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This is the author's manuscript

Original Citation:

Availability:

This version is available <http://hdl.handle.net/2318/1711799> since 2019-11-14T08:36:46Z

Published version:

DOI:10.1007/s10346-019-01278-5

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(Article begins on next page)

1 **DESIGN OF ACTIVE DEBRIS FLOW MITIGATION MEASURES: A COMPREHENSIVE ANALYSIS OF**
2 **EXISTING IMPACT MODELS**

3

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8

9 **Abstract:**

10 Debris flows occur in mountainous areas characterized by steep slope and occasional severe rainstorms. The massive
11 urbanization in these areas raised the importance of studying and mitigating these phenomena. Concerning the strategy
12 of protection, it is fundamental to evaluate both the effect of the magnitude (that concerns the definition of the hazard),
13 in terms of mobilized volume and travel distance, and the best technical protection structures (that concerns the
14 mitigation measures) to reduce the existing risk to an acceptable residual one. In particular, the mitigation measure
15 design requires the evaluation of the effects of debris flow impact forces against them. In other words, once it is
16 established that mitigation structures are required, the impacting pressure shall be evaluated and it should be verified
17 that it does not exceed barrier resistance.

18 In this paper the author wants to focus on the definition and the evaluation of the impacting load of debris flows on
19 protection structures: a critical review of main existing models and equations treated in scientific literature is here
20 presented. Although most of these equations are based on solid physical basis, they are always affected by an empirical
21 nature due to the presence of coefficients for fitting the numerical results with laboratory and, less frequently, field data.
22 The predicting capability of these equations, namely the capability of fitting experimental/field data, is analysed and
23 evaluated using ten different datasets available in scientific literature. The purpose of this paper is to provide a
24 comprehensive analysis of the existing debris flow impact models, highlighting their strong points and limits.
25 Moreover, this paper could have a practical aspect by helping engineers in the choice of the best technical solution and
26 the safe design of debris flow protection structures. Existing design guidelines for debris flow protection barrier have
27 been analysed. Finally, starting from the analysis of the hydro-static model response to fit field data and introducing
28 some practical assumptions, an empirical formula is proposed for taking into account the dynamic effects of the
29 phenomenon.

30

31 **Keywords:** debris flow, impact models, landslide-structure interaction, predicting capability

32

33 **INTRODUCTION**

34 In the last decades, the climate changes have rapidly triggered the glacier melting, the permafrost degradation and the
35 generation of extreme events like rapid and severe rainstorms. All these aspects have contributed to increase the
36 possibility of occurrence of a particular type of landslide: debris flows (Zimmerman and Haerberli 1992).

37 Debris flow is a paroxysmic phenomenon due to a rapid or extremely rapid mobilization of a mixture of water, sediments
38 and floating material into a steep channel (Iverson 1997; Hungr 2005). Their high density, greater than 1700 kg/m^3
39 (ONR-24800 2009) and high runout velocity, up to 20 m/s (Hungr et al. 2014) make them decisive in the morphological
40 evolution of mountain areas, often extensively urbanized and therefore characterized by high hazard degree (Fioraso
41 2000). Their worldwide diffusion and the colonization of virgin areas, joined with the world population increase, grow
42 up the probability for debris flow to cause disasters.

43 Like avalanches, debris flows occur with little warning and exert great loads on obstacles they encounter. Like water
44 floods, they are fluid enough to travel long distances in channels and inundate vast areas (Iverson 1997). Moreover,
45 their unpredictability hampers collection of detailed real event data.

46 Since risks cannot be eliminated but only mitigated, many mitigation strategies have been developed in the last years.
47 When a potential source area is identified, since stabilization is not always a practical option, the consequences of
48 failure must be considered. The latter are the basis for the design of mitigation measures and for the management of the
49 residual risk (Jakob et al. 2016).

50 Mitigation measures can be divided in two different types:

- 51 - Active measures, which are focused on the hazard and essentially they prevent the debris flow triggering, transport
52 and deposition and can therefore change debris magnitude and frequency characteristics (Huebl and Steinwendtner
53 2000; Kienholz 2003).
- 54 - Passive measures, which are focused on the potential damage and are used to change the vulnerability of debris flow
55 either with hazard mapping (Bankoff et al. 2004; Griswold 2004) or through immediate disaster response (Kienholz
56 2003; Badoux et al. 2009; Santi et al. 2010).

57 Although passive measures are more advisable than active ones, the latter are necessary in order to correctly manage
58 residual risk (Jakob and Hungr 2005). A correct land-use planning and a hazard management implicate lower costs
59 (economical and social) but usually active measures are required, especially where there was an inadequate risk
60 management.

61 Since protection structures are designed to withstand the impact force of the moving mass, the estimation of the
62 potential impact pressure becomes a key aspect for safely design these mitigation measures. The scientific community
63 has widely faced the challenge of debris flow hazard assessment, but universally recognized models for the design of
64 these structures are still missing (Vagnon et al. 2016).

65 Data availability and universal applicability are the main issues for the development of predicting impact models. The
66 lack of data from monitoring of debris flow events forces to perform small-scale (e.g. Armanini and Scotton 1992;
67 Huebl and Holzinger 2003; Canelli et al. 2012; Wendeler and Volkwein 2015; Ashwood and Hungr 2016; Vagnon and
68 Segalini 2016) and full-scale flume experiments (DeNatale et al. 1999; Bugnion et al. 2012). Laboratory tests are useful
69 but they are affected by scale effects that cannot be properly quantified and, consequently, they may not replicate or be
70 comparable with field data (Iverson 1997; Huebl et al. 2009). Thus, their results must be interpreted with a healthy dose
71 of scepticism (Iverson 2015) since performing analogue experiments of large scale phenomena require satisfy all the
72 relevant similarity criteria but this is impossible for debris flows (Turnbull et al. 2015). Researchers have to choose if
73 maintain stress similarity or lost information on particle effects. The first scenario requires increasing the effective

74 gravity and consequently, performing centrifuge experiments. The use of uniform material, as in chute experiments, is a
75 huge simplification because produces limited pore-fluid pressure effects and excessive pore-fluid shear resistances that
76 can lead to underestimation of solid-fluid drag and dynamic effects. Consequently, an appropriate scale analysis is
77 always required for the study of mass movements such as debris flows.

78 In 1996, the US Geological Surveys (DeNatale et al. 1999) performed debris flow impact tests on flexible barriers in a
79 full-scale concrete flume, highlighting that the inherent variability of a well-controlled, staged debris flow made it
80 difficult to isolate the effect of any single parameter. Instead, Bugnion et al. (2011) in their full-scale experiments of
81 hillslope debris flows, stated that pressures depend primarily on the flow speed, which in turn appears to depend on the
82 grain-size distribution and water content.

83 As a consequence, even if the results of full- and small- scale tests are comparable to those observed in real-scale
84 measurements and simulate quite well the physics of idealised debris flows, they may not describe well the rheology,
85 the complex topography and the presence of obstacles (buildings, infrastructures etc.) along the debris flow path (Gao et
86 al. 2017).

87 Many of these formulations yield a rough estimation of the debris flow impact pressure against structures due to their
88 empirical nature, their validation only based on small-scale observations that could lead to high discrepancies with field
89 observations (e.g. Hungr et al. 1984; Revellino et al. 2004; Zanchetta et al. 2004; Shen et al. 2018). Then, dimensions,
90 types and inertial resistance of the barriers are completely neglected in most of these models (Vagnon and Segalini
91 2016).

92 Finally, for what it concerns the model validation, it is performed on limited datasets, both in terms of number and type
93 of observations.

94 The aim of this paper is to analyse debris flow impact models proposed in scientific literature and evaluate the
95 discrepancies between measured and numerically predicted results using ten datasets available in scientific literature.
96 Sixteen formulations were accurately discussed, highlighting their strong points and their shortcomings. As a result of
97 the data analysis, a new formulation is here presented.

98 The insights carried out from this paper will be useful for engineers to design debris flow protection measures.
99 Moreover, the presented results will help engineers in the choice of the best debris flow impact model as a function of
100 phenomenon features and mitigation measures technical characteristics.

101 The present study is organized into six sections: the first one is introductory and presents the most common mitigation
102 methods for debris flows. The second section provides a review of debris flow impact models. Section three
103 summarizes current international standards in debris-flow mitigation design. In section four and five, the impact models
104 are compared and statistically analysed for evaluating their reliability in predicting measured pressure. Finally, an
105 empirical model is proposed and the main results of this study are summarised and discussed.

106

107 **DEBRIS FLOW CONTROL BY BARRIERS**

108 The use of mitigation measures depends on the adopted protection strategy and on the objectives established from the
109 risk assessment (Huebl 2001). The choice of the best mitigation measure must be evaluated with respect to its technical,
110 economical, ecological and political feasibility.

111 Generally, the areas susceptible to debris flow phenomena are narrow and not suitable for installing large structures.
112 Although setting only one structure is often by no means sufficient to make debris flow harmless and, moreover an
113 integration of both active and passive measures should be encouraged (Takahashi 2007), in the last three decades many
114 active mitigation measures have been installed worldwide. Active debris flow mitigation measures affect the initiation

115 or the transport or the deposition of debris flows. Mitigation measures have a direct effect on the magnitude and on the
116 frequency of the phenomenon, changing the probability of the event or manipulating the debris flow itself. They must
117 be designed to resist the impact force and their main tasks are: i) dissipate the debris flow kinetic energy and ii) retain
118 totally or partially the debris flow material (VanDine 1996; Mizuyama 2008; Brighenti et al. 2013; Song et al. 2017;
119 Wendeler et al. 2018).

120 Common debris flow structural measures include close-type check dams (Fig. 1a), open-type sabo dams (Fig. 1b),
121 concrete slit sabo dams (Fig. 1c) and flexible net barriers (Fig. 1d).

122

123 **Fig. 1** Different types of debris flow active mitigation measures: check dam (photograph by Los Angeles County Flood
124 Control District) (a), open type Sabo structures (photograph of steel check dam in Nagano prefecture, Japan) (b),
125 concrete slit barrier (photograph by LCW Consult web site of protection works in St. Luzia River, Madeira, Portugal)
126 (c) and flexible net barrier (photograph by Geovertical S.R.L web site of protection works in Terranova Pollino,
127 Basilicata Region, Italy) (d)

128

129 The design of these structures should comply with two requirements: firstly, it should take into account geographical,
130 geological and site conditions. Secondly, at the end of design process the structural resistances shall be always greater
131 than the effects of the forces exerted on the structure. For what it concerns the barrier resistance, it can be easily
132 evaluated since the resistance of each single component is well known and accurately calculated. By contrast, the
133 definition of the impacting load on the structure is an open issue (Vagnon et al. 2017a): as it will be described in the
134 next section, there are many models for quantifying the stress distribution on the barrier, but none of them is universally
135 recognized. Moreover, their predicting capability, that is an evaluation of the discrepancy between measured and
136 estimated impact forces, especially using data collected from real events, is unknown. In particular, for mitigation
137 measure designers become of utter importance to know under which dynamic features the impact models lead to an
138 underestimation or excessive overestimation of the load conditions.

139 Finally, it is important to underline that the accuracy of dimensioning procedure is therefore highly dependent on the
140 quality of process scenario. Inaccurate assumptions may result in inefficient design or in a partial or total failure of the
141 structure and can lead to negative consequences for the vulnerable area. This means that poor quality input data (flow
142 velocity, thickness, density, volume etc.) arises uncertainties in the evaluation of impacting load.

143

144 **DEBRIS FLOW IMPACT MODELS**

145 For an efficient design of mitigation structures, the debris flow impact pressure exerted on barriers is of utter
146 importance because it is the main factor that causes structural collapse (Hungry et al. 1984; Armanini 1997; Huebl et al.
147 2009; Ferrero et al. 2015). Furthermore, there is an increasingly greater need in predicting impact load for the
148 assessment and the management of risk.

149 Debris flow involves fundamentally independent physical and dynamical processes that couldn't be controlled by one
150 or two parameters. The flow is heterogeneous and the mixture density evolves strongly as a function of time and space
151 due to mixing, phase separation and particle sorting: this aspect leads to drastic change of the local material
152 composition and it can result in huge impact pressure differences during impact (Iverson 1997). Moreover, density,
153 velocity and flow height should be considered as field variable due to their variation through space, along the channel
154 path, and time, during the flow process (Hungry et al. 1984; Kwan 2012). For all these reasons, the development of a
155 reliable debris flow impact scheme results extremely complex.

156 The design of mitigation structures should require simplified models to predict impact pressure with high reliability;
157 these models should be universally recognized and should include few parameters, related to material and flow
158 characteristics easy to estimate.

159 Finally, the modelling of debris flow surges is difficult because the impact pressure depends on a dynamic component
160 exerted by the heterogeneous flow, that can reach $10 - 5 \times 10^3 \text{ kN m}^{-2}$, and an impulsive component generated by the
161 single impact of boulders: the latter can vary between 10^2 and 10^4 kN m^{-2} (Suwa and Okuda 1983; Zhang 1993).

162 Moreover, the flow composition and the impact mechanism strongly influenced the impact load and its distribution on
163 structures (Song et al. 2017). Many studies (Choi et al. 2015; Sovilla et al. 2016; Ng et al. 2019) on this topic have
164 highlighted that when dry granular flows impacting a rigid barrier, a pileup mechanism developed. On the contrary,
165 viscous flows exhibited the formation of a vertical-jet mechanism upon impact. This happens when the flow inertia is
166 larger than restoring gravitational field (Poudyal et al. 2019). In fact, in viscous flow, the effect of particle shearing in
167 kinetic energy dissipation is less significant compared to frictional-grain stresses in dry granular flow.

168 It is obvious that debris flow modelling requires many assumptions for simplifying its real complex nature: a) the
169 mixture has to be considered as an equivalent fluid with averaged characteristics of density, b) the simultaneous
170 occurrence of maximum velocity and thickness values, c) the rigid behaviour of the flow at the impact (Osanaï et al.
171 2010; Suda et al. 2010; Kwan 2012).

