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2 **Development and validation of the Terrain Stability model for assessing landslide**
3 **risk during heavy rain infiltration**

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14 **Abstract**

15 Slope stability is a key topic, not only for engineers but also for politicians, due to the
16 considerable monetary and human losses that landslides can cause every year. In fact, it is
17 estimated that landslides have caused thousands of deaths and economic losses amounting to
18 tens of billions of euros per year around the world. The geological stability of slopes is affected
19 by several factors, such as climate, earthquakes, lithology and rock structures, among others.
20 Climate is one of the main factors, especially when large amounts of rainwater are absorbed in
21 short periods of time. Taking into account this issue, we developed an innovative analytical
22 model using the limit equilibrium method supported by a geographic information system (GIS).
23 This model is especially useful for predicting the risk of landslides in scenarios of heavy
24 unpredictable rainfall. A hydrological steady-state assumption was incorporated into this
25 approach. The model, called Terrain Stability (TS), was developed and programmed in
26 MATLAB. This model allows a simulation of the slope stability in a 2D spatial distribution. Many
27 variables measured in the field – topography, precipitation, type of soil – can be added,
28 changed or updated using simple input parameters. To validate the model, we applied it to a
29 real example, that of a landslide which resulted in human and material losses (collapse of a
30 building) at Huididero, La Viñuela (Málaga) in February 2010.


31 **Keywords:** Emergencies, stability, rainfall, landslides, critical

32


33 **1. Introduction**





34 In the 1980s, most stability analyses were performed by graphical methods or by using
35 manual calculators. Nowadays, the quickest and most detailed analyses can be performed
36 using any ordinary computer [Wilkinson et al., 2002]. Currently, the vast majority of stability
37 analyses are performed with commercial software like Slide, SLOPE/W, Phase2, GALENA,
38 GSTABL7 and GEO5 [Gonzalez de Vallejo et al., 2002]. These programmes are used to follow




39 breakage and circular failures, even reaching as far as the cliff toe, to provide the critical FOS.
40 However, these programmes have some limitations. First, they require a payment license and
41 can be expensive. **Second, their calculations are restricted to limited initial parameters which**
42 **are, in general, not very interactive.** 

43

44 For the stability analysis, different approaches can be used, such as the limit equilibrium
45 methods [Cheng et al., 2007; Liu et al., 2015], the finite elements method [Griffiths et al., 2007;
46 Tschuchnigg et al., 2015; Griffiths, 2015] and the dynamic method [Jia et al., 2008], among
47 others. Limit equilibrium methods are well known, and their use is simple and quick. **These**
48 **methods allow us to analyse almost all types of landslides, such as translational, rotational,**
49 **tackle, creep and fall, among others.**  limit equilibrium methods can be combined with
50 probabilistic techniques [Stead et al., 2000] or with other models, like stability analysis of
51 coastal erosion [Castedo et al., 2012]. However, they are limited in general to 2D planes and
52 easy geometries. Numerical methods – finite elements methods – give us the most detailed
53 approach to analysing the stability conditions for the majority of evaluation cases, including
54 complex geometries and 3D cases. Nevertheless, they present some problems, such as their
55 complexity, data introduction, mesh size effect and the time and resources they require
56 [Ramos Vásquez, 2017].

57 Software such as the programmes mentioned above provide useful tools for determining the
58 stability FOS  for giving the most probable breakage (shearing) surfaces. This technique is
59 fast and allows the field or emergency engineer to make timely decisions. Although this
60 methodology is only available in some current software (Slide V 5.0, **STB 2010, Geo-Slope**) 
61 based on limit equilibrium methods, it is highly recommended because of its reliability for
62  representing real conditions in the field [Chugh, 1981]. **While these software programmes can** 
63 **reproduce the actual water table, they cannot reproduce the effect of matrix suction caused**
64 **by rain infiltration [Herrada et al., 2014].** This rain infiltration produces a substantial reduction
65 of cohesion (a key soil parameter for stability) that cannot be reproduced by actual software
66 and then several real situations cannot be predicted.

67 Delft University has developed a well-known and free software programme to analyse
68 landslides, the STB 2010 [Verruijt, 2010]. This programme is based on a limit equilibrium
69 technique, using a modified version of Bishop's method to calculate the FOS only for circular
70 failures. It is a user-friendly tool, but it does not allow the calculation of water infiltration on a
71 hillside. This is a critical point, as it is well known that rainfall infiltration is one of the main
72 causes of landslides worldwide [Michel et al., 2015]. Reviewing these issues, a new solution
73 must be developed for cases where landslides are linked to heavy rainfall. In this study, we
74 developed a new model and programmed it using MATLAB. The primary result of this model
75 was a stability index, namely the minimum **FOS, F_s**  based on the limit equilibrium technique,
76 in this case the Bishop's method. The model also provides a possible failure curve, which can
77 be used to coincide with analysis of the actual event as tested with field data. Topographical
78 data can also be introduced into the model from the digital elevation model (DEM) in a GIS.

79



80 2. Terrain Stability model development

81 In the model we developed, we used the limit equilibrium technique for its versatility,
 82 calculation speed and accuracy. An analysis can be done studying the whole length of the
 83 breakage (shearing) zone or just small slices. Starting with the original method of slides
 84 developed by Petterson and Fellenius (1936), some methods are more accurate and complex
 85 [Spencer 1967, Morgenstern and Price, 1965] than others [Bishop, 1955 and Janbú, 1954].
 86 Using Spencer's method [Spencer, 1967; Chung 1986] here would mean dividing our slope into
 87 small slices that must be computed together. This method is divided into two equations, one
 88 related to the balance of forces and the other to momentum. Spencer's method imposes an
 89 equilibrium not only for the forces but also for the momentum on the surface of the rupture. If
 90 the forces for the entire soil mass are in equilibrium, the sum of the forces between each slice
 91 must be also equal to zero. Therefore, the sum of the horizontal forces between slices must be
 92 zero as well as the sum of the vertical ones (equations 1 and 2).

