint shear behaviour. In the third part, the developed model is  
105 validated in the third part: model blind-predictions are faced to direct shear test results  
106 from rock mechanics literature in order to evaluate the model's accuracy (Part 3). In the  
107 last part, the MC shear behaviour model integrating rock joint roughness is used to  
108 demonstrate the key role of joint roughness on the shear behaviour of a gravity dam  
109 foundation (Part 4). All the numerical calculations presented in this paper were performed  
110 using the Discrete Element Method (DEM).

## 111 **1. Materials and Methods**

112 This part presents: i) the experimental data used to develop the empirical equations  
113 linking the joint roughness to the shear behaviour model input parameters, ii) the shear  
114 behaviour model used in the study, and iii) the DEM code in which the model will be  
115 implemented to predict the shear behaviour at the laboratory and gravity dam scales.

### 116 ***1.1. Experimental data used to develop the shear behaviour model***

117 The experimental data come from a previous work (Rullière et al. 2020) and are  
118 summarized in this article. Thirty direct shear tests on granite joint replicas were carried  
119 out in CNL conditions and under 0.1, 0.2, 0.35 and 0.6 MPa normal stress. Additional  
120 joint compression tests were conducted to characterize the joint normal stiffness. The tests  
121 were specifically performed for the research work presented in this paper.

122 It is specified in this introduction to the experimental data that our work does not  
123 take into account the scale effect between the sample tested in the laboratory and the real  
124 scale model. Several authors such as (Bandis et al. 1981) have worked on the scale effect  
125 and have shown the influence of sample size on shear strength. This question of scale

126 effect between laboratory and field is, in general, a universal problem of this kind of study  
127 that our research has not taken into account.

### 128 *1.1.1. Joint roughness statistical indicator ( $Z_2$ )*

129 Many studies have been conducted to define the roughness concept because it is  
130 considered to be the main parameter governing the shear behavior of rock discontinuities.  
131 Barton and Choubey (1977) developed the joint roughness coefficient (JRC) to estimate  
132 joint roughness using a visual comparison of 10 standard profiles. With the advancement  
133 of non-contact methodologies used to scan rock joint surfaces, new parameters have been  
134 developed to assess roughness. In this context, the 2D directional parameter  $Z_2$  is widely  
135 accepted and used in the rock mechanics community as a non-subjective roughness  
136 indicator (Tse and Cruden 1979; Kulatilake et al. 1995; Tatone and Grasselli 2010;  
137 Magsipoc, Zhao, and Grasselli 2019; Ram and Basu 2019). Indeed, for a joint profile, the  
138 parameter  $Z_2$  is a 2D directional parameters that describes the local topographic slope of  
139 the joint profile.  $Z_2$  can be seen as a topographical slope and corresponds to the root-  
140 mean-square of the first derivative (Myers 1962).  $Z_2$  can be calculated as:

$$141 \quad Z_2 = \left[ \frac{1}{L} \sum_1^{n-1} \left( \frac{y_{i+1} - y_i}{x_{i+1} - x_i} \right)^2 \right]^{\frac{1}{2}} \quad (1)$$

142 Where  $L$  is the length of the joint profile,  $x_i$  and  $y_i$  are the coordinates of the  
143 discretized joint profile (0.5 mm sampling interval).

144 The calculation of  $Z_2$  requires only a topographic scan of the joint surface and, in  
145 comparison to other roughness indicators, it is rather simple and non-time-consuming to  
146 implement the calculation of  $Z_2$  in an algorithm.

147 Five granite joints (J0, J1, J2, J3, J4) with different roughness's ( $Z_2$  from 0 to  
148 0.373) were selected to produce the replicas. Prior to replica production, the granite joints  
149 were scanned with a non-contact laser profilometer (Figure 1). Data from the scans were

150 then implemented in a specifically designed algorithm linking the surface morphology to  
151 the roughness statistical parameter  $Z_2$  (Rullière et al. 2020). Table 1 shows the  $Z_2$  value  
152 for each of the five granite joints used to produce the replicas.

153 [FIGURE 1 HERE]

154 [TABLE 1 HERE]

### 155 1.1.2. Direct shear test

156 Following the production of the replica (six replicas per selected roughness), thirty direct  
157 shear tests were conducted in CNL conditions. Twenty direct shear test results, carried  
158 out on J0, J1, J2, J3 and J4 under 0.1, 0.2, 0.35 and 0.6-MPa normal stress come from the  
159 results of Rullière et al. (2020). For this work, ten additional direct shear tests were carried  
160 out for J0, J1, J2, J3 and J4, under 0.1 and 0.6-MPa. These new results provide more shear  
161 tests results over the 0.1 to 0.6-MPa interval and strengthen the Mohr-Coulomb failure  
162 envelope plot. The normal stress levels used in this study correspond to those observed  
163 in dam foundations or other civil engineering works (Sow et al. 2017). Each replica was  
164 used for a single direct shear test.