172 On the basis of these hypotheses, in the last decades many methods were developed and, in general, they can be
173 classified into hydraulic and solid-collision (Huebl et al. 2009) and shock-wave propagating upstream models (Chou et
174 al. 2012; Albaba et al. 2018).

175 Hydraulic models, derived from fluid momentum balance and Bernulli's equation, schematize the flowing mass as a
176 homogeneous mean (characterized by an average density between fluid and solid component) and consider the load as a
177 modified value of hydro-dynamic pressure or a multiple of the hydro-static load or a combination of both.

178 The maximum impact pressure considering hydro-static model can be evaluated using the following equation:

179

$$180 \quad p_{\text{peak}} = k \cdot \rho_m \cdot g \cdot h_f \quad (1)$$

181

182 where p_{peak} is the maximum impact load in N m^{-2} , k is an empirical coefficient, ρ_m is the mean density of the debris
183 impacting fluid in kg m^{-3} , g is gravity in m s^{-2} and h_f is the flow height in m. This model is based on a triangular load
184 distribution and the load increase factor, k (e.g. Lichtenhahn 1973; Armanini 1997). The latter can assume values
185 ranging from 2.5 to 7.5.

186 Lichtenhahn (1973) firstly applied this equation for the evaluation of debris flow impact on concrete barrier, proposing
187 k values between 3.5 and 5.5. Later, following the same theoretical principles of the previous study and comparing
188 results with experimental tests, Armanini (1997) evaluated k as 4.5 times the hydro-static pressure. At the same time,
189 Scotton and Deganutti (1997), performing small scale laboratory tests and measuring impact pressure on vertical
190 obstacle, estimated k varying between 2.5 and 7.5, depending on the viscosity of the interstitial fluid and flow hydraulic
191 conductivity.

192 The popularity of these formulations is due to their simplicity and the few number of parameters involved: in fact they
193 only require debris density and flow height and usually flow height is considered equal to channel depth. On the other
194 hand, they do not take into account flow rheological properties and they are only applicable with small velocity values
195 or rather with flatter terrain.

196 Concerning hydro-dynamic models, the impact on the structure has a constant load distribution and the general equation
197 is:

$$198 \quad p_{\text{peak}} = \alpha \cdot \rho_m \cdot v_f^2 \quad (2)$$

200 where α is the dynamic coefficient and v_f is the flow velocity in ms^{-1} .
201 The parameter α include information about the flow type, the formation of vertical jet-like wave at the impact, the grain
202 size distribution and the barrier type (Canelli et al. 2012).

203 Watanabe and Ikeya (1981) firstly applied this model for the analysis of volcanic mudflow in Japan. They found that α
204 ranged between 2 and 4 as a function of the grain size distribution of the mixture.

205 The following equation is the Hungr et al.' equation (1984) and it is maybe the most famous hydro-dynamic
206 formulation used for evaluating debris flow impact pressure against obstacles:

$$207 \quad p_{\text{peak}} = 1.5 \cdot \rho_m \cdot v_f^2 \cdot \sin\beta \quad (3)$$

208 where β is the least angle between the face of the barrier and the flow direction.
209 The dynamic coefficient equals to 1.5 was defined after the back-analysis of data from monitoring of real debris flow
210 events occurred in British Columbia (Canada). This coefficient was included for considering the generation of a
211 stagnant wedge behind protection barriers.

212 Moreover, in scientific literature exists a wide range of proposed values for the dynamic coefficient: Daido (1993)
213 suggested values varying between 5 and 12, Zhang (1993) recommended a range between 3 and 5, Bugnion et al.
214 (2011) measured values from 0.4 to 0.8, Canelli et al. (2012) between 1.5 and 5.

215 The values listed above prove that the range of variation of dynamic coefficient (between 0.4 and 12) deeply conditions
216 the evaluation of peak pressure: consequently, from an engineering point of view, the design of protection barriers is
217 strongly influenced by the choice of one formulation respect to another, arising uncertainties in the reliable evaluation
218 of the probability of failure of the mitigation measure.

219 Huebl and Holzinger (2003) developed a modified hydro-dynamic formula introducing Froude number (F_r) to
220 normalised impact force and achieved a scale-independent relationship:

$$221 \quad p_{\text{peak}} = 5 \cdot \rho_m \cdot v_f^{0.8} \cdot (g \cdot h_f)^{0.6} \quad (4)$$

222 Using small-scale flume experiments and 155 sets of data coming from other authors, Cui et al. (2015) estimated the
223 peak impact pressure as:

$$224 \quad p_{\text{peak}} = 5.3 \cdot F_r^{-1.5} \cdot \rho_m \cdot v_f^2 \quad (5)$$

225 Combining Equation 1 and 2, many Authors hypothesized new relationships to estimate maximum impact pressure
226 against barrier. The general relation is:

$$227 \quad p_{\text{peak}} = \rho_m \cdot g \cdot h_f + \rho_m \cdot v_f^2 \quad (6)$$

237 Cross (1967) firstly modified the equation for perfect fluid used for evaluating tsunami impact forces introducing static
 238 coefficient, k , and dynamic coefficient, α , respectively equal to 0.5 and 3 as follow:

239

$$240 \quad p_{peak} = k \cdot \rho_m \cdot g \cdot h_f + \alpha \cdot \rho_m \cdot v_f^2 \quad (7)$$

241

242 Later, Arattano and Franzi (2003), analysing measured data in Moscardo Torrent (Italy), validated Equation 6.

243 Other studies were carried out hypothesizing the total reflection of a flow against a vertical wall and, imposing the
 244 dynamic equilibrium (Lamberti and Zanuttigh 2004). The relation is:

245

$$246 \quad p_{peak} = C_c \cdot \frac{(1+\sqrt{2}F_r)^2}{2} \cdot \rho_m \cdot g \cdot h_f \quad (8)$$

247

248 where C_c is an empirical coefficient calibrated considering the vertical acceleration caused by the presence of fine
 249 particles and boulder equal to 1.5.

250 Another equation to evaluate the dynamic impact of a debris flow against a vertical wall is presented by Armanini et al.
 251 (2011):

252

$$253 \quad p_{peak} = \left(1 + \frac{1}{2} \cdot F_r^2\right) \cdot \left(1 + \frac{\alpha \cdot F_r^2}{1 + \frac{1}{2} F_r^2}\right) \cdot \rho_m \cdot g \cdot h_f \quad (9)$$

254

255 where α is a coefficient equal to 1.

256 Recently, Vagnon and Segalini (2016), performing several small scale flume tests, proposed a new model that takes into
 257 account either flow characteristics, material properties and barrier dimensions. The numerical expression is given as:

258

$$259 \quad p_{peak} = \left(\frac{1}{2} K_a (n^2 - 1) \cos\theta + \alpha F r^2 \cos\beta - \tan\varphi' \frac{n-1}{\sin\theta} \cos\beta \cos\theta\right) \rho_m g h_f \quad (10)$$

260

261 where K_a is active lateral earth pressure coefficient derived from Rankine theory, θ is slope angle in deg, β is the angle
 262 between the barrier and the normal at channel bottom, measured in deg, φ is the debris friction angle in deg and n is the
 263 filling ratio, that is the ratio between the barrier height and the flow height. In this relation, dynamic coefficient, α , can
 264 vary between 0.5 and 1.2.

265 Solid-collision models are based on the Hertz contact theory as the following:

266

$$267 \quad F_B = K_c n a^{1.5} \quad (11)$$

268

269 where K_c is the load reduction factor that depends on barrier stiffness (Hung et al. 1984; Kwan 2012; Ng et al. 2016), n
 270 takes into account the radius of impacting boulder, its Poisson's ratio and the Young's modulus of the boulder itself and
 271 the barrier. The parameter a , depends on boulder mass and impact velocity. In scientific literature, many solid-collision
 272 models have been presented (Kwan 2012; Faug 2015; Ng et al. 2016; Song et al. 2017): their use is related to the ratio
 273 between boulder dimension and debris flow thickness. Ng et al (2016) highlighted that if this ratio is lower than 0.6,
 274 solid-collision contribution can be neglected and the impact of debris flow can be calculated using Equation 2 with α
 275 equals to 2.5.

276 For what it concerns shock-wave models, the impacting force is considered as a combination of inertial and depth-
277 dependent forces associated with features of the incoming flow (Albaba et al. 2018). Although the shock-wave solution
278 is obtained from the jump conditions of the mass and momentum balances, its predictions are in good agreement with
279 experimental results (Chou et al. 2012). The main limitation of these methods is the high sensitivity of peak force to
280 sampling length (that defines the sample dimension for averaging flow motion characteristics): in particular, Albaba et
281 al. (2018) highlighted that only for slope angle greater than 42.5° and sampling length greater than 35 times the average
282 particle diameter, the peak force is well predicted.

283 One further remark has to be made concerning the use of all the presented methods in numerical modelling: many
284 numerical codes (Hutter et al. 1994; Brighenti et al. 2013; Albaba et al. 2015; Ashwood and Hungr 2016; Leonardi et al.
285 2016; Vagnon et al. 2017b; Wendeler et al. 2018) have been developed on the basis of hydro-static and hydro-dynamic
286 simplified approach.

287 In this paper only hydraulic models has been treated and discussed, since not all the necessary parameters were
288 available for evaluating the contribution of solid-collision and shock-wave models.

289 Table 1 summarizes general equations of all the previous models, highlighting the range of variation for static (k) and
290 dynamic (α) coefficient. Moreover, Table 1 includes information about the texting procedure and how α and k
291 coefficients were evaluated (cfr. columns three and four of Table 1). These coefficients depend on the type of mitigation
292 measure considered and the characteristics of flow, in terms of grain size and viscosity. Mainly, they were calibrated
293 performing small-scale tests on rigid (e.g. Armanini 1997; Scotton and Deganutti 1997; Kwan 2012) or flexible barriers
294 (e.g. Canelli et al. 2015; Ashwood and Hungr 2016; Wendeler et al. 2018) and more rarely, they were evaluated as a
295 result of full-scale test or real debris flows (Arattano and Franzi 2003; Bugnion et al. 2011).

296

297 **Table 1** – Summary of analysed hydraulic model for evaluating peak debris impacting pressure on barriers.

298

299 INTERNATIONAL GUIDANCES FOR MITIGATION DESIGN: AN OVERVIEW

300 At present, there are few existing international technical guidelines about debris flow mitigation measures, one of which
301 is undoubtedly the ONR series (ONR 24800 to ONR 24803) developed by the Austrian “Wildbach und
302 Lawinenverbauung” office (WLV). The debris flow load models and the design, construction and life cycle assessment
303 of protection works are arranged in these standards (Suda et al. 2010). In particular, ONR 24801 defines two models for
304 calculating debris flow impact pressure: the first one, named as simple model, is based on Equation 1, assuming that h_f
305 is equal to the barrier height and k ranges normally between 3 to 6. The second one, named as complex model,
306 corresponds to Equation 4. In both cases, the standard specifies that flow parameters must be given by an expert for
307 torrential control. Moreover, twelve stress combinations are defined depending on the functional type of mitigation
308 structure in order to take into account uncertainties and provide adequate design of mitigation measure. At European
309 level, ONR series are the only standards concerning debris flow mitigation structure design: recently, the European
310 Organisation for Technical Assessment published the new European Assessment Document (EAD 340020-00-0106
311 2016) concerning the flexible kits for retaining debris flows and shallow landslides/open hill debris flows. This
312 regulation defines the main components and the methods to assess the performance of the kit. These guidelines concern
313 only the certification of barrier performances but no information on load distribution are given.

314 Other existing standards are the Japanese NILIM (National Institute for Land and Infrastructure Management) 904 and
315 905 (Osanaï et al. 2010) that define the design characteristics of Sabo barriers. Compare to Austrian ONR series, these
316 guidances are less specific (Moase 2017) since they suggest considering different combination of external forces (static

317 water pressure, sediment pressure and fluid forces of debris flow) without providing any formulations for their
318 calculation.

319 One of the most comprehensive standard is that developed by Geotechnical Engineering Office (GEO) of Hong Kong.
320 In particular, the GEO report 270 (Kwan 2012) presents guidelines on the design of debris-resisting barrier. For what it
321 concerns impact model, the design guidance recommends that the design loading on barrier is based on multi-surge
322 scenario and the total load is considered as the sum of dynamic impact and boulder impact (if the existence of boulders
323 or large hard inclusions in the flow cannot be precluded). Boulder impact should be calculated using the simplified form
324 of Equation 11 as following:

$$325 \\ 326 F = K_c 4000 v^{1.2} r^2 \quad (12)$$

327
328 where K_c is equal to 0.1, v is the boulder impact velocity normal to the barrier and r is the radius of boulder. Dynamic
329 load should be calculated using Equation 2 with α equals to 2.5.

330 Analysing the previous standards, it is clear that there is not a universally recognized impact model: each guideline is
331 based on local experience of use of debris resisting barriers.

332

333 **EVALUATION OF THE PREDICTING CAPABILITY OF DEBRIS FLOW IMPACT MODELS**

334 As stated above, the design of mitigation measures requires defining the load exerted by the flow on structures. The
335 equations listed in Table 1 show that universally recognize model does not exist and thus it becomes fundamental for
336 designers to know the capability of these models to fit experimental and field measurements. The choice of a model in
337 relation to another mainly depends on: a) how accurately is the calculated pressure compared to the measured one; b)
338 the limitations (if any) in the applicability of the model.