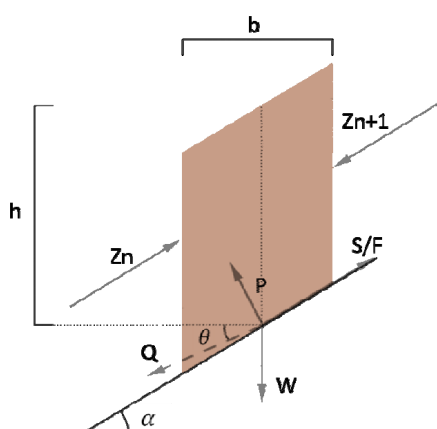
$$93 \quad \sum [Q \cos \theta] = 0 \quad 1$$

$$94 \quad \sum [Q \sin \theta] = 0 \quad 2$$

95 In this equation, Q is the resultant of the pair of forces between slices, and θ is the angle of the
 96 resultant (Figure 1). From this, it can be stated that the sum of the moments of the forces
 97 between slices around the critical rotation centre is zero:

$$98 \quad \sum [QR \cos(\alpha - \theta)] = 0 \quad 3$$

99 This takes into account that the sliding surface is considered circular, so the radius of the
 100 curvature is constant and can be extracted.



101

102 **Figure 1.** Representation of the forces acting on a slice, considered in Spencer's method [Spencer, 1967].
 103 W is the external vertical loads; Z_n and Z_{n+1} are the forces acting on the left- and right-hand side of each
 104 slice, respectively, with their horizontal and vertical components; P and S are the normal and tangential
 105 forces at the base of the slice.




106 These equations must be solved to get the FOS, (F_s), and tilt angles of the forces among the
107 slices (θ). To solve these equations, an iterative method is required until a limiting error is
108 reached. Once F_s and θ are calculated, the remaining forces are also obtained for each slice.
109 Spencer's method is considered very accurate and suitable for almost all kinds of slope
110 geometries and may be the most complete equilibrium procedure. It may also be the easiest
111 method for obtaining the FOS [Duncan and Wright, 2005]. Depending on the type of slope
112 analysed, this model is able to establish the failure curve following the typical rotational circle,
113 among other uses [Verruijt, 2010].

114

115 The FOS, F_s , classically defined as a ratio of stabilizing and destabilizing forces, determines the
116 stability of a slope as follows:

$$117 \quad F_s = \frac{\Sigma(\text{Forces standing against/oppose sliding})}{\Sigma(\text{Forces that induce sliding})} \quad 4$$

118


119 According to limit equilibrium methods, the two equilibrium conditions (forces and moments)
120 must be satisfied. ~~Spencer's method [Spencer, 1967] is more precise and simple in the TS~~
121 ~~model.~~ Taking into account these elements, the FOS is then obtained from the following
122 expression 

$$123 \quad F_s = \frac{1}{\Sigma W \sin \alpha} \Sigma [c' b \sec \alpha + \tan \phi' (W \cos \alpha - ub \sec \alpha)] \quad 5$$

124

125 According to equations (4) and (5), the slope FOS (F_s) can be considered unstable if its value is
126 lower than 1, or stable if it is higher than 1. It should be noted that, when applying the factor in
127 the engineering and architecture fields, the limiting value tends to be higher than 1, with
128 common values being 1.2 or even up to 1.5 [Burbano et al., 2009]. This is just a confidence
129 measure for your calculations. The FOS can also be defined as the ratio between the shear
130 strength (τ), based on the cohesion and the angle of friction values, and the shear stress, based
131 on the cohesion and the internal friction angle required to maintain the equilibrium (τ_{mb}).

132

133 As mentioned, the minimum FOS to consider a talus  stable is equal to 1. However, several
134 authors [Yong et al., 1977; van Westen and Terlien, 1996] suggest that the angle of a slope
135 would have to be defined by a value of the FOS superior to unity to take into account the
136 exogenous factors of the slope. Following Jimenez Salas [1981], a value of $F_s \geq 1.3$ can be
137 considered stable by most standards.

138

139 To analyse the slope using the Spencer's method, a set of equations must be solved to satisfy
140 the forces and momentum equilibrium and to obtain the F_s . The values of F_s and θ are the
141 unknowns that must be solved. Some authors suggest that the variation of θ can be arbitrary



142 [Morgenstern y Price, 1965], although the effect of these variations in the final value of F_s is
143 minimal. The variation of the angle depends on the soil's ability to withstand only a small
144 intensity of the shear stress.

145 Having said that, if we assume that the forces between slices are parallel (in other words, that
146 θ is constant), equations (1) and (2) become the same, resulting in:

147

$$148 \quad \Sigma Q = 0 \quad 6$$

149 The assumption that the forces between slices are parallel gives optimal results for the
150 calculation of the critical safety coefficients in equation 5 [Spencer 1967]. To solve these
151 equations, we used the FSOLVE function of the MATLAB software, giving an initial FOS and
152 angle. The FSOLVE function is a tool inside the optimization toolbox from MATLAB that solves
153 systems of nonlinear equations. When using this tool, an initial value must be provided to start
154 the calculation.

155

156 The application of a homogeneous pore pressure distribution (permanent interstitial pressure)
157 has been included in the model [Bishop and Morgenstern, 1960]. In this case, the permanent
158 interstitial pressure on the base of the slice was determined by the following expression:



159

$$160 \quad u = r_u \gamma h \quad 7$$

161 In this expression, u is the pore pressure (permanent interstitial pressure) at the base of the
162 slice, and the weight of it affects the W evaluation. The factor r_u is a coefficient of pore
163 pressure (interstitial pressure coefficient), which determines the rain infiltration factor on the
164 slopes. As it is well known, the water that infiltrates the soil may produce a modification of the
165 pore pressure, affecting its resistant capacity. This factor may vary from 0 (dry conditions) to
166 0.5 (saturated conditions). Finally, to obtain the pore pressure, u , the Coulomb equation is
167 needed:

168

$$169 \quad S = c + (\sigma - u) \tan \phi \quad 8$$

170 In this equation, S is the tangential shear resistance, c is the cohesion, σ is the normal stress
171 and ϕ is the internal friction angle. To introduce soil shearing parameters like cohesion or the
172 friction angle, a drained status must be considered [Duncan, 1996].