165 Figure 2 shows the shear strength versus shear displacement curves for all the  
166 direct shear tests used in this study. The key data are summarized in Table 2. It classically  
167 appears from the direct shear tests results that the rougher the joint, the higher the shear  
168 strength (peak or residual).

169 [FIGURE 2 HERE]

170 [TABLE 2 HERE]

171 Figure 3 depicts the Mohr-Coulomb failure envelopes plotted from the  
172 experimental direct shear test results for the peak and residual shear strengths. Apparent  
173 cohesion and friction angle values at both peak and residual stages are shown in Table 1.  
174 Again, it appears that the rougher the joint, the higher the apparent cohesion and friction

175 angle values. Since the results were obtained on an unbounded rock joint replica, it was  
176 considered that the apparent cohesion could be mobilized at peak stage only (EPRI 1990;  
177 Rullière et al. 2020). Therefore,  $C_{APP \text{ Residual}} = 0\text{-kPa}$ .

178 [FIGURE 3 HERE]

179 The shear stiffness  $K_s$  was calculated for shear stresses ranging from 25 to 75%  
180 of the peak shear strength (Kumar and Verma 2016). This precaution allows setting aside  
181 the curved areas of the shear strength-shear displacement curve (initial loading and peak  
182 phase). Table 1 presents the  $K_s$  mean value obtained for all the joints tested. It appears  
183 that the rougher the joint, the higher the shear stiffness.

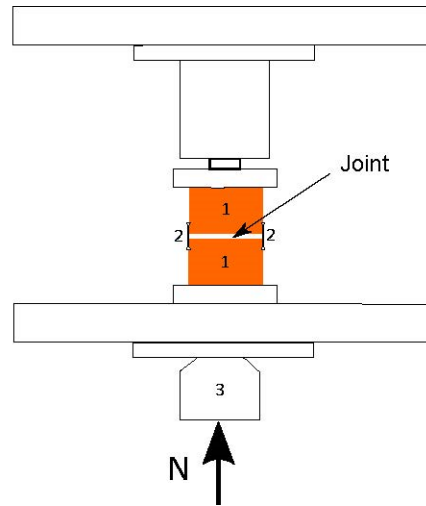
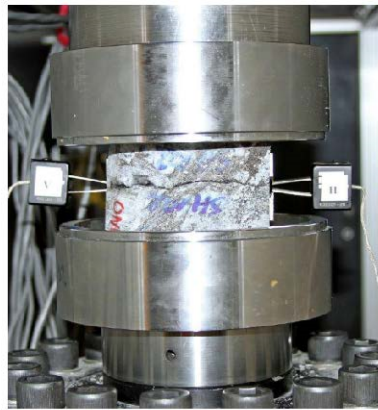
184 Figure 4 shows normal versus shear displacement curves for all the direct shear  
185 tests (Rullière et al. 2020). It is verified that the dilatant behaviour depends on both of the  
186 joint roughness and the normal stress imposed during the shear tests. As shown in Figure  
187 4, a rougher joint lead to larger normal displacement during a direct shear test in CNL  
188 conditions. The dilatancy angle ( $dN$ ) is calibrated from the maximal value of the  $\Delta v / \Delta u$   
189 ratio (usually around the peak shear displacement value). Table 1 shows the  $dN$  mean  
190 value obtained for all the joints tested. As expected, the rougher the joint, the higher the  
191 dilatancy angle.

192 [FIGURE 4 HERE]

### 193 1.1.3. Compression test results

194 Joint normal stiffness ( $K_N$ ) assessment requires specific experimental tests called joint  
195 compression test (Bandis et al. 1981; Fan, Cao, and Tang 2018). Such a test involves  
196 recording the relative normal displacements of a rock joint under a given normal load  
197 (Figure 5).

198 [FIGURE 5 HERE]



199

200 In practice, several loading-unloading cycles are performed to determine the  
201 normal stiffness of a rock joint (Bandis et al. 1983; Marache 2002). The first cycle allows  
202 the joint to be fully interlocked, so that the deformations recorded during the following  
203 cycles correspond to the joint's deformations only and not to a change of joint  
204 interlocking.