339 The predicting capability of the previous models was evaluated by comparing the predicted results with data coming
340 from ten different datasets available in scientific literature. As listed in Table 2, their choice was made on the basis of
341 availability of both dynamic information (thickness, velocity and density) and impact features (impact load) and for
342 ensuring that the datasets covered different testing scenarios (soil type, water content, channel slope, magnitude) and
343 different scale approaches. The datasets included values of flow velocity, thickness and impacting peak pressure
344 collected from small scale tests performed in specifically created flumes (Scheidl et al. 2013; Cui et al. 2015; Ashwood
345 and Hungr 2016; Vagnon and Segalini 2016), from full-scale debris flow (DeNatale et al. 1999; Bugnion et al. 2011)
346 and from monitoring of recurrent debris flow in the Jiangjia Ravine basin in China (Hu et al. 2011; Hong et al. 2015)
347 and Illgraben debris flow monitoring site in Switzerland (McArdell 2016; Wendeler et al. 2007).

348 Moreover, The choice of analysing both small-scale tests and field data was made to understand if the approach here
349 presented could produce reliable results and to dispel any doubts on the result interpretation. The author is conscious
350 that miniaturized tests are affected by scale effects and investigated materials are usually not satisfyingly representative
351 of material involved in debris flow phenomena, but if a trend is recognized in both datasets, the analysis can be
352 considered as a sign of a reliable approach.

353 In this paper, the data of peak impact pressure were normalized by those of hydro-static pressure as follows:

354

$$355 \check{p} = \frac{p_{peak_{measured}}}{\rho \cdot g \cdot h_f} \quad (13)$$

356

357 These values were plotted as a function of the Froude number, Fr , of the flow in order to achieve scale-invariant
358 description. Froude number is defined as the square root of the ratio between kinetic and gravity force of the flow and it
359 is useful for demarcating quasi static rate-independent from speed-squared force contributions (Faug 2015). An
360 important remark has to be done for the evaluation of Fr values: a recent study published by Ng et al. (2019) highlighted
361 that the choice of frontal velocity and maximum flow depth within the frontmost region of the flow is crucial for
362 properly characterising the impact mechanism. In particular, an estimation of non-frontal Fr values may lead to an
363 underestimation of impact pressure by a factor of two.

364 In general, laboratory and field data accordingly show the same range of variation of the normalized peak pressure; for
365 what it concerns the Froude number, the range of variation related to laboratory tests is wider than the field one. As
366 stated above, this discrepancy can be attributed to scale effects. However, the two dataset globally have the same trend
367 for \tilde{p} as function of Fr .

368 The normalized pressures, \tilde{p} , were compared to all the listed above predicting models, considering for each one the
369 upper and the lower limit of the range of variability of empirical coefficients k and α . For the sake of simplicity and
370 readability of figures, the models were pooled into three groups (hydro-static, hydro-dynamic and mixed models) and
371 plotted separately for better highlighting limitations and strong points for each model.

372 In Fig. 2, the predicting capability of hydro-static models is evaluated: in general, these formulations underestimate the
373 normalized peak pressure measured both in small- and full-scale tests (Fig.s 2a and 2b) and field data (Fig. 2c),
374 regardless of the value of empirical coefficient k .

375 In general, from an engineering point of view, the fact that hydro-static models underestimate the debris flow impact
376 pressure (with the exception of Scotton and Deganutti (1997) model with $k = 7.5$ for Fr lower than 3) points out an
377 inadequacy for the design of protection structures.

378

379 **Fig. 2** Comparison between normalized debris flow impact force and hydro-static predicting models as function of
380 Froude number considering small- (a) and full-scale experiments (b) and field data (c).

381

382 For a deeper analysis, the ratio between the measured peak pressure and the estimated one was calculated and reported
383 in Fig. 3 as function of Froude number for both small- (a, d) and full-scale (b, e) and field datasets (c, f). The upper
384 (7.5) and the lower (2.5) limits of the k range of variation were chosen in order to define the suitability for predicting
385 impact pressure of the hydro-static models.

386 The peak pressure ratio gives information about the discrepancy between predicted results using hydro-static methods
387 and measured ones: if this ratio is lower than 1, the predicting value overestimates the measured one, vice-versa the
388 peak pressure is underestimated. If this ratio is 1, there is a perfect correspondence (green continuous line in Fig. 3)
389 between measured values and predicted one. From an engineering point of view, for providing safe results, predicting
390 models should exhibit a ratio lower than unity, and in particular varying between 0.5 and 1, so that the impacting
391 pressure is reasonably overestimated. This aspect will be better explained in the next section.

392 Analysing Fig. 3, hydro-static formula with $k = 2.5$ (that corresponds to the lower limit of Scotton and Deganutti (1997)
393 equation) always underestimates the measured peak pressure (Fig.s 3a to 3c). On the contrary, Scotton and Deganutti
394 (1997) equation considering $k = 7.5$ simulates well about the 55% of the field data (Fig. 3f). These two equations define
395 the variability domain of the existing hydro-static formulations.

396 Analysing Fig. 2 and Fig. 3, there is an evident non-linear relation between estimated pressure and Froude number: the
397 higher is Fr a, the higher is the distance between measured and estimated pressure value. Moreover, considering

398 Scotton and Deganutti (1997) formulation (with $k = 7.5$), it seems that the prediction capability of this model is high for
399 $Fr < 3$; in this case the percentage of peak pressure ratio lower than 1 increases till 70%.

400 It becomes obvious that hydro-static models are suitable only for low Froude numbers (generally lower than 3, Huebl
401 and Holzinger 2003; Cui et al. 2015), namely when the flow is characterized by low velocity and dynamic components
402 are negligible (Huebl and Holzinger 2003) (cfr. Fig. 3).

403 In this study, starting from this observation, an attempt to take into account dynamic effects in hydro-static models will
404 be presented and discussed.

405

406 **Fig. 3** Relationship between measured peak pressure and calculated hydro-static peak pressure with k respectively equal
407 to 2.5 (a to c) and 7.5 (d to f) as a function of the Froude number considering both small- (a, d) and full- scale (b, e) and
408 field dataset (c, f). The green continuous line represents the perfect correspondence between measured values and
409 estimated ones.

410

411 Concerning hydro-dynamic models, they have a good capability in predicting the peak impact pressure (Fig. 4) except
412 for models with dynamic coefficient, α , lower than 1 (Bugnion et al. (2011), cfr. Fig.s 5a to 5c). The two key points are:
413 a) verify if the predicting capability is influenced by flow regime; b) considering maximum dynamic coefficient value
414 ($\alpha = 12$ from Daido (1993)) the peak values are excessively overestimated. The latter point has a great impact on the
415 design of mitigation measures, in particular on their construction costs (cfr. next section).

416 In Fig. 4a, for very low Froude values ($Fr < 2$), it seems that hydro-dynamic models are affected by of the influence of
417 flow regime, characterized by low velocity and high impact thickness. In this case, only Daido (1993) equation reaches
418 to satisfactorily predict peak pressure.

419

420 **Fig. 4** Comparison between normalized debris flow impact force and hydro-dynamic predicting models as function of
421 Froude number considering small- (a) and full-scale tests (b) as well as field data (c).

422

423 Focusing on Hungr et al. (1984) equation (continuous blue line in Fig. 4), which is certainly the most famous, in the
424 86% of cases it overestimates the measured peak pressure. This formulation has a high predicting capability for high
425 Froude values, meanwhile for Froude equal or lower 2 it underestimates measured pressure (Fig.s 4a and 4c). In Fig. 5,
426 this aspect is better clarified: observing the pressure ratio, it decreases when Froude number increases following an
427 inverse power law. Except for Bugnion et al (2011) equation (Fig.s 5a to 5c) for which the pressure ratio is almost never
428 lower than unity, in the other formulations this relationship is verified for $Fr > 2$. Daido (1993) formulation with $\alpha = 12$
429 (Fig.s 5d to 5f) deserves a debate of its own: in fact it excessively overestimates the pressure values (except for $Fr < 2$,
430 confirming the observation made above), reducing pressure ratio close to 0. This latter point will be better argued in
431 next section.

432

433 **Fig. 5** Relationship between measured peak pressure and calculated hydro-dynamic peak pressure with α respectively
434 equal to 0.4 (a to c) and 12 (c to d) as a function of the Froude number considering both small- (a, d) and full-scale (b,
435 e) and field dataset (c, f)

436

437 Summarizing, the analysis performed on small- and full-scale and field data has highlighted that pure hydro-static and
438 pure hydro-dynamic models are not totally adequate to predict debris flow impact pressure on structures. In Froude

439 region lower than 3, where velocity are low and impacting thickness are high (cfr. Fig. 6) hydro-static formulations
440 perform well; on the contrary, hydro-dynamic models underestimate pressure values since kinetic effect are not
441 dominant (Huebl and Holzinger 2003, Cui et al. 2015; Faug 2015).

442

443 **Fig. 6** Relationship between velocity (blue squares) and thickness (red diamonds) as function of Froude number for
444 small-scale tests (a) and full-scale and field data (b). A negative correlation exists between velocity and thickness: when
445 Froude number increases, velocity increases and consequently flow height decreases and vice-versa.

446

447

448 In the light of previous observations, mixed models are more suitable for predicting impact loads as clearly showed in
449 Fig. 7 in which all data (both from small- and full-scale tests and field measurements) fall into the region defined by
450 these formulations.

451

452 **Fig. 7** Comparison between normalized debris flow impact force and mixed predicting models as function of Froude
453 number considering small- (a) and full-scale test (b) as well as field data (c).

454

455 For evaluating the performance of each formulation, the peak pressure ratio was evaluated, as shown in Fig. 8. For
456 readability of the figure, only field data were plotted. However, mixed models showed the same behaviour regardless
457 the choice of dataset. Generally, despite their prediction capability is more suitable than hydro-static and hydro-
458 dynamic formulations, not all mixed models can be universally usable in practise. For instance, Huebl and Holzinger
459 (2003) and Cui et al. (2015) equations perform well for Froude values lower than 3 (Figs 8a and 8b). Arattano and
460 Franzi (2003) formulation provides a good correspondence between field data and predicted ones: on the contrary,
461 Cross (1967) equation excessively overestimates pressure peak values (providing a peak pressure ratio close to 0, cfr.
462 Fig. 8c). Armanini et al. (2011) equation shows a neglecting dependence with Froude number, which causes,
463 particularly for low values ($Fr < 2$), an underestimation of the peak pressure (Fig. 8e). This dependence is not evident in
464 the other models (Fig. 8c and 8f).

465

466 **Fig. 8** Relationship between measured peak pressure and calculated peak pressure for different mixed models as a
467 function of the Froude number considering field dataset (a to f).

468

469 A COMPREHENSIVE STATISTICAL ANALYSIS OF PREDICTING IMPACT MODEL

470 In this section, the predicting capability of debris flow impact models has been statistically analysed.

471 In Table 3, the results of the comparison between all the described models and small- and full-scale/field datasets are
472 presented. The percentage is referred to the number of predicted values greater than measured ones. This condition
473 occurs when the model overestimates the impacting peak pressure. If the percentage is high, the model has a good
474 capability of overestimating measured peak pressure; on the contrary, if the percentage is low, the model is not suitable
475 for predicting impact pressure. However, the percentage of overestimated peak pressure values is not sufficient for
476 evaluating the reliability of a predicting model. From an engineering point of view the excessive overestimation shall be
477 avoided as much as underestimation since it is related to mitigation measure construction costs. Thus, for each models
478 was calculated the percentage of data that fall into four classes of peak pressure ratio, defined as follow:

- 479 - From 0 to 0.5 (orange class): it represents an excessive overestimation and consequently higher construction
480 costs. If the predicting model shows a high percentage of data in this class, it should be discarded.
- 481 - From 0.5 to 0.7 (yellow class): if the predicting model shows a high percentage of data in this class, a careful
482 analysis of cost benefit should be conducted when considering the suitability of the model.
- 483 - From 0.7 to 1 (green class): if the predicting model shows a high percentage of data in this class, it is
484 extremely accurate in estimating impact pressure.
- 485 - From 1 to 1.3 (yellow class): taking into account the uncertainties related to parameter measurement, a careful
486 analysis should be performed for choosing or discarding the model.
- 487 - Greater than 1.3 (red class): if the predicting model shows a high percentage of data in this class, it is not
488 suitable for estimating debris flow impact pressure.

489 This classification gave indications about the level of overestimation/underestimation, which is important especially for
490 defining the degree of safety of the mitigation measure and, indirectly, its construction costs.

491 Figures 9 to 11 add more information to Table 3 about predicting capability of analysed impact models: in general, all
492 the hydro-static models (Fig. 9) are not suitable to describe debris flow impact behaviour since they has an high
493 percentage (greater than 30%) of values that fall into red class, regardless the k-coefficient value. Hydro-dynamic
494 formulations have a high propensity to excessively overestimate impact pressure when $\alpha > 2$ (Fig.s 10d to 10h). Vice-
495 versa, when dynamic coefficient is lower than unit (Fig.s 10a and 10b), these models are not suitable for predicting
496 impact load. Hungr et al. (1984) and Canelli et al. (2012) models with $\alpha = 1.5$ (Fig. 10c) seem to be a good compromise
497 between overestimation and prediction capability. Except for Croos (1967) equation (Fig. 11a) that exhibits an
498 excessive overestimation, mixed models prove their adequacy as predicting methods since they show high percentage of
499 values that fall into green class. In particular, Arattano and Franzi (2003), Armanini et al. (2011) and Vagnon and
500 Segalini (2016) models (Fig.s 11b, 11c and 11g) result the most suitable for predicting real debris flow impact on
501 structures due to the high percentage of data that fall into green class (more than 40%).

502

503 **Fig. 9** Predicting capability analysis of hydro-static models using field dataset.

504

505 **Fig. 10** Predicting capability analysis of hydro-dynamic models using field dataset.

506

507 **Fig. 11** Predicting capability analysis of mixed models using field dataset.

508

509 TOWARDS A GENERALISED IMPACT MODEL

510 As highlighted in previous Sections, hydro-static formulations can be used, with a reliable degree of safety, to predict
511 impact pressure for flows with $Fr < 3$. Moreover, analysing Table 3 and Fig. 9, it is evident how the predicting
512 capability of these methods is very low: considering the highest k value, less than 55% of the peak pressure values are
513 overestimated. This aspect cannot be neglected by mitigation measure designers and consequently, it makes these
514 formulation not completely suitable for the estimation of impact forces on structures.

