A field experiment for calibrating landslide time-of-failure prediction functions

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\section*{A B S T R A C T}

Over the last decades, time-of-failure semi-empirical prediction functions have been developed and applied to different landslides with mixed results. In this study, a field experiment was carried out to calibrate these functions with the simultaneous consideration of small-size landslides and landslides that occur on slopes modified by human activities. Four years of continuous monitoring using an integrated platform consisting of traditional sensors (i.e., inclinometers, piezometers, load cells, topographic measurement) and innovative remote-sensing equipment (i.e., Terrestrial SAR Interferometer) resulted in the collection of a notably large amount of data. Several landslides affecting different slopes (i.e., cut slopes, cut slopes covered by spritz-beton and slopes stabilised by anchored bulkheads) were observed as part of the experiment, thus facilitating the inference of detailed information for the pre-failure behaviour. Nine landslides were back-analysed, thus allowing for calibration of the failure prediction functions for different types of slopes. From these observations, it was found that events occurring on slopes modified by human interventions could be effectively predicted using the Voight function if suitable parameters are used. As a general remark, the landslides that originate from cut slopes in natural terrain behave similar to large landslides reported in the literature (similar values of $\alpha$ and $\beta$) while landslides that originate from cut slopes covered by spritz-beton and slopes stabilised by anchored bulkheads show $\alpha$ values that are significantly lower and $\beta$ values that are significantly higher than those of landslides on natural terrains.

\section*{1. Introduction}

Predicting the short- and long-term evolution of a slope presents a fundamental challenge for studies of the interaction of landslides with large infrastructures. Landslide behaviour has been extensively investigated in the scientific community since the beginning of the 20th century [1,2]. However, only from the 1960s have some authors attempted to define semi-empirical approaches for predicting the time of failure ($t_f$) using instrumental monitoring of slope displacement [3–6]. The first attempts to predict the $t_f$ of unstable slopes based on displacement time evolution date to the early 1960s [3,7,8]. By analysing the rupture of eighty samples in tri-axial compression lab tests, Saito observed that displacement was the most useful parameter for prediction of $t_f$. Hence, Saito and Uezawa [7] developed a method to obtain the time to failure using the slope displacement data. The Saito method was based on the “slope creep” theory [2,9] and provided a good fit to the steady strain rate phase (i.e., secondary creep). Next, relevant improvements to the Saito's slope creep theory were accomplished [4,10–12]. Based on large-scale experiments, Fukuzono showed that the logarithm of the velocity of the surface displacement is proportional to the logarithm of the acceleration. In other words, under invariant loading conditions, the pre-failure behaviour can be described by a power law equation in the following form:

$$\frac{d^2x}{dt^2} = A \left( \frac{dx}{dt} \right)^\alpha \quad \text{or} \quad \tilde{\Omega} = A(\tilde{\Omega})^\alpha$$

(1)

where $x$ is the downward surface displacement along the slope, $t$ is the time, $\tilde{\Omega}$ is the velocity, $\Omega$ is the acceleration, and $A$ and $\alpha$ are constants. According to the results of Fukuzono [4] and studies by Varnes [13] and Yoshida and Yachi [14], the $\alpha$ value for natural landslides ranges from 1.5 to 2.2. Specifically, $\alpha$ is larger than 1 for approximately 80% of the available landslide dataset, and is equal to 2 for approximately 50% of those measurements.

Hence, the following equation has been suggested for the prediction of $t_f$:

$$v^{-1} = [\alpha(\alpha - 1)(t_f - t)]^{-1/(\alpha - 1)}$$

(2)
where $v$ is the surface displacement velocity. The inverse velocity vs. time curve is linear for $\alpha = 2$, convex for $\alpha > 2$ and concave for $1 > \alpha < 2$. Hence, in the case of $\alpha = 2$, the failure time can be simply computed using the following equation:

$$t_f = \frac{(t_f/v_1) - (t_o/v_2)}{(1/v_1) - (1/v_2)}$$

(3)

Also, $t_f$ corresponds to the intercept of the straight line of the inverse velocity vs. time plot with the x-axis (time axis, Fig. 1).

The efficacy of this method for predicting landslides ($t_f$) has been demonstrated by several authors [12,15–23]. However, a generalized expression and additional calibration are urgently required [24], particularly for man-made slopes and natural slopes affected by human activities.

Over the last four years, the authors of this paper have had the opportunity to continuously monitor the displacement of a large slope affected by human activities (i.e., a tunnel excavation) using Terrestrial SAR Interferometry [25–26]. This long-term continuous monitoring allowed collecting a large set of interesting data on the pre-failure displacement of different types of landslides. This “field” laboratory provides the foundation for the study presented in this work, which aims to assess the most common $\alpha$ and $A$ parameters of the Fukuzono and Voight equations (Eq. (1)).