205 Five joint compression tests were performed, for each granite joint. The joints  
206 were subjected to an increasing normal stress, up to 30% of the compressive strength  
207 matrix of the material, prior to a progressive discharge. The charge-discharge cycles were  
208 repeated three times and the  $K_N$  was measured along the last loading cycle. Figure 6  
209 shows the normal stress versus normal displacement curves obtained from the joint  
210 compression tests. The  $K_N$  value of each joint is presented in Table 1. If the  $K_N$  values  
211 are compared to joint roughness, it seems that the rougher a joint, the higher the  $K_N$ .

212

[FIGURE 6 HERE]

### 213 ***1.2. Shear behaviour model used in the study***

214 The shear behaviour model used in this study follows an elastic-plastic formulation based  
215 on the work of Goodman, Taylor, and Brekke (1968). As a joint is subjected to a shear  
216 stress, the response of the joint is initially exclusively elastic. The displacements can be

217 correlated to the normal or shear stress variations through constant joint normal or shear  
218 stiffness and classical linear elastic laws:

$$219 \quad \begin{pmatrix} d\sigma_N \\ d\tau \end{pmatrix} = \begin{pmatrix} K_N & 0 \\ 0 & K_S \end{pmatrix} \cdot \begin{pmatrix} d\Delta_V \\ d\Delta_U \end{pmatrix} \quad (2)$$

220 Where  $d\sigma_N$  is the normal stress variations,  $d\tau$  is the shear stress variation,  $K_N$  is the joint  
221 normal stiffness,  $K_S$  is the joint shear stiffness,  $d\Delta_U$  is the shear displacement increment  
222 and  $d\Delta_V$  is the normal displacement increment.

223 A direct shear test conducted in Constant Normal Load (CNL) conditions is  
224 piloted by constant tangential displacement increments ( $d\Delta_U$ ). Therefore, the shear stress  
225 progressively increases according to (2). On the contrary, the normal stress increment is  
226 null. In this type of test, this means that the model assumes no normal displacement  
227 increments ( $d\Delta_V$ ) during the elastic stage.

228 The elastic stage extends until the shear stress  $\tau$  reaches the shear strength value  
229  $\tau_{\text{peak}}$  defined by the Mohr-Coulomb criterion which serves as both a (constant) yield  
230 surface and plastic limit condition in the  $(\tau; \sigma_N)$  plane, in the absence of elasto-plastic  
231 hardening. As rock joints are unbounded, the cohesion of the Mohr-Coulomb criterion  
232 corresponds to the apparent cohesion,  $C_{\text{app}}$ , as presented in the introduction.

$$233 \quad \tau = C_{\text{app}} + \sigma_N \cdot \tan(\varphi) \quad (3)$$

234 Where  $\tau$  is the shear stress,  $\sigma_N$  is the normal stress,  $C_{\text{app}}$  is the apparent cohesion and  $\varphi$  is  
235 the friction angle.

236 As soon as the peak shear stress is reached, the shear behaviour model considers  
237 that the joint is in a failure phase: the elastic stage is completed and the plastic stage  
238 begins. The shear stress decreases instantaneously to the residual and constant value  
239  $\tau_{\text{residual}}$ . This choice of a brittle behaviour, as opposed to a gradual softening, is chosen for  
240 simplicity, in line with engineering practices.

241 In the plastic stage, a flow rule governs the dilatancy whereby normal  
242 displacement variations are correlated to shear displacement variations and to the joint  
243 dilatancy angle:

$$244 \quad d\Delta_v = d\Delta_u \cdot \tan(dN) \quad (4)$$

245 Where  $d\Delta_u$  is the shear displacement increment,  $d\Delta_v$  is the normal displacement  
246 increment and  $dN$  the dilatancy angle.

### 247 ***1.3. The Discrete Element Method***

248 A rock mass can be considered as a set of rocky blocks separated from each other by  
249 joints. For such a set, a mechanical stress will lead to displacements that are almost  
250 exclusively due to the displacements along the joints (the rocky block deformations are  
251 considered as insignificant). This consideration exactly matches the DEM hypothesis:  
252 blocks are considered as infinitely stiff and the displacements are exclusively due to block  
253 contacts.

254 The DEM method and UDEC software (Itasca 2019) were used in this paper to  
255 perform the different numerical calculations, either to simulate laboratory direct shear  
256 tests in CNL conditions or to assess the shear behaviour of a gravity dam founded on a  
257 complex jointed rock mass. In UDEC, the hydro-mechanical behaviour of a set of blocks  
258 and joints is simulated by adjusting the joint properties (mechanical properties and  
259 behaviour models).