515 In the light of these aspects, is it possible to revise hydro-static model, improving its capability to overestimate field
516 data?

517 Fig. 12 shows that a power law governs the trend between peak pressure ratio and Froude number. In particular the
518 relation is:

519

520
$$\frac{p_{peak,measured}}{\rho \cdot g \cdot h_f} = a \cdot Fr^b \tag{14}$$

521

522 where a and b are respectively equal to 1.38 and 1.64.

523

524 **Fig. 12** Relationship between field measurements of the peak pressure and calculated peak pressure using hydro-static
525 formulation with $k = 1$ as a function of the Froude number of the flow

526

527 Starting from Equation 14 and taking into account that estimated values should overestimate measured ones, the
528 following modified hydro-static equation is proposed:

529

530
$$p_{peak} = 1.5 \cdot a \cdot Fr^b \cdot \rho \cdot g \cdot h_f = k^* \cdot \rho \cdot g \cdot h_f \tag{15}$$

531

532 where a and b are respectively equal to 1.38 and 1.64 as derived from data interpolation (Fig. 12) and 1.5 is an
533 increasing coefficient for overestimating the impact pressure. The choice of 1.5 is done in order to obtain an average
534 peak pressure ratio of 0.8 (red line in Fig. 13) so that the estimated pressure is reasonably overestimated.

535 It has been observed that in 86% of cases this formulation overestimates field data; this percentage is comparable with
536 that obtained using hydro-dynamic and mixed models. The statistical analysis of predicting capability is comparable
537 with that of mixed model, with high percentage of values falling into yellow and green classes (Fig. 13b). Moreover,
538 analysing Fig. 13a, the Froude dependence exhibited by classic hydro-static models (cfr. Fig. 2 and Fig. 3) is not present
539 in this modified formulation. On average, the peak pressure ratio (as function of Fr) has a horizontal trend. Only in
540 correspondence of $Fr = 2$, few estimated values underestimate measured ones; however this aspect also occurs in Fig. 2
541 and Fig. 3 for other hydro-static models.

542

543 **Fig. 13** Relationship between peak pressure ratio for the modified hydro-static model as a function of the Froude
544 number (a) and statistical evaluation of the predicting capability of the proposed model (b).

545

546 This new formulation, although it follows the same theoretical concepts of hydro-static models, has an empirical nature
547 since its empirical coefficient results from the analysis of field data. Fig. 14 shows the comparison between the
548 proposed equation and the impact models that, on the basis of the previous statistical analysis, have exhibited the best
549 predicting capability. Moreover, Lamberti and Zanuttigh (2004) and Armanini et al. (2011) models have some
550 similarities with the proposed. In fact, the three formulations have analogous trends of \tilde{p} as function of Fr ; in general,
551 mixed hydro-static models overestimate field data, particularly with high Fr , compared to the proposed one. Moreover,
552 the new formulation introduces (on the basis on the used field dataset) the concept of reasonable overestimation (the
553 increasing value of 1.5 of Equation 15 has been chosen for obtaining peak pressure ratio equals to 0.8) that it is not
554 taken into account by other formulations. Obviously, the empirical coefficient k^* should be reviewed increasing field
555 data in order to reach more robust statistical analyses.

556

557 **Fig. 14** Comparison between the proposed model (black line) and others hydro-dynamic (Hungr et al. 1984 and Canelli
558 et al. 2012) and mixed (Arattano and Franzi 2003, Huebl and Holzinger 2003, Lamberti and Zanuttigh 2004, Armanini
559 et al. 2011, Cui et al. 2015 and Vagnon and Segalini 2016) models.

560

561 **DISCUSSION AND CONCLUSIONS**

562 In this paper, sixteen debris flow impact models have been described and their predicting capability, or better their
563 capability to fit experimental and field data, has been evaluated using three different dataset: one coming from small-
564 scale flume tests, one from full-scale experiments and one from real data collected at Jiangjia Ravine (China) and
565 Illgraben (Switzerland) basins. At first sight, the small-scale dataset showed dimensionless pressure values shifted to
566 higher Froude numbers compared to those obtained from on-site dataset. This difference is a consequence of the scale
567 effects that affect the small-scale tests and probably due to the influence of the triggering mechanism.

568 This study is a first attempt to compare the most famous debris flow impact models, analysing their strong points and
569 limitations and evaluating their capability of fitting experimental and field data for helping designers in the choice of
570 the best models to design mitigation measures (Kwan 2012).

571 For the sake of simplicity and for a direct comparison between the described methods, the models have been classified
572 into three groups: hydro-static, hydro-dynamic and mixed models. For each model, key points and limitations have been
573 highlighted and the main findings can be summarized as follow:

- 574 1. Hydro-static models require few input parameters (flow density and thickness) for evaluating impact pressure on
575 structures. This aspect is particularly important for what it concerns the level of uncertainties coming from the
576 whole debris flow scenario: since these parameters can be easily evaluated analysing past events, the result
577 variability depends mainly by the dimensionless coefficient k . By contrast, the performed analyses have shown
578 that the predicting capability reached an acceptable level of safety for Froude number lower than 3. Moreover, the
579 predicting capability decreases of about the square of the Froude number, confirming that when the velocity of the
580 flow increases (and consequently the flow thickness decreases) these models are not able to predict impact load.
581 The performed statistical analysis also confirmed the limited suitability of these models.
- 582 2. Hydro-dynamic models provide impact pressure considering the flow density and the square velocity of the flow.
583 The latter parameter is particularly difficult to measure during debris flow event and for this reason the related
584 uncertainties can result high. As highlighted above, except for those models with α lower than 1 (Bugnion et al.,
585 2011), hydro-dynamic formulations have a good capability to predict and overestimate impact pressure especially
586 for high Froude numbers (predicting capability is low when Fr is lower than 2). However, the main limitation is
587 the excessive overestimation in predicting impact load that may results in a large increment of costs for structure
588 construction. A dynamic coefficient equal to 2 is suggested, as a good compromise between predicting capability
589 and excess of overestimation.
- 590 3. Mixed models seem to be best methods for predicting debris flow impact pressure on barriers, since they include
591 both information about the static and the dynamic component of the flow. The increase in the numbers of
592 parameters increases the uncertainties and, consequently, the degree of reliability of these methods decreases.

593 The main hypothesis behind the described methods is that the entire load is totally transferred to the structure, without
594 any dissipation during the impact. In terms of barrier design, this hypothesis should lead to over-conservative design
595 since stiffness and drainage capability are not taken into account. Analysing field results, the overestimation induced by
596 this hypothesis is not always verified probably due to the hit of single boulders on the barrier, condition that required
597 the introduction of specific equations (Kwan 2012; Faug 2015; Ng et al. 2016; Song et al. 2017) or the increase of the
598 dimensionless coefficient (k and α).

599 Finally, the model proposed in this paper exhibits a good capability to predict impact load. It is able to take into account
600 both the static and the dynamic behaviour of the flow without being affected by of the influence of flow regime. Further
601 monitoring field data will be helpful eventually to review the statistics at the basis of this new formulation and to

602 improve its predicting capability. Moreover, additional monitoring debris flow systems would be very welcome to
603 improve the knowledge about these disastrous phenomena and help to design mitigation measures with increasing level
604 of safety and reliability.
605 It is expected that the results proposed in this paper will be useful for designer, helping them for the best choice of
606 debris flow impact models on barriers.

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815 the Swiss Alps. *Catena Suppl* 22:59–72.
816

817 **List of Figures**

818 **Fig. 1** Different types of debris flow active mitigation measures: check dam (photograph by Los Angeles County Flood
819 Control Distric) (a), open type sabo structures (photograph of steel check dam in Nagano prefecture, Japan) (b),
820 concrete slit barrier (photograph by LCW Consult web site of protection works in St. Luzia River, Madeira, Portugal)
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822 Basilicata Region, Italy) (d).

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824 **Fig. 2** Comparison between normalized debris flow impact force and hydro-static predicting models as function of
825 Froude number considering small- (a) and full-scale experiments (b) and field data (c).

826

827 **Fig. 3** Relationship between measured peak pressure and calculated hydro-static peak pressure with k respectively equal
828 to 2.5 (a to c) and 7.5 (d to f) as a function of the Froude number considering both small- (a, d) and full- scale (b, e) and
829 field dataset (c, f). The green continuous line represents the perfect correspondence between measured values and
830 estimated ones.

831

832 **Fig. 4** Comparison between normalized debris flow impact force and hydro-dynamic predicting models as function of
833 Froude number considering small- (a) and full-scale tests (b) as well as field data (c).

834

835 **Fig. 5** Relationship between measured peak pressure and calculated hydro-dynamic peak pressure with α respectively
836 equal to 0.4 (a to c) and 12 (c to d) as a function of the Froude number considering both small- (a, d) and full-scale (b,
837 e) and field dataset (c, f)..

838

839 **Fig. 6** Relationship between velocity (blue squares) and thickness (red diamonds) as function of Froude number for
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841 Froude number increases, velocity increases and consequently flow height decreases and vice-versa.

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845

846 **Fig. 8** Relationship between measured peak pressure and calculated peak pressure for different mixed models as a
847 function of the Froude number considering field dataset (a to f).

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849 **Fig. 9** Predicting capability analysis of hydro-static models using field dataset.

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854

855 **Fig. 12** Relationship between field measurements of the peak pressure and calculated peak pressure using hydro-static
856 formulation with $k=1$ as a function of the Froude number of the flow

857

858 **Fig. 13** Relationship between peak pressure ratio for the modified hydro-static model as a function of the Froude
859 number (a) and statistical evaluation of the predicting capability of the proposed model (b).

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861 **Fig. 14** Comparison between the proposed model (black line) and others hydro-dynamic (Hungry et al. 1984 and Canelli
862 et al. 2012) and mixed (Arattano and Franzini 2003, Huebl and Holzinger 2003, Lamberti and Zanuttigh 2004, Armanini
863 et al. 2011, Cui et al. 2015 and Vagnon and Segalini 2016) models.

864

| Author | Coefficient | Data source | Notes |
|-------------------------------|---------------------------|---|----------------------|
| Lichtenhahn (1973) | 3.5 – 5.5 | Theoretical and construction experience | Hydro-static formula |
| Armanini (1997) | 4.5 | Theoretical and laboratory experiments | Hydro-static formula |
| Scotton and Deganutti (1997) | 2.5 – 7.5 | Laboratory experiments | Hydro-static formula |
| Watanabe and Ikeya (1981) | 2.0 – 4.0 | Field measurements of volcanic mud flows | Hydro-dynamic model |
| Daido (1993) | 5 – 12 | Analytical results | Hydro-dynamic model |
| Bugnion et al. (2011) | 0.4 – 0.8 | Measurements of generated hillslope debris flow at Canton Aargau, Switzerland | Hydro-dynamic model |
| Canelli et al. (2012) | 1.5 – 5.5 | Laboratory experiments and field measurements | Hydro-dynamic model |
| Hungr et al. (1984) | 1.5 | Back-analysis data in British Columbia, Canada | Hydro-dynamic model |
| Zhang (1993) | 3.0 – 5.0 | Field measured data in Jiangjia ravine, China | Hydro-dynamic model |
| Huebl and Holzinger (2003) | 5 | Field and laboratory experimental data | Mixed model |
| Cui et al. (2015) | 5.3 | Field and laboratory experimental data | Mixed model |
| Cross (1967) | $k = 0.5$ $\alpha = 3$ | Tsunami wave pressure | Mixed model |
| Arattano and Franzi (2003) | 1 | Field measured data in Moscardo Torrent, Italy | Mixed model |
| Lamberti and Zanuttigh (2004) | 1.5 | Theoretical and laboratory experiments | Mixed model |
| Armanini et al. (2011) | 1 | Theoretical and laboratory experiments | Mixed model |
| Vagnon and Segalini (2016) | 0.5 – 1.2 | Theoretical and laboratory experiments | Mixed model |

Table 2 – Summary of the main dataset characteristics.