2. The experimental setting and collected data

The experimental setting consists of a natural slope affected by anthropic activities associated with the excavation of a tunnel. Following a destructive landslide (with a volume of approximately $10^4\ m^3$) on March 2007, the slope was instrumented for continuous and real-time monitoring to control the safety conditions during the subsequent stabilisation activities and tunnelling excavations. The instrumental setting consisted of the following equipment (Figs. 2 and 3): (i) An integrated remote platform, including a Terrestrial SAR Interferometer (TInSAR) model IBIS-L (by IDS S.p.A.); a weather station and an automatic photo camera installed on the frontal slope (with respect to the landslide [26]) at a distance ranging from 700 to 900 m (active from November 2007); (ii) three inclinometers with a variable length between 45 and 75 m, one full-screen piezometer installed at a depth of 16 m and one ‘Casagrande’ piezometer installed at a depth of 48 m (active from June 2007); (iii) monitoring via a topographic station of prisms installed on the bulkheads (active from September 2008); (iv) three load cells for monitoring the anchors installed on the bulkheads (active from September 2008) and (v) convergence monitoring of the tunnel (active from November 2009).

The continuous monitoring of the slope from the middle of 2007 to the end of 2011 allowed collecting a large set of displacement data for the overall slope following various work phases conducted on the slope. Specifically, the following data are available: approximately 20,000 photos, 250,000 measurements of weather data, 400,000 SAR images collected from the same position, 160 inclinometric measurement 160 piezometric measurements, 160 topographic measurements, and, finally, 85 load cell measurements.

2.1. Engineering geological features of the slope

During the experimental period, the slope was investigated in detail using (1) field geomorphological, geological, geomechanical and seismic surveys and (2) boreholes and laboratory tests of samples, thus enabling a detailed engineering geological model to be derived [21,26]. The steep relief of the slope consists of jointed and weathered metamorphic rocks overlaid by Pliocene and Pleistocene sandy marine deposits. Sandy colluvial deposits a few meters thick constitute an irregular blanket along the slope. Furthermore, geological and geomorphological features show evidence of an old and deep roto-translational slide whose total volume was on the order of $1 \times 10^4\ m^3$ that affected the jointed gneiss and the overlying Pliocene and Pleistocene sands. The main sliding surface (up to 50 m deep) and many secondary surfaces were reconstructed using geomorphological surveys, stratigraphic logs, interpretations of the surface displacements (derived from TInSAR and topographical data) and deep displacements (derived from...
inclinometer data) (Fig. 3). Minor shallow translational movements, involving colluvial deposits and bedrock, with volumes ranging between $1 \times 10^3$ and $1 \times 10^4$ m$^3$ were observed in the middle-lower part of the landslide mass. Among these minor landslides, we include the event that destroyed the already-built excavation structures in 2007 (Fig. 2).

### 2.2. Work activities on the slope

During the experimental period, several works were carried out on the slope, first for stabilisation and subsequently for tunnelling purposes. More specifically, the following activities were conducted for stabilisation purposes:

- Realising gabions (May–June 2008, March 2009 and July 2009),
- Realising anchored bulkheads consisting of 0.1-m diameter vertical piles 10 to 22 m in length anchored by 35-m long cables inclined at 35° from the horizontal plane and cemented in the last 12 m (from August 2008 to January 2009),
- Cutting slopes with a maximum inclination of 85° (March 2007 and August–September 2009), and
- Spritz-beton coverage of certain cut slopes (August–October 2009).

The tunnel excavation began on November 2009, after the slope stabilisation was concluded. Three main excavation phases were carried out using different excavation methods in an attempt to mitigate the destabilisation effects on the slope. The excavation was definitively halted in February 2010 after the completion of a 28-m long tunnel due to the triggering of slope displacements that were higher than expected.

### 3. Pre-failure displacement patterns of the slope

Different patterns of displacement in both the natural slopes and the slopes modified by human activities were investigated during the overall experimental period, thus allowing the identification of their typical behaviour [21]. Focusing on the pre-failure behaviour, the following three slope categories were considered in this work (Fig. 2): (i) Cut slopes, (ii) Cut slopes covered by spritz-beton, (iii) Slopes stabilised by anchored bulkheads.

#### 3.1. Cut slopes

Five landslides occurred between January 2009 and March 2010, and four of them that involved cut slopes have been analysed in detail. These landslides were characterised by a translational and roto-translational movement involving the weathered and jointed gneiss widely found on the slope. Data collected by TInSAR identified the landslide location in detail and allowed measuring the pre-failure behaviour using a continuous time series of displacements (with a 5-min sampling rate). The four landslides, characterised by a volume ranging from $10^1$ to $3 \times 10^3$ m$^3$ and a maximum thickness of 3 m, were investigated in detail using the displacement time series (dotted lines in Fig. 4). The evolution time ranged from approximately one day to two weeks, and the total cumulative displacement before failure ranged from 10 cm to 1 m. The maximum velocity reached by
the fastest landslide was 65 mm/h, with a peak of acceleration of 41 mm/h². All of these landslides were characterised by a peak acceleration that occurred some hours before the collapse (ranging from 0.75 to 3.5 h).