173

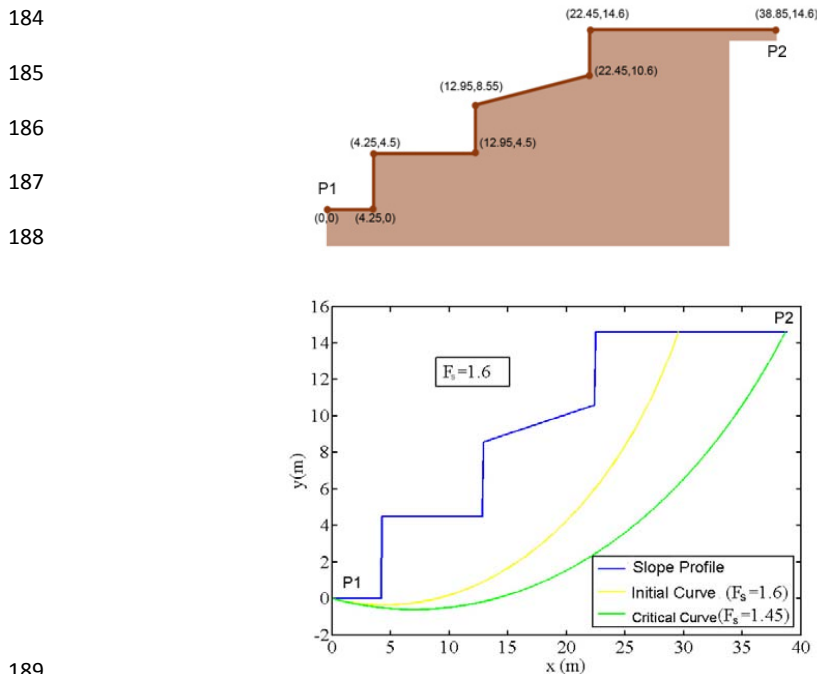
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176 **3. Terrain stability (TS) model behaviour tests**

177 Figure 2 shows the results of applying the Terrain Stability model to an irregular slope,
 178 including the initial and final points of the first failure circle (shown in yellow). This circle
 179 corresponds with the initial value introduced by the user into the FSOLVE function. The points
 180 of the talus are extracted from a DEM model in ArcGIS 10 [Glennon et al., 2008]. The talus
 181 height is equal to 15 m, and the soil is uniform with the following nominal properties: $\gamma =$
 182 19500 N/m^3 , $\phi = 22^\circ$, $c = 15000 \text{ N/m}^2$, $u = 0 \text{ N/m}^2$. According to the in situ
 183 the Geoner SL laboratory, soil is a silty clay from Gibraltar Flish.





189
 190 **Figure 2.** Results following the application of the software showing the talus profile and surface damage.
 191 The F_s and the clearest proof of circular failure are also provided (see the yellow line). P1 coordinates are
 192 (0, 0) and P2 (38.85, 14.6) in metres.

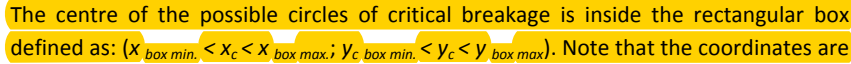

193 The code works as follows: the initial circular failure curve is plotted using the FPLOTT tool, as
 194 shown in Figure 2 (yellow line). In this example, the centre coordinates are equal to $x_c = 7 \text{ m}$; $y_c =$
 195 14 m and the lower cut with the talus coordinates (P1 point) equal to $x_t = 0 \text{ m}$, $y_t = 0 \text{ m}$. The
 196 FOS obtained was 1.6, which is, in principle, a stable talus. It must be taken into account that
 197 the mass susceptible to slipping must be divided into N pieces equal to the number of slices; in
 198 this example, the mass was divided into $N = 500$ slices.


199 The next step is to apply Spencer's method to the different breakage surfaces until the curve
 200 with the lowest FOS, F_s , is found, and that will be the critical surface susceptible to a circular
 201 slip. To determine the minimal FOS using this model, calculate the displacement of the lower
 202 cut point of the critical slip from slope, as well as the rotation centre position of the critical
 203 failure curve. In addition, a number of potential circular failures must be introduced by the





204 user. Then, the user introduces the following constraints into the programme: the initial or
205 lower point of the failure curve (P_1) in its intersection point with the slope, which may or may
206 not match the  in of the slope analysed. Another restriction is the centre of the failure
207 circle, (X_0, Y_0) ,  should initially cut the slope, i.e. the breaking curve must be within the
208 feasible sliding region. With this data, the programme automatically draws a first curve, in this
209 case the yellow line in Figure 2, and calculates the safety coefficient F_s for that initial curve.

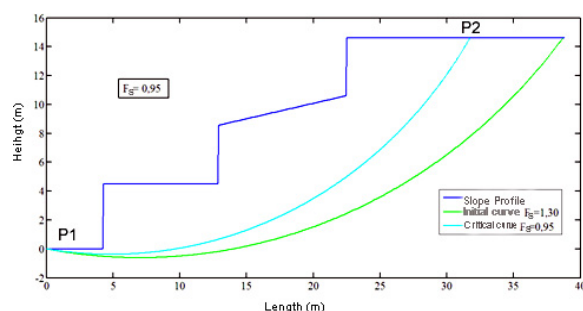
210 On the basis of this first curve (yellow line in Figure 2), the programme enforces new
211 restrictions:

- 212 • The curve passes through the origin of slope $P_1 = (0, 0)$.
- 213 •  The centre of the possible circles of critical breakage is inside the rectangular box
214 defined as: $(x_{box\ min.} < X_c < x_{box\ max.}; y_{c\ box\ min.} < y_c < y_{box\ max.})$. Note that the coordinates are
215 entered with the 2D expression (X, Y) . 


216 Both coordinates of the rotation centre position are free and can change for every circle. From
217 the initial failure curve, characterised by the point $x = (x_c, y_c)$, the MATLAB “fmincon” function
218 is used to obtain a new critical point (x_c^*, y_c^*) where the FOS from the breakage curve is the
219 minimum provided by fmincon. In this example, starting from the initial curve (yellow curve)
220 with point $x = (7, 14)$, the TS model provides a new point $x^* = (4.4910, 28.1091, 0)$  a new
221 FOS, $F_s = 1.45$. In this case, the new search has been carried out with the following restrictions
222 in the rectangular box, such as $2\ m < x_c < 8\ m$ and $16\ m < y_c < 40\ m$. These restrictions are
223 imposed in order to determine the critical circle. With all these restrictions, and because of the
224 first calculated curve (the yellow curve), the developed model calculates the critical curve
225 among the number of curves selected by the user (500 in this case), as well as the failure circle
226 centre, by applying the fmincon (MATLAB function). This defines the curve with a minimum
227 FOS (F_{min}) ~~as the value of F_s~~ (see green curve in Figure 2). When solving this problem, a critical
228 selection is the lower cut-off point of the slope. According to different authors, such as Verruijt
229 (2010) and Castedo et al. (2012), the selected point is the same as the P_1 point.