260 **2. Calibration of the MC shear behaviour model integrating rock joint**  
261 **roughness**

262 ***2.1. Equations linking joint roughness to shear behaviour model input***  
263 ***parameters***

264 The aim of this section is to develop empirical equations that correlate the joint roughness  
265 to the input parameters of the MC shear behaviour model integrating rock joint roughness.  
266 These input parameters are  $C_{app\ peak}$ ,  $\phi_{peak}$ ,  $C_{app\ residual}$ ,  $\phi_{residual}$ ,  $dN$ ,  $K_N$  and  $K_S$  (see section  
267 1 for more details). The experimental data presented in Part 1 are used and three kinds of  
268 mathematical laws are studied: linear, power, and exponential laws. For each kind of law,  
269 the least square method was used to determine coefficients  $a$ ,  $b$  and  $c$  :

270 
$$P = a \cdot Z_2 + b \quad (5)$$

271 
$$P = a \cdot Z_2^b + c \quad (6)$$

272 
$$P = a \cdot e^{Z_2 \cdot x} + c \quad (7)$$

273

274 Where  $a$ ,  $b$  and  $c$  are empirical coefficients and  $P$  stands for any model parameters  
275 among  $C_{app\ peak}$ ,  $\phi_{peak}$ ,  $C_{app\ residual}$ ,  $\phi_{residual}$ ,  $dN$ ,  $K_N$  and  $K_S$ .

276 For the sake of concision, only the power equations are presented. Indeed, it  
277 appeared during the study that the power equations (6) were able to correlate the joint  
278 roughness to the shear behaviour input parameter with great precision. In comparison to  
279 linear and exponential equations, the power equations had the highest coefficient of  
280 determination ( $R^2$ ). However, the reader will find linear and exponential equations in  
281 Appendix A.

282 Based on the data presented in Table 1, the power law equations describing the  
283 shear behaviour and shear failure state of the joint are developed. Power law coefficients

284  $a$ ,  $b$  and  $c$ , which correlate the roughness to the peak apparent cohesion, peak friction  
285 angle, residual friction angle, dilatancy angle, and normal and shear stiffness, are  
286 presented in Table 3. Figures 7 to 9 compare the power law predictions to the  
287 experimental data.

288 The values of coefficients  $a$ ,  $b$  and  $c$  and the equations developed in this section  
289 are valid only for the following conditions:

- 290 • the rock joint roughness is described by the  $Z_2$  statistical parameter.  $Z_2$  ranges  
291 between 0 (smooth joint) and 0.373 (rough joint);
- 292 • the rock joint is subjected to a normal load ranging from 0.1 to 0.6-MPa.

293 [TABLE 3 HERE]

294 [FIGURE 7 HERE]

295 [FIGURE 8 HERE]

296 [FIGURE 9 HERE]

## 297 *2.2.Numerical direct shear tests using the MC shear behaviour model*

298 Once the shear behaviour model was established, thirteen numerical direct shear tests  
299 were performed under the exact same conditions of normal load and roughness as the  
300 experimental direct shear tests presented in the previous section. In other words, data  
301 provided by the numerical shear behaviour model were compared to the experimental  
302 data used to develop it, to control if the numerical results fit the experimental observations  
303 and if the calibration of the numerical model had been carried out well.

304 Using UDEC software (Itasca 2019), the rock joint is represented by the contact  
305 between two infinitely rigid blocks. While the lower block is fixed, the upper block is  
306 subjected to both a normal load and a constant horizontal displacement rate. In the DEM  
307 model, the lower block length is longer than the upper block to avoid any toppling of the

308 upper block. For the numerical direct shear tests, the normal load and horizontal  
309 displacement rate were set as equal to the experimental values applied during the  
310 experimental direct shear tests (normal load ranges from 0.1 to 0.6-MPa and the  
311 horizontal displacement rate is 0.1 mm/min to insure quasi-staticity). The numerical  
312 model also required the roughness of the joint (depicted by  $Z_2$ ).

313 Figure 10 and Figure 11 show the shear stress versus horizontal displacement  
314 curves and normal displacement versus horizontal displacement curves. The dotted lines  
315 correspond to the experimental data, while the full lines correspond to the numerical  
316 results.

317 From Figures 10 and 11, it appears that despite its bilinear aspect, the numerical  
318 model manages to describe the rock joint shear behaviour efficiently. This is particularly  
319 true during the peak phase, and to a lesser extent for the residual phase (end of test).  
320 Nonetheless, as shown in Figure 10, the post-peak joint behaviour is not realistically  
321 described by the shear behaviour model since as soon as the peak shear strength is  
322 reached, the release of the shear stress is instantaneous, while the experimental tests show  
323 a progressive decrease of the shear stress towards a residual shear stress value. Figure 11  
324 shows that the shear behaviour model satisfactorily reproduces the dilatant behaviour of  
325 a rock joint, although there is no contraction stage and for the shear behaviour model, the  
326 dilatancy starts as soon as the shear stress reaches its peak (these are related to the  
327 equations that govern the MC shear behaviour model, see Part 1). However, it should be  
328 noted that in rock-mechanics engineering practice, the post-peak shear behaviour has not  
329 been extensively studied and the MC model approximations could be sufficient for a vast  
330 majority of applications.