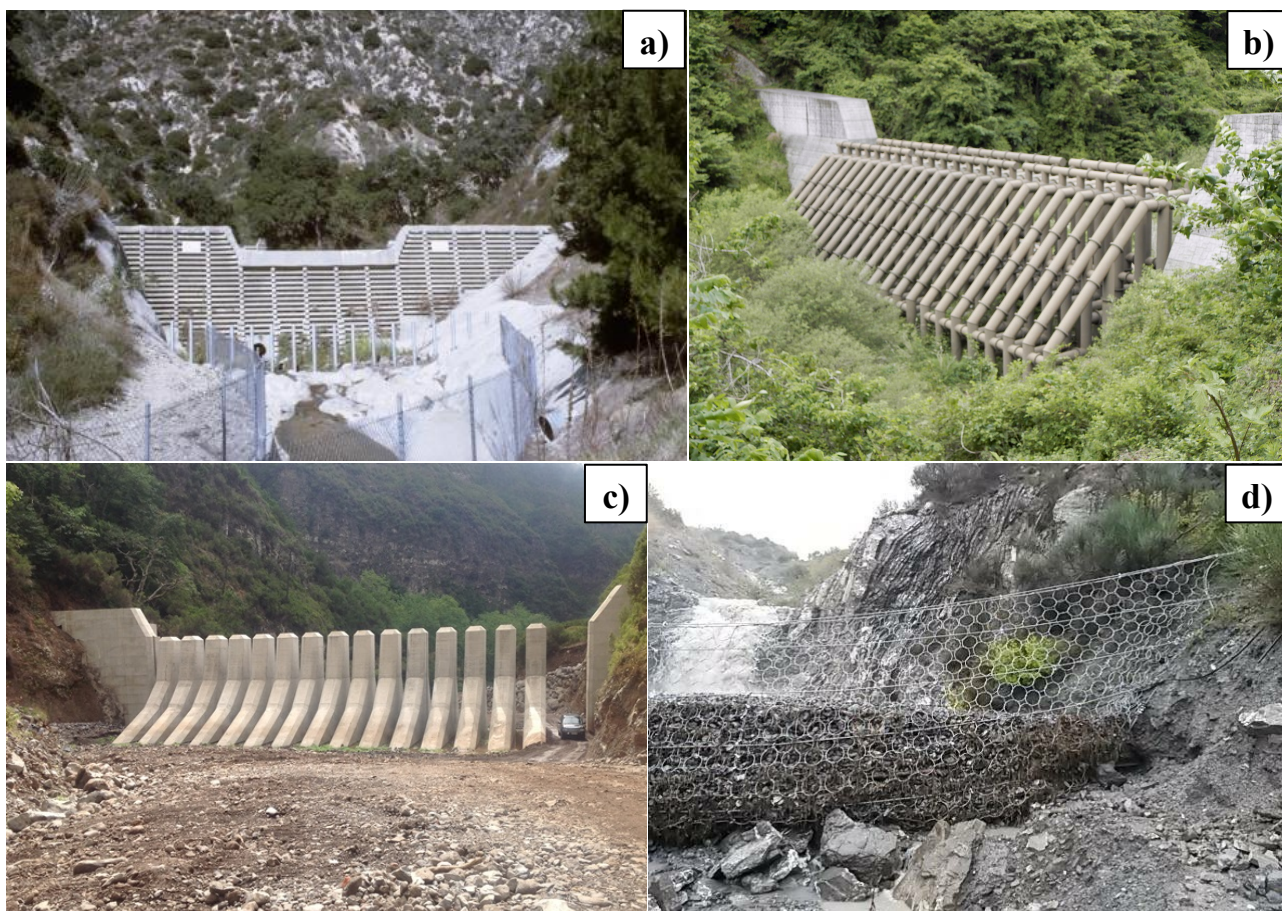
| | Dataset | Apparatus/Basin | Material | Measured Physical Quantities | Dimension of the dataset |
|-------------------------|--------------------------|--|--|---|---|
| Small-scale experiments | Scheidl et al. 2013 | Wood flume with constant inclination of 30° and measuring section 4.5 m long and 0.45 m wide. The reservoir section can release a volume equals to 0.33 m ³ of mixture. | Mixture with constant dry mass and variable water content (from 0.16 to 0.27). The grain size distribution ranges between 0.0002 and 50 mm | Flow velocity, impacting height and horizontal impact forces recorded in real time during the experiments. | 30 experiments but only 16 selected for further analyses |
| | Cui et al. 2015 | Steel flume 0.2 m wide and 3 m long with slope ranging from 10° to 15°. | Material collected in the Jiangjia Ravine basin (China) with grain size distribution varying between 0.001 and 10 mm. The liquid concentration varies from 0.34 to 0.76. | Flow velocity, impacting height and pressure recorded in real time during the experiments. | 27 tests with different density (1600-2300 kg m ⁻³) and slopes (10-13°). |
| | Ashwood and Hungr 2016 | Steel flume 0.3 m wide and 3 m long with slope ranging from 22° to 34°. | Uniform quartz sand and pea gravel | Flow velocity, impacting height, profile, barrier deflection and pressure recorded in real time during the experiments. | 28 tests with different density (1560-1780 kg m ⁻³) and slopes (122-34°). |
| | Vagnon and Segalini 2016 | Steel flume 4 m long and 0.39 m wide in which the slope is variable between 30° and 35° | Saturated sand with constant liquid concentration (0.4) and mixture density (1920 kg m ⁻³). The grain size distribution varies between 0.0001 and 5 mm. | Flow velocity, impacting height and impact forces recorded in real time during the experiments. | 63 test with different volume released and different slopes. |
| Full-scale experiments | DeNatale et al. 1999 | A 41m long, 8m wide channel constructed on the side of a rock quarry near Velthein, Switzerland. Average inclination of 30° | Mixture of soil, bedrock and water | Flow height at the middle of channel, velocity of upper flow surface, impact pressure on two different steel plates | 16 tests with single or multiple releases. Density varies between 1760 and 2110 kg/m ³ |
| | Bugnion et al. 2011 | USGS debris flow flume, 95m long, 2m wide and 1.2 deep, with constant slope of 31° | Poorly graded, clean and saturated gravelly sand | Flow height, velocity, impact pressure on flexible barrier | 6 experiments on flexible barriers |
| Field measurements | Wendeler et al. 2007 | Barrier system installed at the Illgraben basin, Switzerland | Muddy debris flow | Flow height, load cells, velocity from video record | May 18th 2006 event |
| | Hu et al. 2011 | Jiangjia Ravine basin, located near the city of Dongchuan (China). the basin has an area of 48.6 km ² and the mainstream has a length of 13.9 km. | The bulk density ranges from 1600 to 2300 kgm ⁻³ with fluid concentration ranging from 0.15 to 0.6. The dimension of the solid particles varies between 0.001 and | Flow velocity, impacting height and impact forces recorded in real time during debris flow events. | 38 surges occurred on August 25, 2004 after short intense rainfall |

100 mm.

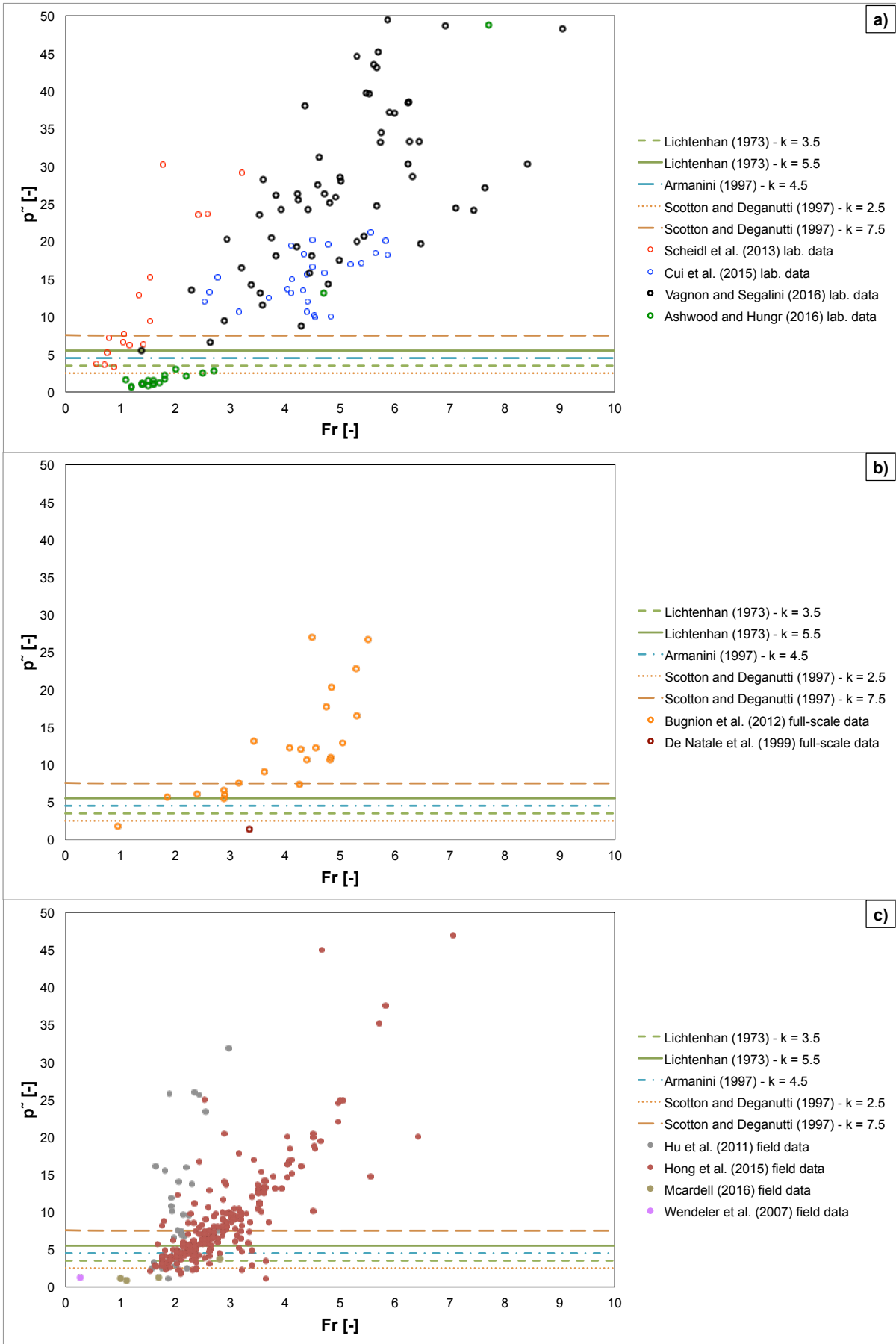
| | | | | |
|------------------|--|--|--|---|
| Hong et al. 2015 | Jiangjia Ravine basin, located near the city of Dongchuan (China). the basin has an area of 48.6 km ² and the mainstream has a length of 13.9 km. | The bulk density ranges from 1600 to 2300 kgm ⁻³ with fluid concentration ranging from 0.15 to 0.6. The dimension of the solid particles varies between 0.001 and 100 mm. | Channel width, flow velocity, impacting height, density, duration and impact forces recorded in real time during debris flow events. | 139 events during 1961 and 2000 |
| McArdell 2016 | Illgraben basin, Switzerland | Quartzite and dolomite boulders, clast-size particles | Flow height, load cells, velocity from video record | 29 July, 8 August and 24 August 2013 events |

Table 3 – Evaluation of the peak pressure prediction capability for debris flow impact models.

| Model type | Equation | Empirical coefficient [-] | Percentage of overestimated peak pressure values [%] | | |
|------------------------------|-------------------------------|------------------------------|--|--------------------|-------------------|
| | | | Small-scale dataset | Full-scale dataset | Field measurement |
| Hydro-static models | Lichtenhan (1973) | 3.5 | 14.2 | 8.3 | 14.6 |
| | | 5.5 | 17.2 | 12.5 | 35.6 |
| | Armanini (1993) | 4.5 | 15.7 | 8.3 | 25.4 |
| | Scotton and Deganutti (1997) | 2.5 | 11.2 | 8.3 | 5.4 |
| | | 7.5 | 20.9 | 33.3 | 54.7 |
| Hydro-dynamic models | Watanabe and Ike (1981) | 2 | 82.1 | 100 | 91.2 |
| | | 4 | 91 | 100 | 97.6 |
| | Hungr et al. (1984) | 1.5 | 72.4 | 91.7 | 86.1 |
| | Daido (1992) | 5 | 93.3 | 100 | 99 |
| | | 12 | 100 | 100 | 98.7 |
| | Zang (1993) | 3 | 88.1 | 100 | 95.3 |
| | | 5 | 93.3 | 100 | 99 |
| | Bugnion et al. (2011) | 0.4 | 0 | 4.2 | 2 |
| | | 0.8 | 34.33 | 66.7 | 16.9 |
| | Canelli et al. (2012) | 1.5 | 72.4 | 91.7 | 86.1 |
| | | 5.5 | 93.3 | 100 | 99 |
| Mixed models | Cross (1967) | $k = 1; \alpha = 3$ | 88.1 | 100 | 94.3 |
| | Arattano and Franzi (2003) | 1 | 48.5 | 83.3 | 66.8 |
| | Huebl and Holzinger (2003) | | 38.1 | 79.2 | 80.9 |
| | Lamberti and Zanuttigh (2004) | | 64.2 | 100 | 87.9 |
| | Armanini et al. (2011) | 1 | 70.1 | 91.7 | 76.8 |
| | Cui et al. (2015) | | 20.9 | 54.2 | 65.4 |
| | Vagnon and Segalini (2016) | 1.2 | 85.1 | 87.5 | 84.6 |
| Number of small-scale tests | | | 134 | | |
| Number of small-scale tests | | | 24 | | |
| Number of field measurements | | | 298 | | |



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875 **Fig. 1** Different types of debris flow active mitigation measures: check dam (photograph by Los Angeles County Flood
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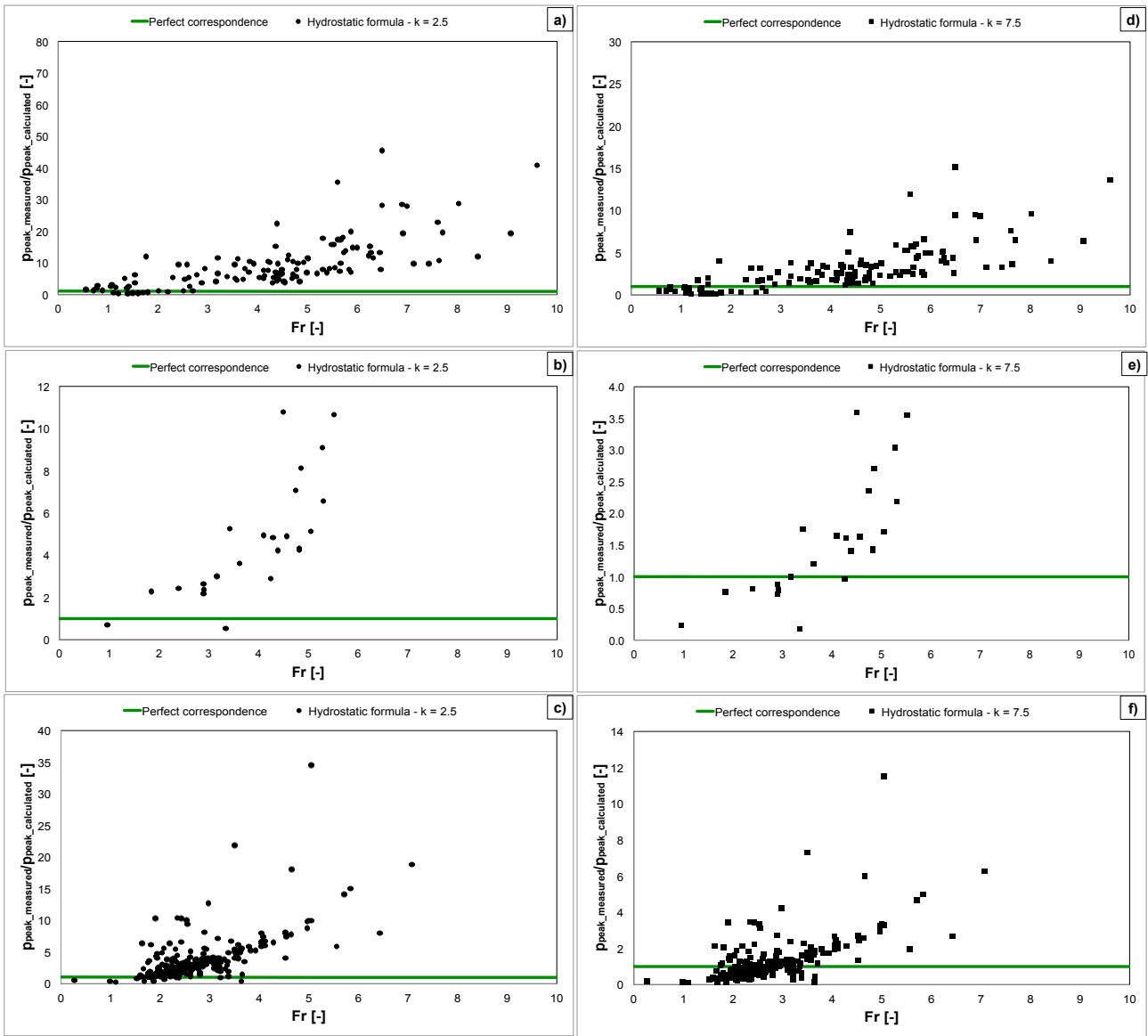


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Fig. 2 Comparison between normalized debris flow impact force and hydro-static predicting models as function of Froude number considering small- (a) and full-scale experiments (b) and field data (c).