230

231 To complete the second phase in the TS model operation, the effect of rain infiltration must be
232 introduced by the coefficient of the pore pressure factor r_u . In this example, the infiltration
233 factor was introduced at the base of each slice to account for the infiltration and pore pressure
234 at the base of the break surface of the slope. If r_u increases, the cohesion of the soil mass of
235 the slope decreases, directly affecting the reduction of the slope’s FOS.  result is that a dry
236 talus has a $F_s = 1.45$, but if including the r_u parameter equal to 0.3,  the FOS decreases to a
237 value of $F_s = 0.95$, that means an FOS below the unity, so an unstable circular failure appears
238 (see Figure 3). Entering the infiltration factor, r_u , in Spencer’s method to introduce the
239 infiltration effects in slopes, the geotechnical cutting elements of the analysed soil are
240 reduced, also reducing the values of the F_s , both for the initial yellow curve and the optimum
241 green curve (Figure 2). Note that the initial curve in the run shown in Figure 3 is different from
242 the one in Figure 2, as it depends on the data introduced.



243

244 **Figure 3.** Outcome of the TS model after the introduction of the infiltration factor, producing an unstable
245 circular failure ($F_s = 0.95$). 

246 After the outcome produced here, it can be stated that the rainwater infiltration factor is
247 necessary to predict instabilities in slopes. If this infiltration factor is small enough, taking into
248 account the safety coefficients, the design may still be adequate, but there was a lack of
249 critical information for calculating this parameter. 

250 4. Example of this application in the municipality of La Viñuela, Málaga, Spain

251 In 2010, La Viñuela, Málaga, Spain experienced ~~experienced~~ torrential rainfall. The main
252 consequence was a devastating landslide with serious personal and material losses, as shown
253 in Figure 4. The coordinates where this event occurred were in degrees (36.88371409801, -
254 4.204982221126).





255

256 **Figure 4.** A) Spanish map with the location of La Viñuela (Google Maps). B) Real images taken by the
257 authors at La Viñuela in 2010.

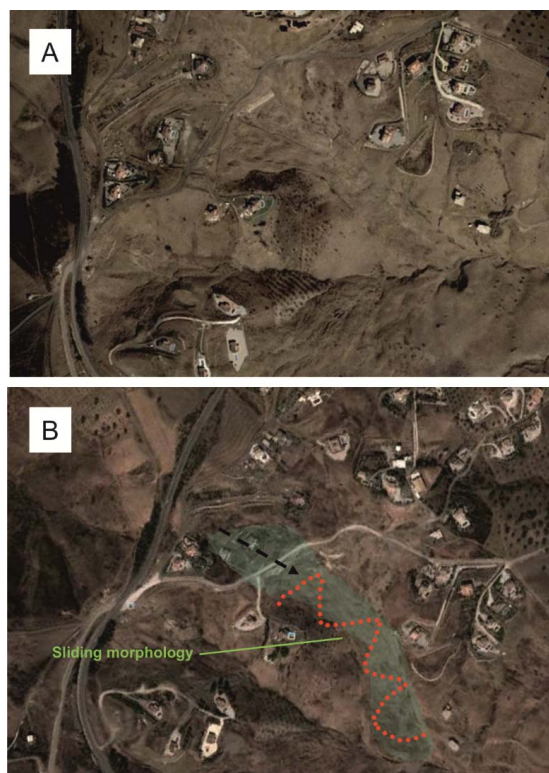


258 4.1 Geological and hydrological environment

259 The study area is located in the county of La Viñuela, specifically in the Hunderero village,
260 which is located immediately north of the Pantano de La Viñuela and south of the Betic
261 mountain range (South Iberian Peninsula). 

262 According to the Cruden and Varnes' classification [1996], the slide ~~suffered in the Paraje del~~
263 ~~Hunderero de la Viñuela~~ corresponds to a rotational slide-like complex movement because it
264 was generated in two sequences at different speeds. This type of mechanism is characteristic
265 of homogeneous cohesive soils, as was the one analysed here. 

266 This event caused serious damage to different buildings. Regarding the damage caused, in the
267 initial stretch of the slope (its head), a house was dragged and destroyed and another was
268 seriously damaged. On the right bank of the mentioned house, another building was affected.
269 In total, this event left a balance of two buildings destroyed and one seriously compromised.
270 Although 15 people lived in these houses, there were no fatalities. About 20 houses were to be
271 constructed at the head of the slope; fortunately, the event happened before this
272 construction. Figure 5 shows an aerial picture from 2006 before the disaster as well as the
273 affected area and landslide in 2010.



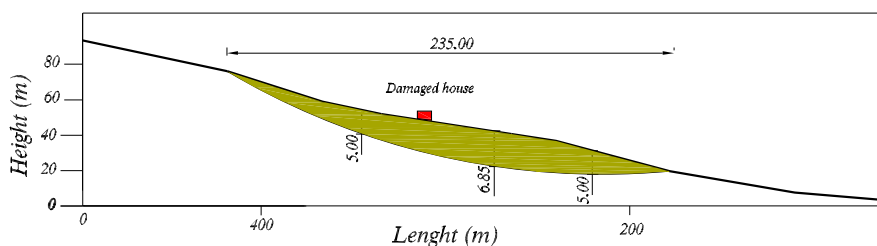
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275 **Figure 5.** A) An aerial photograph from before the event (2006). B) An aerial photograph taken after the
276 landslide (2010).



277 4.2 Event features and geometry

278 It is well known that mass movements, such as landslides, are highly complex morphodynamic
279 processes. We selected the Hundidero as our study area because it is prone to landslides. In
280 order to analyse this case study using our model, we first calculated the initial displaced
281 volume of the study area. According to the dimensions of the problem, the initial displaced
282 volume was calculated; equivalent to the volume of half an ellipsoid [Varnes, 1978; Beyer,
283 1987; Cruden and Varnes, 1996] equal to $Vol = 1/6 \pi$ (width x length x depth). In our particular
284 case, the width was equal to 70 m, the length equal to 235 m and the depth equal to 5 m,
285 making up a total volume of 4.364 m^3 (Figure 6). Taking an average of 33% elongation, as
286 proposed by Nicoletti and Sorriso-Valvo [1991] and Cruden and Varnes [1996], we determined
287 that the total material displaced in this landslide had an approximate volume of 5.804 m^3 . In
288 this mass displacement, it is also necessary to consider material added by erosion and dragged
289 from the initial mass displaced. In Figure 5, the straight line indicates the first rotational
290 movement, and the zigzag line shows the planar drag and glide after the first rotational
291 movement. The green region is the total area displaced or affected by mass movement. After
292 the first circular movement, the mass moved rapidly, associated with a continuous rise in
293 incremental pore pressure and the rapid reduction of shear strength, without allowing
294 pressure dissipation.