331 Figure 12 displays the model peak and residual shear strength and contrasts it with  
332 the experimental recorded values. For the peak stage, the results predicted by the model

333 are very close to the experimentally recorded values (generally, the difference is less than  
334 10%). For the residual phase, the differences are slightly larger, but can be explained by  
335 the numerous jumps present on the experimental curves (Figure 2).

336

337 [FIGURE 10 HERE]

338 [FIGURE 11 HERE]

339 [FIGURE 12 HERE]

340 The shear behaviour model can be considered as calibrated, as the numerical shear  
341 test results matches the experimental results.

342 The next step consists of a validation in which we aim to assess the ability of the  
343 shear behaviour model to predict the rock joint shear behaviour of all rock joints. To this  
344 end, the aim of the next section is to compare the model blind-predictions to other  
345 published experimental direct shear test results that were not used to develop the  
346 numerical model.

### 347 **3. Validation of the MC shear behaviour model with literature data**

#### 348 ***3.1. Published data used for the validation step***

349 We consider four articles in the rock mechanics literature that present experimental direct  
350 shear tests carried out in the validity conditions of the shear behaviour model developed  
351 in this study. In these four studies, the joint roughness indicator ( $Z_2$ ) ranges between 0  
352 and 0.373 and joints were subjected to normal load ranging from 0.1 to 0.6-MPa during  
353 the direct shear tests (Lee, Park, and Song 2014; Jang and Jang 2015; Li et al. 2018; Ram  
354 and Basu 2019). Please note that the data from these studies were not used to develop the  
355 numerical model previously described.

356 Ram and Basu (2019) studied the shear behaviour of unfilled natural rock joints  
357 with reference to the weathering grade of the joint. Due to the different weathering grade  
358 used by the authors, only seven direct shear tests results could be used (Table 4). These  
359 shear tests were carried out on smooth rock joints ( $Z_2$  ranges from 0.057 to 0.130) under  
360 a normal stress of about 0.2-MPa.

361 Jang and Jang (2015) conducted more than 180 direct shear tests on rock joint  
362 replicas. Forty direct shear test results could be used directly in this research paper:  $Z_2$   
363 ranges from 0.082 to 0.351 and the normal stresses levels are 0.2 and 0.53-MPa MPa  
364 (Table 4).

365 From Lee, Park, and Song (2014), six direct shear tests results were extracted:  $Z_2$   
366 ranges from 0.142 to 0.253 and the normal stress levels range from 0.1 to 0.5-MPa (Table  
367 4).

368 Lastly, in Li et al. (2018), only one direct shear test was conducted in the  
369 conditions presented above ( $Z_2 < 0.373$  and the normal load from 0.1 to 0.6-MPa).

370 In brief, this amounts to fifty-four direct shear results selected from several  
371 published rock mechanics papers that were used to validate the shear behaviour model  
372 developed in this paper. The selected direct shear tests were carried out at various normal  
373 stress levels (from 0.1 to 0.53-MPa) on natural rock joints or on rock joint replicas that  
374 exhibited several levels of roughness ( $Z_2$  ranging from 0.057 to 0.351). All of the selected  
375 papers gave the peak shear strength, whereas only eleven residual shear strengths were  
376 available.

### 377 **3.2.Results**

378 Table 4 and Figure 13 compare the experimental direct shear tests results from literature  
379 to the shear behaviour model blind-predictions. In Table 4, the relative difference between  
380 experimental results and model blind-predictions are also given.



406 with the ISRM (1978) description. In Rullière et al. (2020), the granite joints exhibited  
407 mainly sliding mechanisms during the direct shear tests. On the other hand, in the study  
408 of Jang and Jang (2015), the rock joints came from different rock types including granite  
409 and schist joints, which are described being “*very rough and stepped*” in accordance with  
410 the ISRM (1978). Very rough or stepped asperities will tend to gather the shear stresses  
411 during a direct shear test, and schist joints will mainly be damaged by shearing  
412 mechanisms. The reader should note that the direct shear tests conducted in (Lee, Park,  
413 and Song 2014; Li et al. 2018; Ram and Basu 2019) were carried out on granite and gneiss  
414 joints. In Jang and Jang (2015), as the rock type was not linked to the joint identification  
415 code, it is therefore impossible to know which rock joint replica came from granite or  
416 schists joints. Nevertheless, we attributed the large difference between the experimental  
417 data and the model blind-predictions to the rock type (schist), as explained above. If the  
418 specific large differences of Jang and Jang (2015) are excluded (see dashed circles in  
419 Figure 13), the mean relative error between the experimental data from the rock  
420 mechanics literature and the model blind-predictions is 7.9%.