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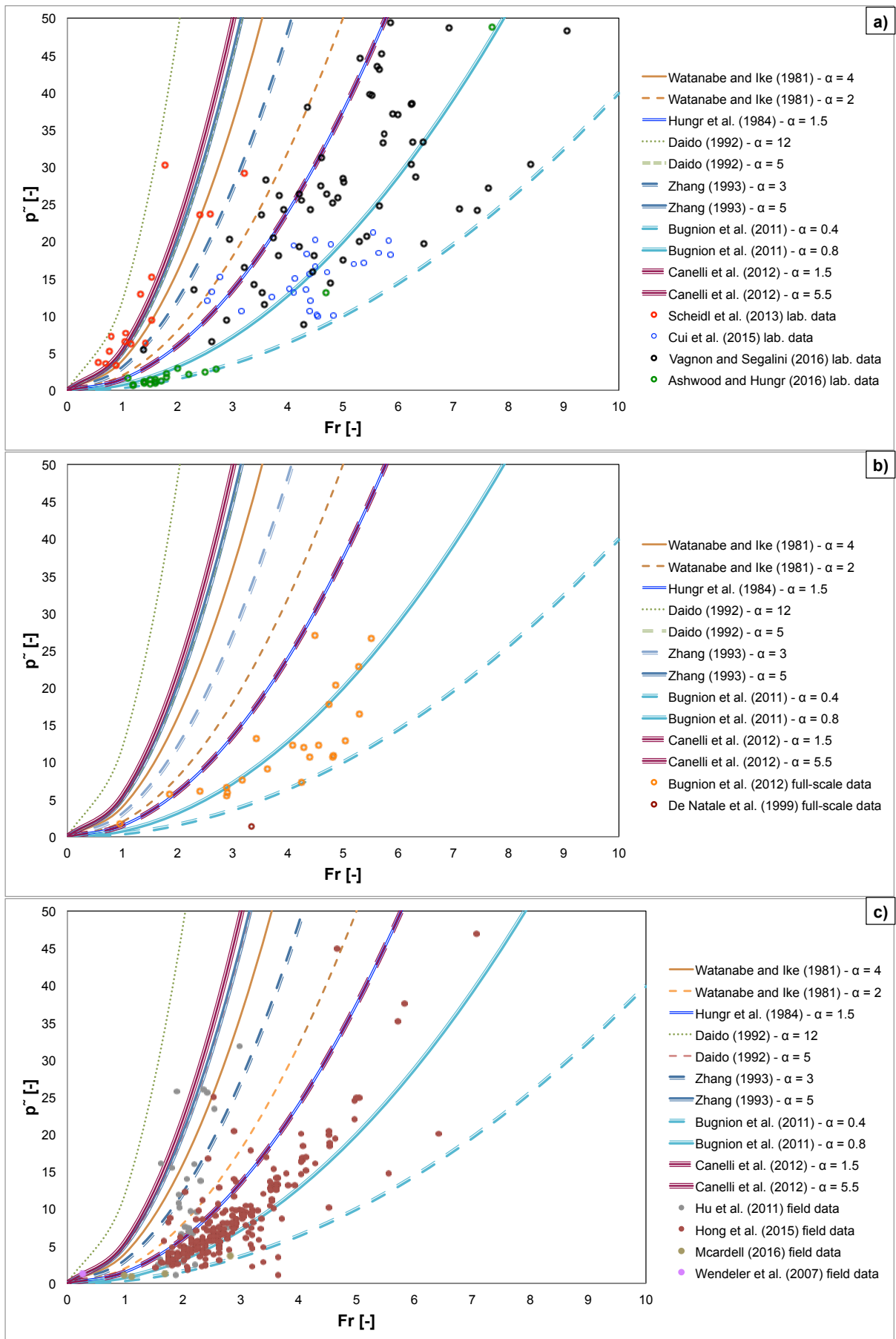
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Fig. 3 Relationship between measured peak pressure and calculated hydro-static peak pressure with k respectively equal to 2.5 (a, to c) and 7.5 (d to f) as a function of the Froude number considering both small- (a, d) and full- scale (b, e) and field dataset (c, f). The green continuous line represents the perfect correspondence between measured values and estimated ones.