295

296 **Figure 6.** Characterisation and longitudinal section of the rotational sliding (Geolen S.A., 2010). The
297 location of the dragged house is noted in red. Analysed by the TS model.

298 The initial spit of land had an approximate size of 235 m in length by 70 m in width. Due to this
299 initial displacement, there was a drag and a huge posterior planar displacement of about
300 550 m length, affecting a zone with several parcels of land and buildings. These dimensions
301 were confirmed using aerial photography and field data. The soil is basically composed of clays
302 of variable thicknesses, of fine grain, with fluvial sediments and silty clay. The authors obtained
303 this data by conducting a field survey, as well as through the laboratory tests carried out by the
304 laboratory Geolen S.A. [Geolen, 1998] from a geological and geotechnical point of view,
305 according to a survey of those present as the laboratory extracted the materials, different
306 lithological levels can be distinguished, as shown in Table 1.

307

308

309



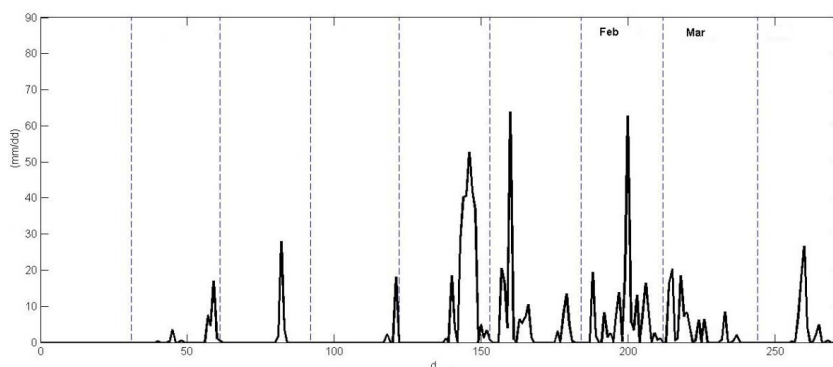
310 **Table 1.** Lithology of the area affected by the failure, according to the laboratory tests of
 311 Geolen S.A. No groundwater level was detected.

Level/layers	Lithology	Depth (m)
LEVEL 1	Silty sand with natural schistose pebbles	0.90
	Silty clay with marl intercalations	4.20
LEVEL 2	Colmenar unit, upper oligocene–lower miocene	
	Sandy clay	9.00
LEVEL 3	Colmenar unit, upper oligocene–lower miocene	(end of the probe)

312

313 The laboratory tests included a sieve analysis (following UNE 103 101) in three of the samples
 314 extracted from the field, at depths of 1.80–2.00 m, of which 70.3% was composed of clay and
 315 silt; according to this, the sample is classified as cohesive. The liquid limit and the plastic limit
 316 were determined on two of the samples (following UNE 103 103 and UNE 103 104,
 317 respectively), yielding liquid limit values of 57.5% and 64.2% and a plasticity index of 37%,
 318 respectively. According to the lab results, the material can be classified as high plasticity
 319 material with the potential of having a high water content. However, based on the field
 320 inspection and the analysis of the rainfall series in the La Viñuela region in 2010 (see Figure 7),
 321 it can be inferred that the main causes of the event were:

- 322 • the poor geomechanical parameters of the material that formed the affected hillside,
 323 and
- 324 • the hydrometeorological conditions in the days preceding and days after the event,
 325 according to the histogram.



326

327 **Figure 7.** Rainfall histogram at La Viñuela from August 2009 to April 2010.

328 Most of the landslides observed during these days occurred as a consequence of exceptionally
 329 intense rainfalls. The precipitation data was provided by the meteorological station of La
 330 Viñuela (Figure 7). It can be observed that large amounts of precipitation fell during the
 331 months of December, January, February and March of 2009 and 2010, with peaks of almost 60
 332 l/m² single day (January). In total, 890 l/m² fell to the end of April. This is a key point in



333 slope stability to consider when dealing with areas capable of having high infiltration rates. The
 334 rotational slide analysed had occurred between level 2 and level 3, when the water content
 335 reached that depth, as confirmed by the infiltration calculations in the terrain (see graphs in
 336 Figure 9, reaching depths of up to 5 m). Two direct shear tests (consolidated and drained) were
 337 conducted in unaltered samples extracted from the boreholes at 3.00–3.60 m and 4.00–4.60
 338 deep. The cut-off values of the soil are specified in Table 2. Those values were used in the
 339 developed software to obtain the safety coefficient and the theoretical failure curve.

340 **Table 2.** Summary chart of the characteristics of the soil analysed at the GEOLEN S.A. laboratory: ϕ the
 341 angle of internal friction, c the cohesion, γ_{sat} the saturated specific gravity and γ_a the apparent specific
 342 gravity.

Soil parameter	Result	Units
ϕ	17	°
c	0.27	N/mm ²
γ_{sat}	2000	N/mm ³
γ_a	1650	N/mm ³

343

344 The dynamic and continuous tests were carried out by the Geolen S.A. laboratory with an
 345 automatic penetrometer ROLATEC ML-60 A type. The data obtained was transcribed by the
 346 number of strokes to advance the 20 cms tip, which is called the “penetration number” (N_{20}).

347 This test is included in the ISO 22476-2:2005 standard as a dynamic probing super heavy, and
 348 consists of penetrating the ground with a conical tip of standard dimensions. The depth of the
 349 failed mass can be estimated, as well as the theoretical failure curve for an increase in the soil
 350 consistency (see data in Table 3).

351 The change in the geomechanical response of the soil takes place at a depth of 4–5 m,
 352 according to the results of N_{20} and US (samples without changes) taken along the analysed
 353 column. In this case, the sloped ground mass showed a characteristic striking relationship of a
 354 displaced terrain [Gonzalez de Vallejo et al. 2002]. This differs from the underlying or unmoved
 355 terrain, which indicated a more consistent striking relationship, that was taken within the area
 356 of the landslide behind the house drawn in accordance with the analysis of the hits N_{20} from
 357 Table 3.