421       Regarding the residual shear strength ( $\tau_r$ ), a mean relative difference of 23% was  
422 obtained between the experimental results and the model blind-predictions (Table 4).  
423 Although this mean relative difference is based on a few experimental results, as not all  
424 the selected papers gave the residual shear strength, this gap between the experimental  
425 data and the model blind-prediction can be explained by the following reasons:

- 426       • The model approach to predicting the residual shear strength is very conservative.  
427       The shear stress decreases suddenly to its residual value once the peak is reached  
428       whereas during an experimental direct shear test, the shear stress decreases  
429       gradually to its residual value,

430 • Also, during this progressive shear stress decrease observed in the experimental  
431 tests, some shear stress jumps could be observed, very likely related to sheared  
432 material that interfered between the joint walls.

433 Finally, it appeared that the MC shear behaviour model integrating rock joint  
434 roughness was able to accurately predict the experimental peak shear strength of various  
435 direct shear tests published in the rock mechanics literature. For some specific cases, the  
436 blind-predictions were quite far from the experimental results, but in our opinion these  
437 differences can be explained by the rock type. Indeed, the shear behaviour model was  
438 developed using granite joints while some of the data presented were related to schist  
439 joints.

#### 440 **4. Application of the MC shear behaviour model to a gravity dam**

441 Gravity dams are mainly exposed to shear mechanisms: under the combination of  
442 different actions, shearing can take place at the rock-concrete interface or at rock mass  
443 foundation joints. International gravity dam design guidelines suggest using the Mohr-  
444 Coulomb criterion to assess the shear strength of a rock joint of such structures (USBR  
445 1987; US Army Corps of Engineers 1995; CFBR 2012; Federal Energy Regulatory  
446 Commission 2016).

447 Part 3 showed that the shear strength and behaviour of a rough rock joint subjected  
448 to low normal stresses could be accurately estimated by the numerical shear behaviour  
449 model developed, which takes into account joint roughness (developed in Part 2). In this  
450 Part 4, we wish to highlight: i) that the shear behaviour model developed previously could  
451 be used at the dam scale, and ii) the influence of the rock joint roughness on the shear  
452 behaviour of a gravity dam, subjected to different load cases.



453 ***4.1.Presentation of the numerical gravity dam model and hypothesis***

454 The gravity dam studied is founded on a granitic rock mass from Canadian shield,  
455 intersected by three sets of joints that are prone to discontinuities in the displacement  
456 field, i.e. finite relative displacements across themselves. Among these sets, one is sub-  
457 horizontal and two sub-vertical, presenting dips of  $16^\circ$  N  $0^\circ$ ,  $80^\circ$  N  $0^\circ$  and  $82^\circ$  N  $180^\circ$ ,  
458 respectively. The joints of the same set are spaced according to a uniform distribution,  
459 whose standard deviations and mean values are given in Table 5. The persistence of the  
460 joints was considered as fully persistent (default preset). This agrees with the observations  
461 of the drillings carried out on the dam site, in the context of a dam founded on the  
462 Canadian shield where the persistence is generally very high.

463

464 All the joints exhibited the same roughness characteristics, described by a  $Z_2$  value. In  
465 the following calculations, the  $Z_2$  value was increased from zero (depicting a smooth  
466 joint) to 0.373 (depicting a very rough rock joint). It was also assumed that the rock mass  
467 joints are very long compared with the dam size and they can be considered as persistent  
468 at the numerical study scale. In other words, the joints we that the joints cut the entire  
469 rock foundation mass. There is therefore no increase in the length of the joints. On the  
470 other hand, the joints are free to open. The granite density was set to  $2500 \text{ kg/m}^3$ .

471 [TABLE 5 HERE]

472 The specifications of the case study gravity dam are:

- 473
- 474 • Height of 45m and an operating level of 41 m;
  - 475 • The width at the head of the structure is 4 m;
  - 476 • The upstream face is vertical, while the downstream batter (H/V) is 0.65;
  - The width at the toe is 32 m.