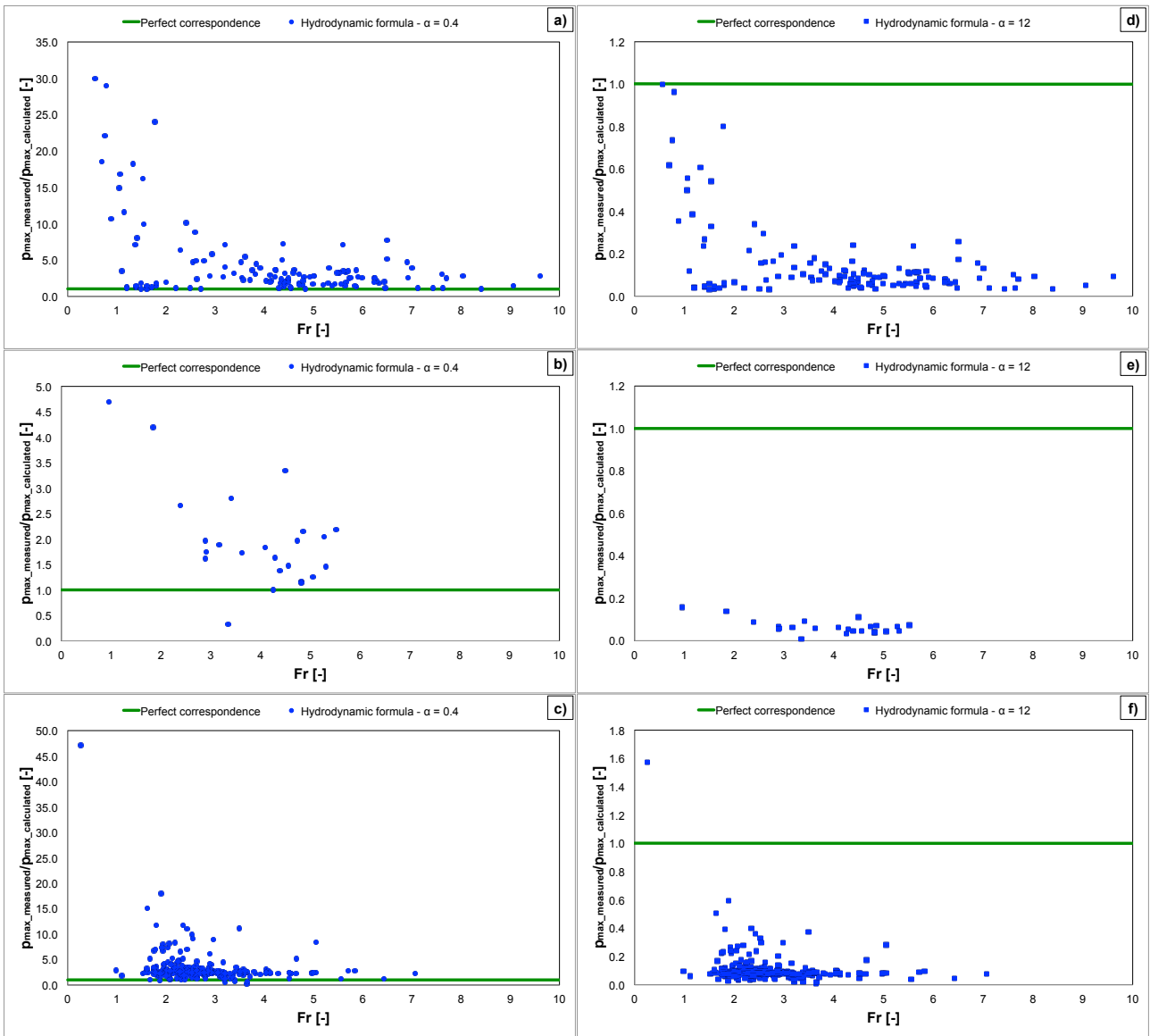


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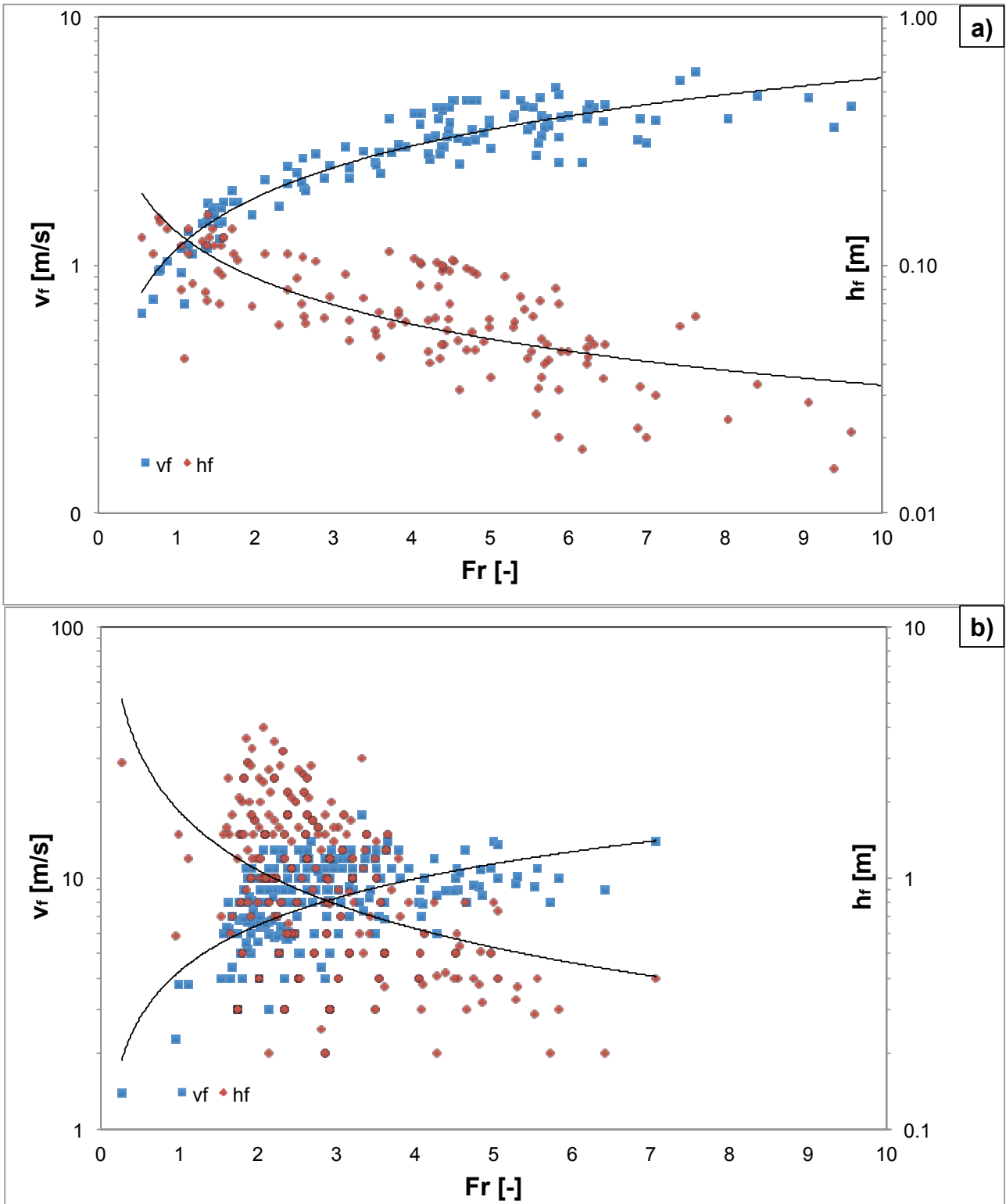
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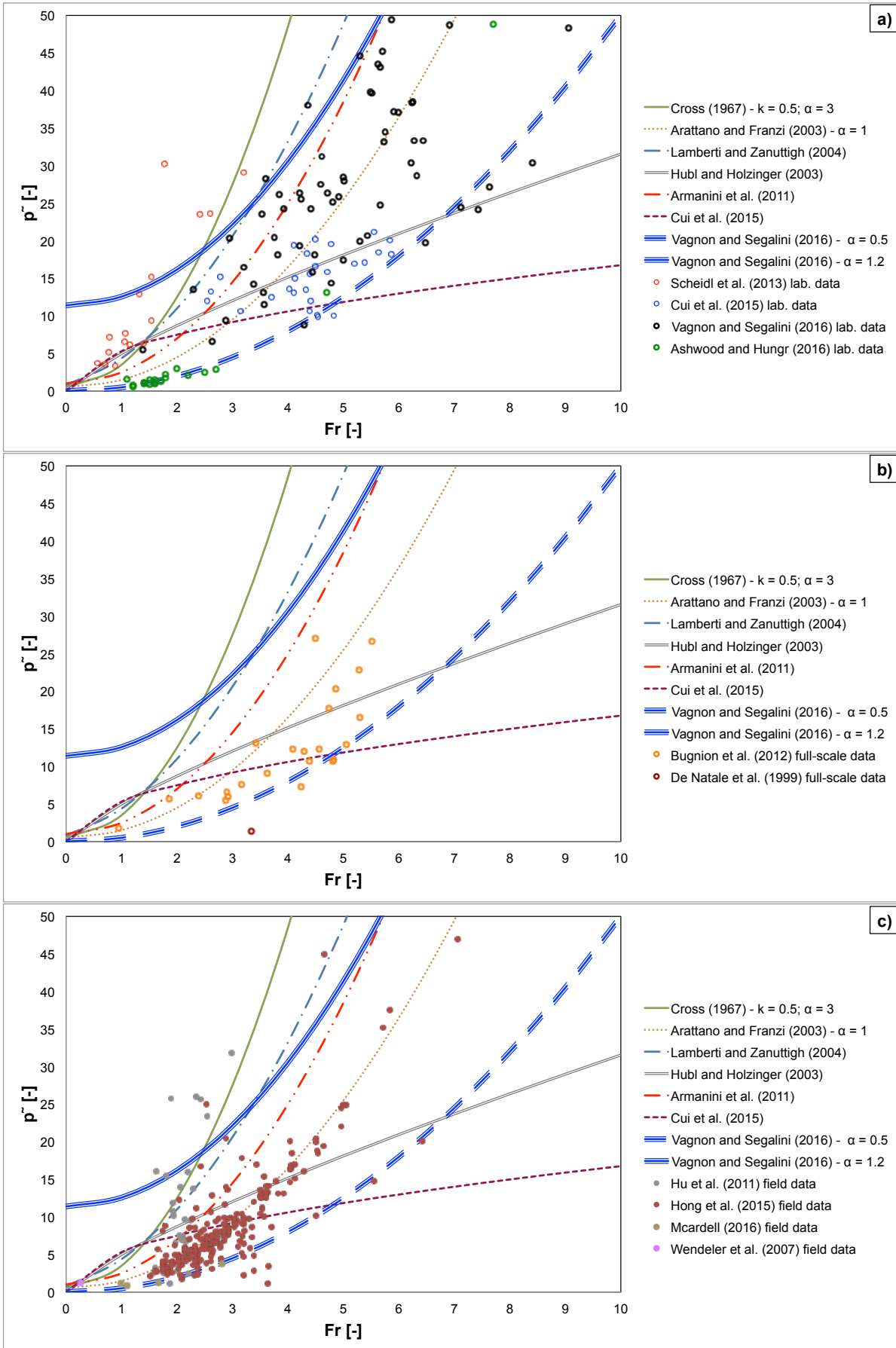
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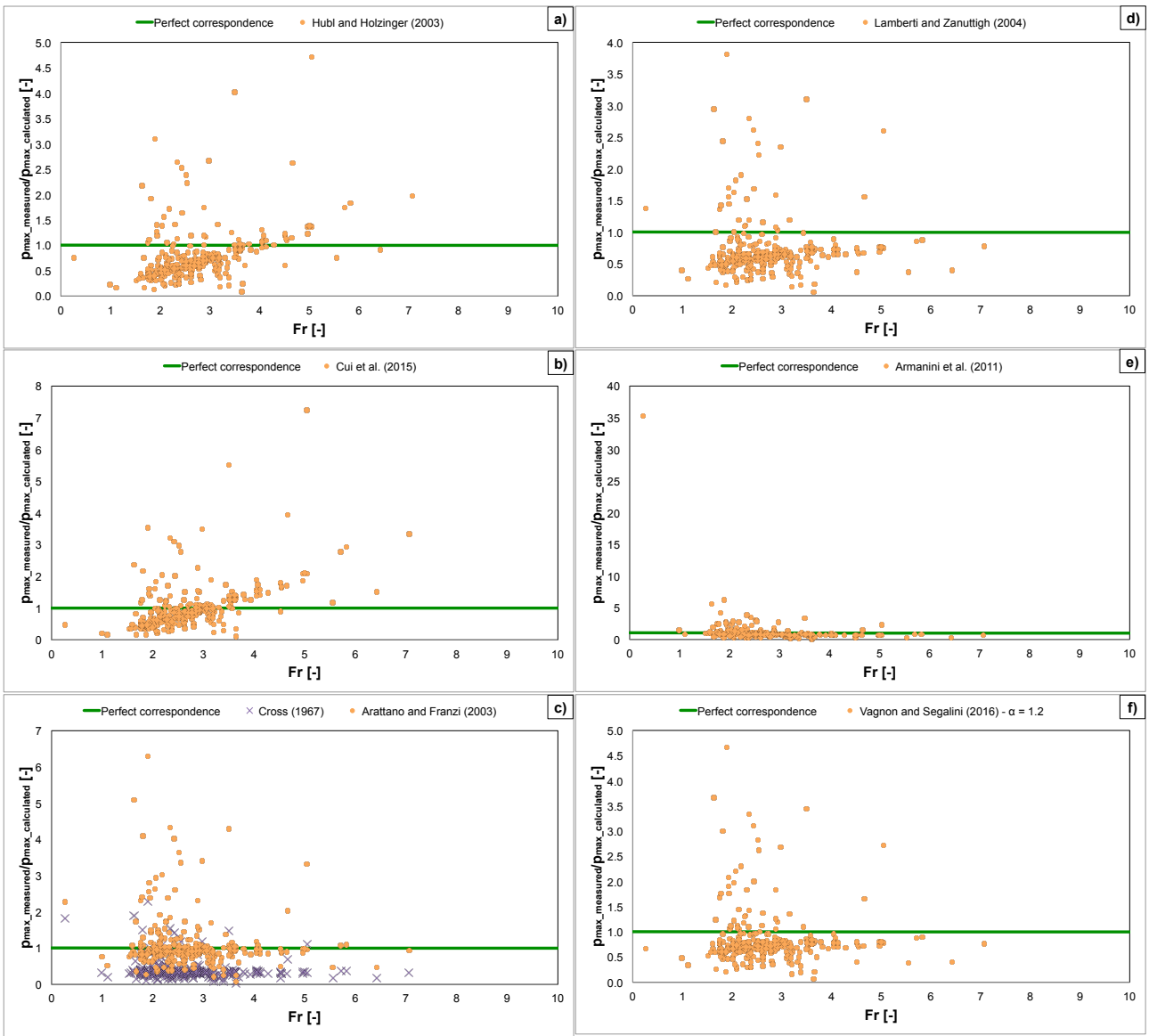


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Fig. 7 Comparison between normalized debris flow impact force and mixed predicting models as function of Froude number considering small- (a) and full-scale test (b) as well as field data (c).



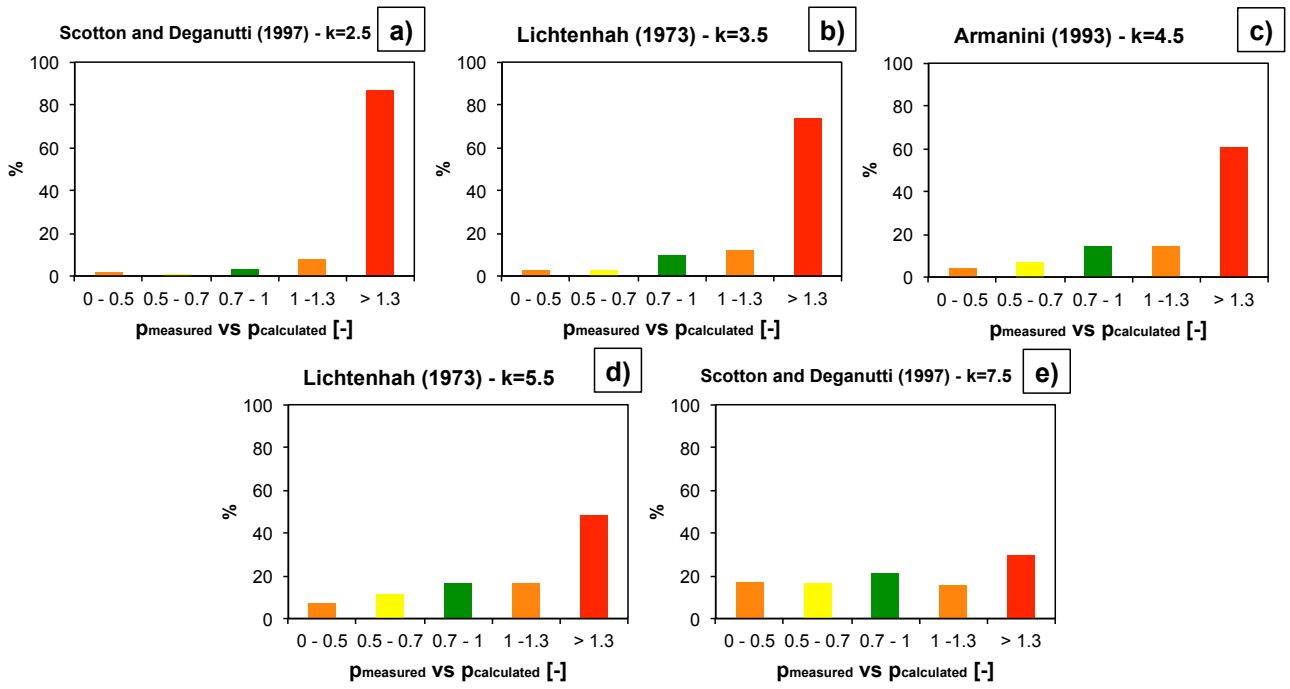
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Fig. 8 Relationship between measured peak pressure and calculated peak pressure for different mixed models as a function of the Froude number considering field dataset (a to f).