358 **Table 3.** Summary chart of the soil analysed at the GEOLEN S.A. laboratory. Bold values show, according
 359 to the data of the field penetrometers, the depth mobilized by the rotational sliding.

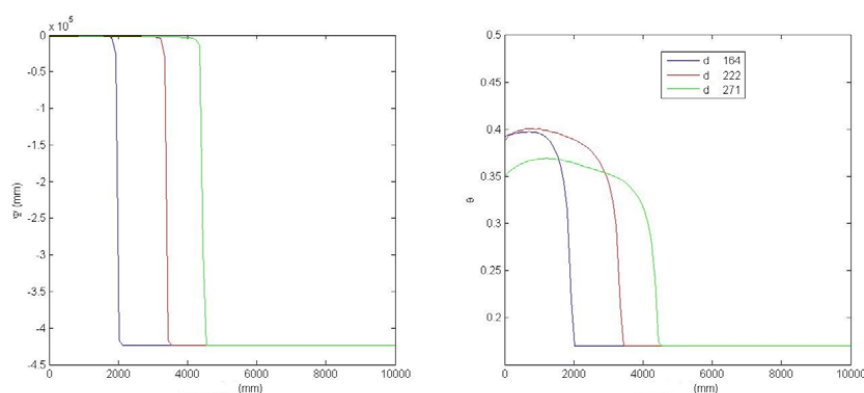
Depth (m)	Hits N_{20}	Consistency	Admissible stress (N/mm ²)
0.00 – 1.00	4	Soft	0.03
1.00 – 2.00	3	Soft	0.02
2.00 – 3.00	6	Slightly hard	0.04
3.00 – 4.00	7	Slightly hard	0.05
4.00 – 5.00	10	Slightly hard	0.07
5.00 – 6.00	19	Moderately hard	0.12
6.00 – 7.00	52	Hard	0.31
7.00 – 8.00	63	Hard	0.35
8.00 – 8.60	84	Hard	0.44

360



361 4.3 Input data

362 To analyse the topography of the critical section, we obtained the DEM data from ArcGIS 10
363 software programme [Esri, 2010], with a scale of 1:1000, through Spanish National Geography
364 (IGN) raster maps, with adequate accuracy. These data were interpolated to a 2 m grid using a
365 triangulated network interpolation methodology. Orthophotos proved very useful to locate
366 the landslide with accuracy and to validate the field survey. The model developed here applies
367 to failure in an initiation zone, in addition to predicting landslides, including those induced by
368 the infiltration of critical rains.



369

370 **Figure 8.** Left: hydraulic potential. Right: volumetric water content. Both have been plotted as a function
371 of the depth (mm) at different times (d).

372 To complete the input data, we plotted the hydraulic potential and the volumetric water
373 content, as a function of depth in the ground for different time steps, using a previously
374 developed infiltration model, as shown in Figure 8 [Herrada et al., 2014]. The figure shows the
375 evolution of how the wetting front advances can be observed. These reached almost 5 m deep
376 at the end of April 2010.

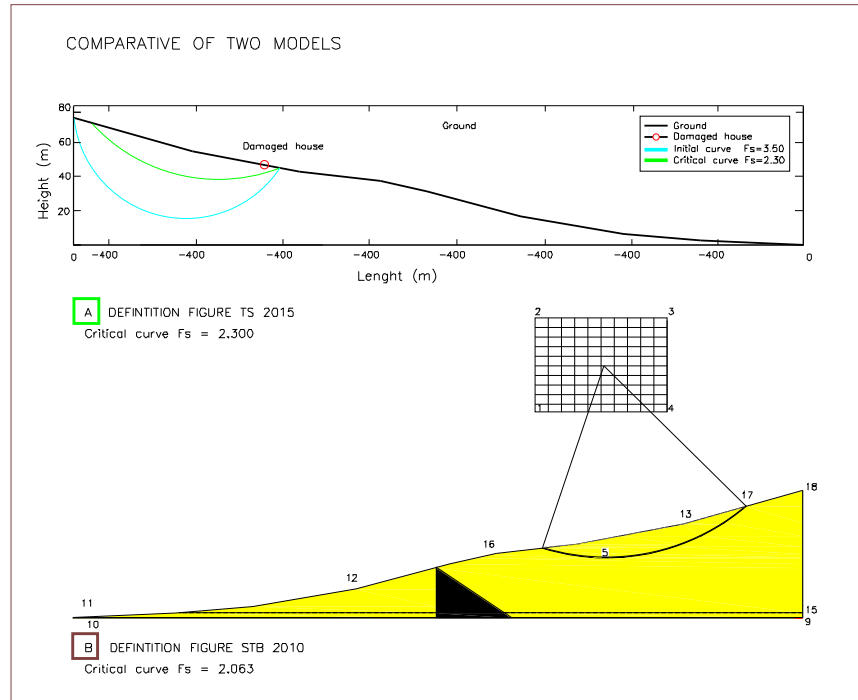
377

378 4.3 Analytical results

379 We applied the TS model using data from the ArcGIS 10 software programme and inventory
380 from the landslide. We did so to obtain the degree of stability of the sliding land based on the
381 angle of internal friction, the cohesion, the density and the angle of the slope we analysed.
382 Figure 9 shows the analytical results from the real talus. In addition we compared the results
383 given by the developed TS model and the results given by STB 2010 model, using free surfaces
384 in both cases. Due to the versatility of our programme, we were able to transfer the initial
385 curve from our previous calculations (shown in yellow) onto this one (shown in red). The worst
386 curve (shown in green) was calculated automatically from the initial curve, resulting in $F_s =$
387 2.300, in the dry state.

388

389



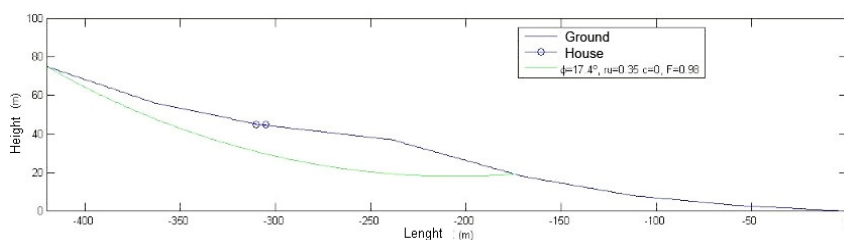
390

391 **Figure 9.** Top: TS model with a critical failure of $F_s = 2.300$. Bottom: results from the STB 2010 model with
 392 an FOS of 2.063.