477 The dam is fully made of concrete (density of 2350 kg/m<sup>3</sup>) and it is assumed that  
478 its construction was of good quality: rock-foundation and construction interfaces were  
479 treated. In other words, excellent adhesion can be considered for concrete-concrete  
480 construction joints and rock-concrete interfaces. Therefore, relative displacements can  
481 only occur in the rock mass, along the rock joints described previously. For the sake of  
482 convenience, it was also assumed that the gravity dam was not equipped with any  
483 drainage system. Figure 14 shows a schematic view of the gravity dam modelled on its  
484 granitic rock mass foundation.

485 [FIGURE 14 HERE]

486 The numerical model is built using UDEC software (Itasca 2019) with a Discrete  
487 Element Method (DEM) approach. The DEM makes it straightforward to describe the  
488 discontinuity of displacements across joints, while a continuous-based FEM approach  
489 would require to adopt a more complex model, typically XFEM (Moës et al., 1999). In  
490 this DEM model, the particles are blocks of different sizes and shapes that come from  
491 three sets of discontinuities that cut the granitic massif. Numerical damping was used  
492 with a 0.8 coefficient and the time step was fixed at 30  $\mu$ s to avoid divergence of the  
493 explicit dynamic scheme. The boundaries of the numerical model were fixed in all  
494 directions and located at a distance of at least five times the structure height to avoid any  
495 interferences.

496 Different load cases were considered:

- 497 (1) Normal Operating Conditions: the reservoir is filled;  
498 (2) Extreme Conditions: the reservoir is filled and the dam is subjected to a seismic  
499 solicitation.

500 In Normal Operating Conditions, the dam is only subjected to mechanical actions  
501 related to the presence of water upstream of the structure (water pressure on the upstream

502 face of the dam, uplift). The reservoir is instantly filled with water from the natural  
503 foundation ground to the normal operating level (41 m of water upstream, 0 m of water  
504 downstream). The hydrostatic pressure field is imposed on the gravity dam rock  
505 foundation joints and assumed to be constant in time.

506 The seismic calculation is performed using a simple pseudo-static method, as  
507 suggested by French regulations. The values of the horizontal ( $a_h$ ) and vertical  
508 acceleration ( $a_v$ ) are:  $a_h = 2.0 \text{ m/s}^2$  and  $a_v = 1.8 \text{ m/s}^2$ .

509 For the two loading cases considered, the analysis of the behaviour of the structure  
510 is carried out based on horizontal and vertical displacements of: i) the dam downstream  
511 face crest, and ii) the dam foundation at a depth of 3 m below the upstream toe.

#### 512 ***4.2. Results and discussion***

513 Table 6 shows the displacements recorded during the numerical modelling of a gravity  
514 dam subjected to load levels. From Table 6, it appears that the rock joint roughness has  
515 an important effect on the horizontal displacements recorded at the dam crest or in its  
516 foundation. However, the effect of rock joint roughness on the vertical displacements is  
517 not obvious.

518 For smooth rock foundation joints ( $Z_2 = 0$ ), when the dam is filled and without  
519 seismic solicitation, the horizontal and vertical displacements recorded at the dam crest  
520 reach respectively a value of 17.2 mm and -0.74 mm. When seismic solicitation is applied,  
521 the displacements at the dam crest increase and reach the values of 24.5 mm for the  
522 horizontal displacement and 2.21 mm for the vertical displacement (Table 6).

523 For the rough rock foundation joints ( $Z_2 = 0.373$ ), the horizontal displacements  
524 observed at the crest under normal operations and seismic conditions are reduced in  
525 comparison to smooth joints. Under normal operating conditions, the horizontal dam crest  
526 displacements reach the value of 11.00 mm maximum versus 17.2 mm for smooth joints,

527 i.e. a difference of 36% (Table 6). Under seismic conditions, the same trend can be  
528 observed: horizontal dam crest displacements reach a value of 19.00 mm versus 24.5 mm  
529 for smooth joints, i.e. a difference of 22% (Table 6). Figure 15 clearly shows that the  
530 horizontal displacements recorded at the dam crest (under normal operating and seismic  
531 conditions) decrease linearly when the rock joint roughness increases.

532 The largest displacements were observed at the crest of the structure; they are the  
533 consequence of displacements of the rock discontinuities located inside the rock mass  
534 foundation, as depicted in Figure 16. In Table 6 and Figure 17, it can also be seen that as  
535 the rock joint roughness increases, the horizontal displacement recorded below the dam  
536 upstream toe decreases linearly.