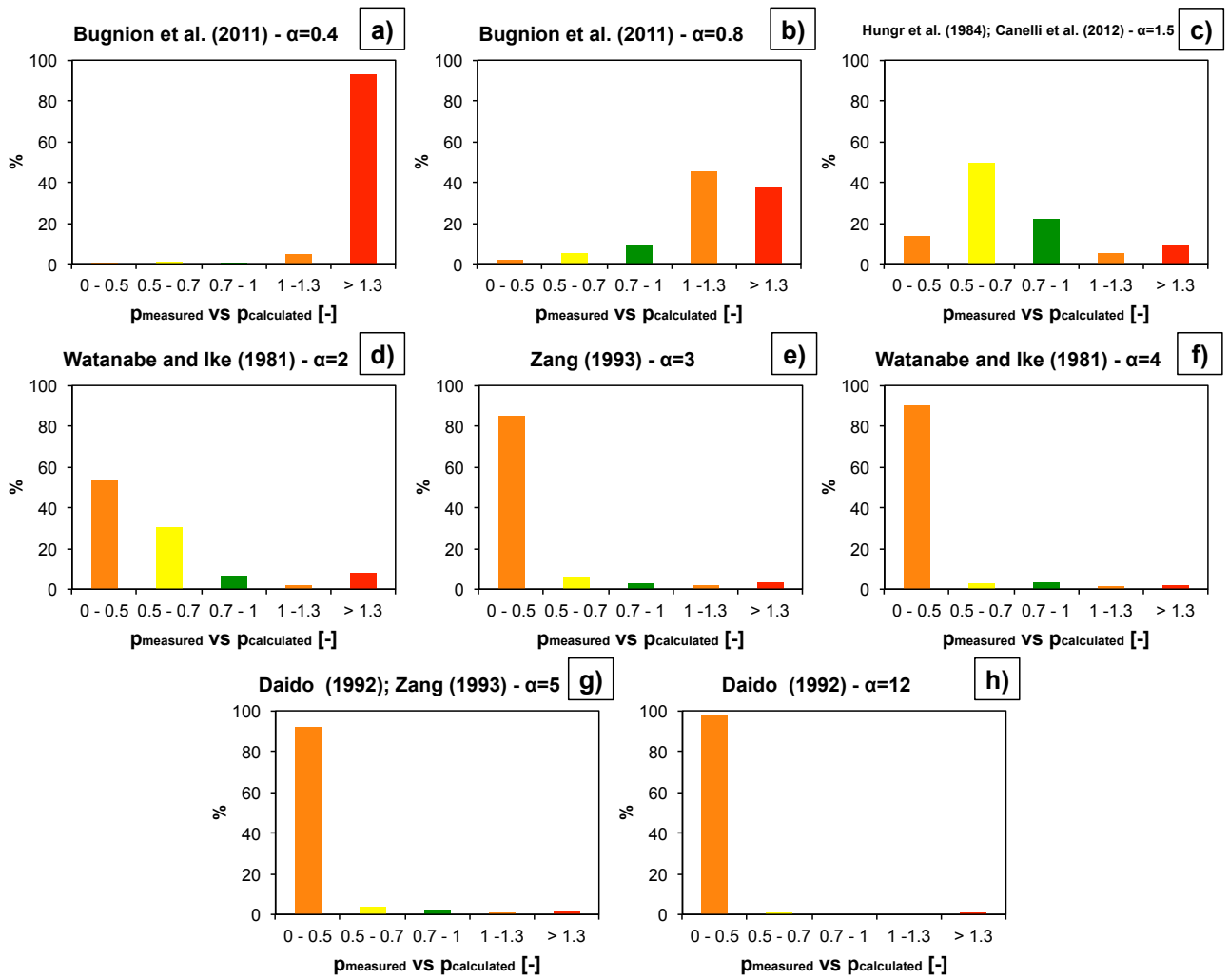
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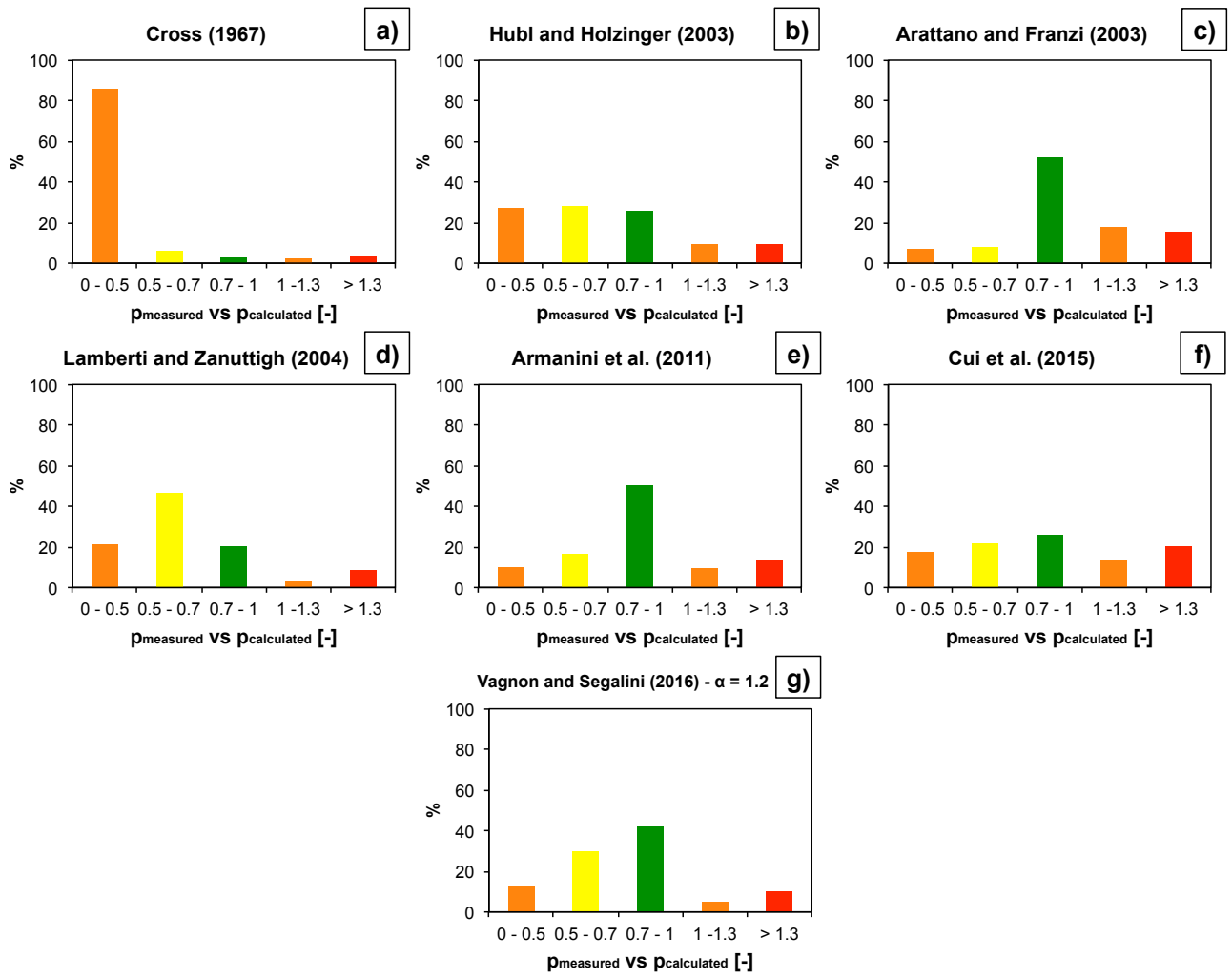


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Fig. 9 Predicting capability analysis of hydro-static models using field dataset.



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 914 **Fig. 10** Predicting capability analysis of hydro-dynamic models using field dataset.
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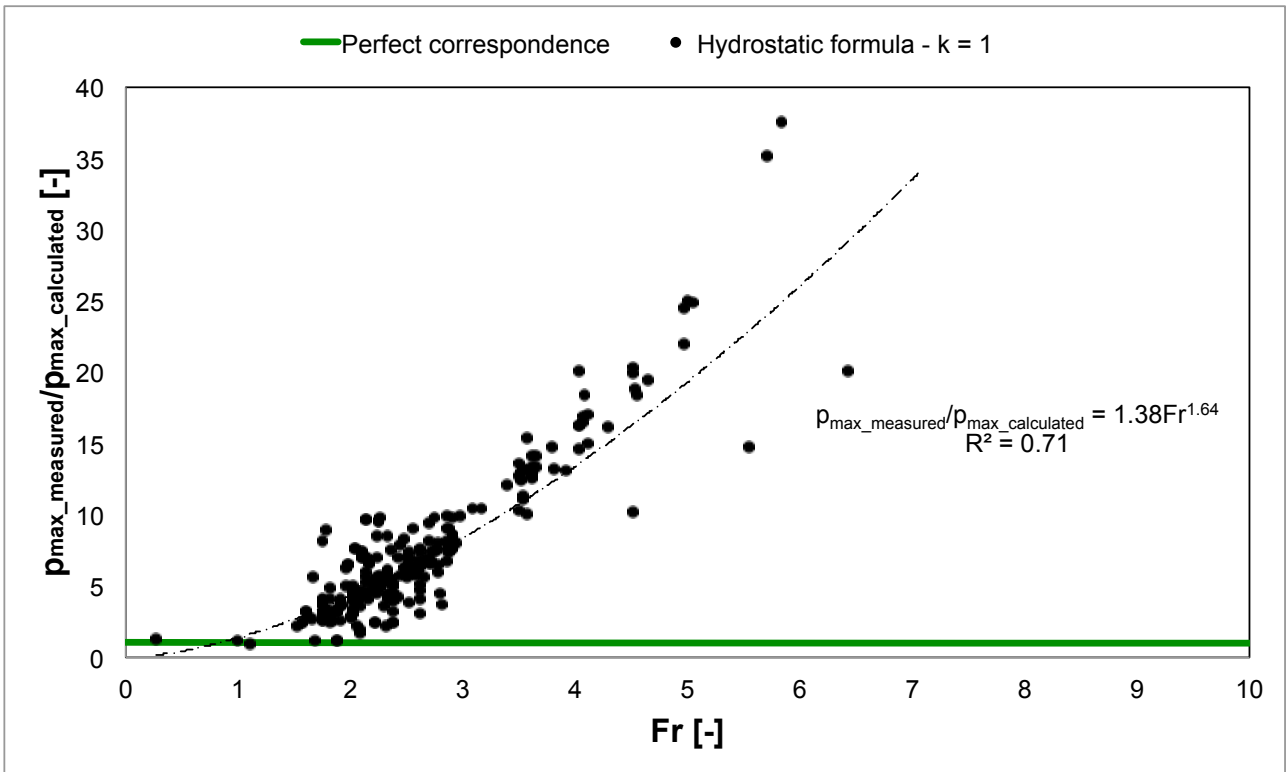


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917 **Fig. 11** Predicting capability analysis of mixed models using field dataset.

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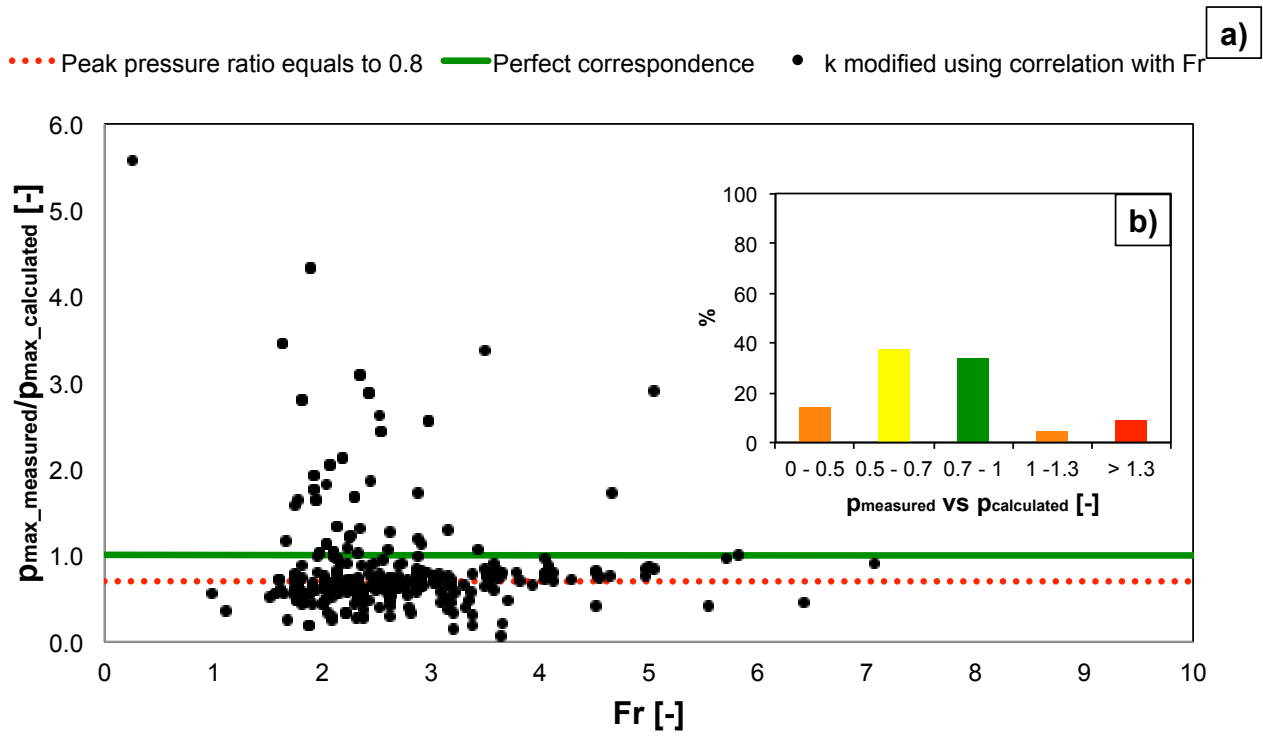
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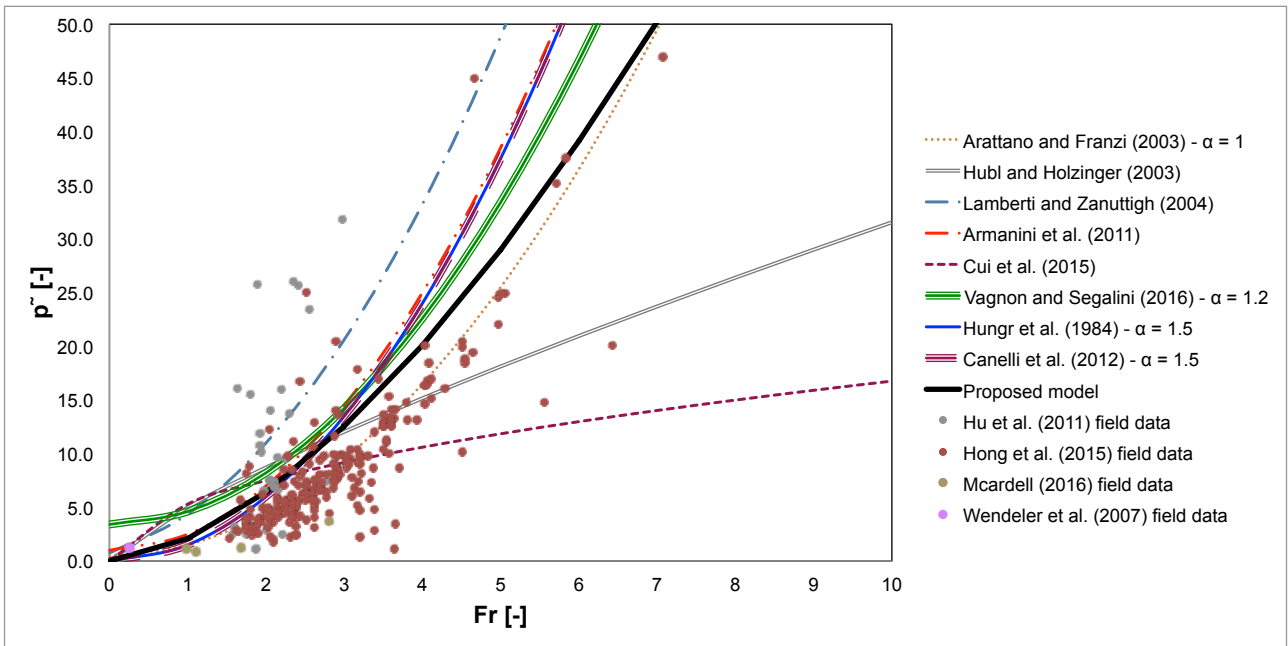


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926 number (a) and statistical evaluation of the predicting capability of the proposed model (b).

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Fig. 14 Comparison between the proposed model (black line) and others hydro-dynamic (Hungr et al. 1984 and Canelli et al. 2012) and mixed (Arattano and Franzì 2003, Huebl and Holzinger 2003, Lamberti and Zanuttigh 2004, Armanini et al. 2011, Cui et al. 2015 and Vagnon and Segalini 2016) models.