393 As can be noted, the failure curves are similar, and the safety coefficients F_s only differ by
 394 0.237. In both cases, the results indicated are conservative estimates, resulting in a stable talus
 395 that was not realistic, as was the case in La Viñuela. In order to get the most unfavourable
 396 curve, which would match the analysis of the actual event, the pore coefficient must be
 397 introduced. At the first runs of the model, the r_u was equal to zero (dry soil – Figure 9), but if
 398 this value is changed to $r_u = 0.35$, results are quite different (Figure 10). The resulting
 399 failure was near the surface and the top cut with the talus found relatively near the houses.
 400 Taking into account the infiltration of rainwater, the slope analysed in the TS model showed a
 401 value of $F_s = 0.98$, in other words, that it was unstable.

402

403 This calculation and the theoretical failure curve provided by our model was able to reproduce,
 404 in a realistic way, the landslide which occurred in La Viñuela. Our model found that the critical
 405 surface area that corresponded with the profile of the terrain was $12.927.45 \text{ m}^2$, which closely
 406 matches the real situation. In the STB 2010 programme, it was $7.825.35 \text{ m}^2$; therefore, our
 407 prediction was more precise.



408

409 **Figure 10.** A new calculation including the pore coefficient r_u showing the worst curve in green. The
410 circles show the house dragged by the landslide.

411 As mentioned earlier, the STB 2010 model does not allow stability calculations to apply to
412 rainfall infiltration on a hillside. Hence, it is not capable of predicting a hillside's instability in a
413 critical rainfall scenario, which was critical in the slope analysed. The STB 2010 model found
414 that the hillside studied had an FOS of $F_S = 2.063$; that means it was a very stable slope.
415 Consequently, our innovative TS model appears to be more efficient and accurate.

416 The Terrain Stability analysis performed using the developed model defines fairly well the slip-
417 breaking curve that intuitively appears to be susceptible to failure, especially when heavy rains
418 occur. As an example, the landslides which occurred in the La Viñuela area could only have
419 been predicted if the infiltration had been taken into account. Even then, it could not have
420 been done with other available software programmes, which were not able to consider it.

421 1. In comparison with other software programmes, like the STB 2010, the model
422 developed here can analyse cliffs from right to left and vice versa.

423 2. Other software programmes, like the STB 2010, use a modified version of Bishop's
424 method, a less precise methodology than Spencer's method. A modified version of
425 Bishop's method solves only the equilibrium in momentum while the Spencer method
426 also considers the equilibrium in forces.

427 3. Another improvement made by the TS code, in comparison with others, is that the use
428 of the Spencer's method allows us to analyse any type of slope and soil profile. In this
429 procedure, we calculated the worst breaking curve by modifying the calculation points.

430 4. In the TS model, from the first slip rotational circle obtained in MATLAB, many circles
431 were then calculated using the `fmincon` function, with some user restrictions.
432 However, other model, like the STB 2010, require the definition of a quadrangular
433 region (to look for the centres of rotational failures) and a point (namely 5, see Figure
434 9) to define the curve as where the failure must pass. Also, the number of circles that
435 the STB 2010 model can analyse for their minimum value is limited to 100.

436 5. This model can detect relevant earth movements derived from rainfall infiltration,
437 both translational and rotational types [Stead et al., 2006], such as those that usually
438 occur in regions like India, the United States, South America and the United Kingdom,
439 among other places. The programmes that do not contemplate this option will
440 overestimate the FOS, potentially with great errors.



441 6. Our model programme has another advantage: it also offers the opportunity to
442 incorporate, in the same code, the stability analysis and the effect of the infiltration
443 factor in the rainfall regime. This is a step forward from open access programs, such as
444 STB 2010, and also alternative payment software, such as Slide.

445 5. Conclusion

446 The terrain stability (TS) analysis defines fairly well areas that intuitively appear to be
447 susceptible to landslides and defines rigorously the failure curve and the slip factor of safety
448 (FOS). We developed this model due to the necessity of a useful tool to predict landslides,
449 especially when heavy rains occur.

450 The TS model we developed uses the Spencer's method, which is more precise than the
451 modified Bishop method, so it differs in the results it provides for the FOS F_s . It also takes into
452 account the factor of water infiltration due to critical rains, which other software programmes
453 do not consider. A failure surface can be determined by constraints using the MATLAB function
454 `fmincon`. The data needed to run the model include soil and climate properties that may vary
455 in space and time. The exit indices of the analysis (F_s) should be interpreted in terms of relative
456 risk. The methods implemented in the TS model are based on data structures, which are based
457 on the data entry of the elevation model (DEM), key to obtaining the profile to be studied.

458 In the case study analysed, the hillside was initially stable and was so determined by the
459 analysis performed with the STB 2010 model. However, the hillside became unstable due to
460 the heavy rains of that hydrological period, which called for the application of the pore
461 pressure coefficient r_u . For analysing cases of heavy rain, this model is probably the most
462 powerful tool for determining slope stability. In addition, thanks to the great versatility of this
463 model, it is applicable to any analysis in other parts of the world, based on the methods of
464 limit equilibrium (Spencer 1967). The TS model can also be used in combination with GIS
465 software, SINMAP and aerial photographic analysis, as well as mapping techniques or even as
466 part of other models like the coastal recession models [Castedo et al., 2012].

467 6. References

468 Aenor Institut (2008). Geotechnical investigation and testing - Field testing - Part 2:
469 Dynamic probing (ISO 22476-2:2005). Madrid. Spain.

470 Bishop, A. W., Morgenstern, N. R. (1960). Stability coefficients for earth slope.
471 Geotechnique 10, 129-150.

472 Bishop, A. W. (1955). The Use of the slip circle in the Stability Analysis of Slope.
473 Geotechnique 5: 1:7-16.

474 Beyer, W.H. (1987). Handbook of Mathematical Sciences. 6th ed., Boca Raton/Florida

475 Burbano G., del Cañizo L., Gutiérrez J. M., Fort L., Llorens M., Martínez M., Paramio J.
476 R., Simic D. (2009). Guía de cimentaciones en obras de carretera. Ministerio Fomento: Madrid.

477 Castedo, R., Murphy, W., Lawrence, J., & Paredes, C. (2012). A new process-response
478 coastal recession model of soft rock cliffs. Geomorphology, 177, 128-143.