537 [TABLE 6 HERE]

538 [FIGURE 15 HERE]

539 [FIGURE 16 HERE]

540 [FIGURE 17 HERE]

541 Therefore, the application of the MC shear behaviour model integrating rock joint  
542 roughness to the gravity dam scale illustrates the important role of rock joint roughness  
543 in the shear behaviour of the dam. This influence of roughness was considered through  
544 the shear behaviour model input parameters. It appeared that displacement linearly  
545 decrease with an increase of rock foundation joint roughness ( $Z_2$  value), either at dam  
546 crest or in its foundation (normal operation and extreme conditions).

## 547 **Conclusion**

548 The aim of this study was to develop a practical shear behaviour model, based on the  
549 Mohr-Coulomb model, which depicts the role of joint roughness on joints shear  
550 behaviour. The MC shear behaviour model integrating rock joint roughness included

551 constitutive stress-displacement laws. The model shear failure corresponded to the Mohr-  
552 Coulomb criterion including apparent cohesion.

553 The shear behaviour model was developed based on more than thirty-five  
554 experimental tests (direct shear tests and compression tests). Power laws that correlated  
555 the joint roughness parameter  $Z_2$  to the shear behaviour input parameter were developed  
556 (calibration step, Part 2). The model was then validated by comparing its blind-  
557 predictions to published experimental data (validation step, Part 3). The results indicated  
558 that the MC shear behaviour model integrating rock joint roughness was able to predict  
559 the peak shear strength of an unbounded rock joint with an average relative error of 7.9%).  
560 The shear behaviour model developed was found to be strongly influenced by the rock  
561 type.

562 Lastly, the shear behaviour model developed was applied at full scale (gravity  
563 dam) to highlight the role of rock joint roughness in dam shear behaviour. This practical  
564 application exhibited that the higher the rock joint roughness ( $Z_2$  value), the smaller the  
565 displacements (either at the dam crest or in its foundations).

566 Regarding the perspectives of this work, it is important to recall that the MC shear  
567 behaviour model integrating rock joint roughness is valid for specific conditions: a normal  
568 load from 0.1 to 0.6-MPa and a roughness indicator  $Z_2$  ranging from 0 to 0.373. Moreover,  
569 the shear behaviour model developed takes into account the effects of joint roughness on  
570 its shear strength and behaviour. However, the reader should know that other parameters  
571 could also have strong effects (rock properties, interlocking, contact properties, the  
572 presence of infilled materials in joints, etc.).

573 Future work will attempt to incorporate other shear behaviour / strength influence  
574 parameters in the shear behaviour model developed. However, this task requires much  
575 more data and implies launching new experimental and exhaustive studies. Finally, it is

576 important to note that the last part of this paper is intended to be only explanatory since  
577 the tests were conducted at the laboratory scale. Thus, more work is required to use the  
578 shear behaviour model developed accurately at a larger scale.

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772

773 **Appendix A**

774 Linear laws correlating the  $Z_2$  value to the shear behaviour model input parameters.

775 
$$K_N = 31.22 \cdot x + 68.14 \quad R^2 = 0.96 \quad (8)$$

776 
$$K_S = 4.91 \cdot x + 1.96 \quad R^2 = 0.77 \quad (9)$$

777 
$$C_{app\ peak} = 821.9 \cdot x - 28.5 \quad R^2 = 0.88 \quad (10)$$

778 
$$\phi_{peak} = 118.5 \cdot x + 28.06 \quad R^2 = 0.96 \quad (11)$$

779  $\varphi_{\text{residual}} = 89.6 \cdot x + 25.6$   $R^2 = 0.98$  (12)

780  $dN = 79.55 \cdot x + 0.26$   $R^2 = 0.99$  (13)

781 Exponential laws correlating the  $Z_2$  value to the shear behaviour model input parameters.

782  $K_N = 15.87 \cdot e^{1.49 \cdot x} + 52.61$   $R^2 = 0.97$  (14)

783  $K_S = 0.257 \cdot e^{5.77 \cdot x} + 1.90$   $R^2 = 0.94$  (15)

784  $C_{\text{app peak}} = 56.64 \cdot e^{5.09 \cdot x} - 55.01$   $R^2 = 0.99$  (16)

785  $\varphi_{\text{peak}} = 1.164 \times 10^4 \cdot e^{0.011 \cdot x} - 1.164 \times 10^4$   $R^2 = 0.96$  (17)

786  $\varphi_{\text{residual}} = 1.081 \times 10^4 \cdot e^{0.006 \cdot x} - 1.081 \times 10^4$   $R^2 = 0.96$  (18)

787  $dN = 1.809 \times 10^4 \cdot e^{0.0041 \cdot x} - 1.809 \times 10^4$   $R^2 = 0.98$  (19)