- 479 Cheng, Y. M., Lansivaara, T., Wei, W. B. (2007). Two-dimensional slope stability analysis
480 by limit equilibrium and strength reduction methods. *Computers and Geotechnics* 34, Nº 3,
481 137-150.
- 482 Chugh, A. K., Smart, J. D. (1981). Suggestions for slope stability calculations. *Computers*
483 *& Structures* 14, 1–2, 43-50.
- 484 Cruden, D. M. and Varnes, D. J. (1996). Landslides types and processes. In: *Landslides*
485 *investigation and mitigations*. Transportation Research Board Special report 24. Turner y
486 Shuster (eds.), 36-75.
- 487 Duncan, J. M., Wright S. G. (2005). *Soil Strength and Slope Stability*. John
488 Wiley, Hoboken, N. J.
- 489 Duncan, J. M. 1996. *Landslide Types and Processes*. *Landslides investigations and*
490 *mitigation*. Ed. Turner A. Special Report, TRB.
- 491 Environmental Systems Research. (1969-2017). Institute Geographic information
492 system (platform and resources ArcGIS). California, EEUU. Recovered in
493 <http://www.esri.es/arcgis/productos/>.
- 494 Fellenius W. (1936). Calculation of the stability o Heat dams. Washington, D.C.: In
495 *Proceeding of the 2nd Internacional Congress on Large Dams*, - Vols. 4, 445. -4:445.
- 496 Geolen Engineering. (2010). *Geotechnical study in the Viñuela*. Sevilla, Spain.
497 Recovered in <http://www.geolen.es>
- 498 Glennon, R., Harlow, M., Minami, M., Booth, B. (2008). *ArcGis 9. ArcMap Tutorial*. Esri
499 North Carolina. U.S.A.
- 500 González de Vallejo, L., Ferrer, M., Ortuno, L., y Oteo, C. (2002). *Ingeniería Geológica*.
501 Madrid: Prentice Hall.
- 502 Griffiths, D. V., Marquez, R. M. (2007). Three-dimensional slope stability analysis by
503 elasto-plastic finite elements. *Géotechnique* 57, Nº. 6, 537–546.
- 504 Griffiths, D. V. (2015). *Slope stability analysis by finite elements. A guide to the use of*
505 *Program slope64*. Geomechanics Research Center Colorado School of Mines.
506 (http://inside.mines.edu/~vgriffit/slope64/slope64_user_manual.pdf).
- 507 Herrada, M. A., Gutiérrez-Martín, A., Montanero, J. M. (2014). Modeling infiltration
508 rates in a saturated/unsaturated soil under the free draining condition. *Journal of Hydrology*,
509 515, 10–15.
- 510 Jiménez Salas J.A., Justo Alpañes, J.L. (1981). *Geotecnia y Cimientos II*. Ed. Rueda.
511 Madrid.
- 512 Janbu, N. (1954). Stability analysis of slopes with dimensionless parameters. In:
513 *Harvard University soil mechanics series*, vol 46.



- 514 Jia, G. J., Tian, Y., Liu, Y., Zhang, Y. (2008). A static and dynamic factors-coupled
515 forecasting model of regional rainfall-induced landslides: A case study of Shenzhen. *Science*
516 *China: Technological Sciences* 51. Suppl. 2, 164-175.
- 517 Liu, S. Y., Shao, L. T., Li, H. J. (2015). Slope stability analysis using the limit equilibrium
518 method and two finite element methods. *Computers and Geotechnics* 63, 291-298.
- 519 Michel. G. P., Kobiyama, M., Fabris, R. (2014). Comparative analysis of SHALSTAB and
520 SINMAP for landslide susceptibility mapping in the Cunha River basin, southern Brazil. *Journal*
521 *of Soils and Sediments* 7. 1266–1277.
- 522 Michel. G. P., Kobiyama, M., Fabris, R. (2015). Critical rainfall to trigger landslides in
523 Cunha River basin, southern Brazil. *Natural Hazards* 75, 2369-2384.
- 524 Morgenstern, N. R., Price, V. E. (1965). The analysis of the stability of general slip
525 surfaces. *Geotechnique* 15, 79-93.
- 526 National Geographic Institute. (1870-2017). Geological and raster maps. Madrid, Spain.
527 Recovered in <http://www.ign.es/web/ign/>.
- 528 Nicoletti, P. G., Sorriso-Valvo, M. (1991). Geomorphic controls of the shape and
529 mobility of rock avalanches. *GSA Bulletin* 103 (10): 1365-1373.
- 530 Ramos Vásquez, A. A. (2017). Análisis de estabilidad de taludes en rocas. Simulación
531 con LS-DYNA y comparación con Slide. *Trabajo Fin de Máster*, Máster Universitario en
532 Ingeniería Geológica, ETSI Minas y Energía, Universidad Politécnica de Madrid.
- 533 Spencer, E. (1967). A method of analysis of analysis of the stability of embankments
534 assuming parallel interslice forces. *Géotechnique* 17, 11-26.
- 535 Stead, D., Eberhardt, E., Coggan, J. S. (2006). Developments in the characterization of
536 complex rock slope deformation and failure using numerical modelling techniques. *Engineering*
537 *Geology* 83. 1-3:217-235.
- 538 Tschuchnigg, F., Schweiger, H. F., Sloan, S.W. (2015) Slope stability analysis by means
539 of finite element limit analysis and finite element strength reduction techniques. Part II: Back
540 analyses of a case history. *Computers and Geotechnics* 70, 178-189.
- 541 Van Westen, C. J., Terlien, M. J. T. (1996) An approach towards deterministic landslide
542 hazard analysis in gis. A case study from manizales (Colombia). *Earth Surface Processes and*
543 *Landforms* 21. 9:853-868.
- 544 Varnes, D.J. (1978). Slope movement types and processes. In R.L. Schuster y R. J. Krizek
545 (Eds.) *Landslides: analysis and control*. Transportation Research Board. Special report 176: 11-
546 33.
- 547 Verruijt, A. (2010). STB—slope stability analysis program. Delft University.
548 (<http://geo.verruijt.net>).



- 549 Wilkinson P.L., Anderson M.G., Lloyd D.M., Renaud P-N. (2002). Landslide hazard and
550 bioengineering: towards providing improved decision support through integrated numerical
551 model development. *Environment Modelling and software* 17:4, 333-344.
552
553 Yong, R. N., Alonso, E., Tabba, M. M., Fransham, P. B. (1977). Application of Risk
554 Analysis to the Prediction of Slope Stability. *Canadian Geotechnical Journal* 14, 540-553.