3.2. Cut slopes covered by spritz beton

Several landslides occurred on cut slopes covered by spritz beton between December 2009 and January 2010. These landslides were characterised by translational and roto-translational movements involving the weathered and jointed gneiss widely outcropping on the slope. In this case, data collected by TInSAR identified the landslide location in detail and provided measurements of the pre-failure behaviour via a continuous time series of displacements (dotted lines Fig. 5). These landslides were generally larger in size (from 10⁷ to 5 × 10³ m³) and more uniform in thickness (from 1.5 to 2 m) with respect to those not covered by spritz beton. Furthermore, these landslides displayed a shorter time between the displacement onset and the final collapse (from 10 h to two days) and a displacement ranging from a few cm to 10 cm. The maximum velocity reached by the fastest landslide was 37.5 mm/h, with a peak of acceleration of 82 mm/h². All of these landslides produced the peak acceleration some hours before the collapse (ranging from 0.5 to 2 h). In selected landslides, the peak
acceleration was followed by a deceleration phase that led to a decreasing in velocity just before the collapse.

3.3. Slopes stabilised by anchored bulkheads

The displacement data for the anchored bulkheads were collected via TInSAR monitoring, topographic monitoring and inclinometric measurements. Hence, redundant time series of displacements were available and allowed for detailed descriptions of the evolution of displacement during each tunnel excavation phase. Specifically, the time series of displacement derived from the topographic systems are available for the first acceleration phase, while both topographic and TInSAR data are available for the last two acceleration phases. Thus, the time series of displacement, velocity and acceleration were derived using these data [20]. The following section provides a brief description of the bulkhead displacement from realisation to the final stoppage of the tunnel excavations (Figs. 6 and 7). The anchored bulkheads began to deform immediately after construction, with a fairly constant velocity of 0.0025 mm/h over approximately one year, thus reaching a total displacement of 22 mm before the tunnelling excavation began. After the launch of the tunnelling activities, the velocity of the bulkheads suddenly increased, reaching maximum values of 0.75 mm/h for the acceleration and deceleration peaks on the order of 0.02 mm/h² (Fig. 7) and accumulating a displacement of approximately 80 mm in 3 months. During the second excavation phase, the first bulkhead was affected by a 4-mm displacement after approximately two days, while the total amount was on the order of 10 mm after approximately three days and a maximum velocity of 0.6 mm/h during the third phase. After the end of excavation, the displacement continued to develop with a variable velocity ranging from 0.003 mm/h to spikes of up to 0.2 mm/h and accumulating approximately 40 mm in fifteen months. Because the total displacements reached 140 mm in 32 months, significantly exceeding the threshold of 40 mm predefined by the designers, the tunnel excavation was definitively stopped at the end of January 2011, thus avoiding collapse. In the last two tunnelling phases, the interferometric monitoring allowed us to easily recognise the typical creep behaviour (Fig. 7).

4. Calibration of the failure prediction methods and achieved results

Aiming to calibrate models suitable for failure prediction of the different deformation events that affect the slope, a parametric analysis was performed based on the large dataset available from past events. Each event was back-analysed using the most common prediction models [10,27,28], thus deriving the A and α coefficient values that lead to the best fit between the real event and the prediction functions (see black dashed lines of Figs. 4 and 5). The best-fit computation was performed using a suitable minimisation algorithm built into the Matlab software. The algorithm, referred to as ‘fmincon’, finds the minimum of a constrained nonlinear multivariable function using a Hessian matrix or the second derivatives of the Lagrangian. In other words, the most suitable A and α values are identified by minimising the error between the pre-failure displacement data derived from Eq. (4) and the real monitored displacement over time (where \( \Omega_2 \) is the velocity at \( t_f \)). It is worth noting that Eq. (4) is valid only for \( \alpha > 1 \) and \( \alpha \neq 2 \):

\[
\Omega = \frac{1}{A\left(\alpha-2\right)}\left\{ A(1-\alpha)t_f + \Omega^1 - \frac{\alpha}{1-\alpha} + \frac{\alpha}{1-\alpha} \left(1-\alpha\right) - \frac{\alpha}{1-\alpha} \left(t_f - t_0\right) \right\} + \Omega^1 - \frac{\alpha}{1-\alpha} \left(1-\alpha\right).
\]

Eq. (4) has been discussed extensively by Voight [10], and the following has been derived for this study for \( \alpha \) values \( < 1 \) starting from Eq. (3) of [10], and assuming \( t_0 = 0 \) and \( \Omega_0 = 0 \):

\[
\Omega = \Omega_f - \frac{1}{A\left(\alpha-2\right)}\left\{ A(1-\alpha)t_f^2 - \frac{\alpha}{(1-\alpha)} + \frac{\alpha}{(1-\alpha)} \left(1-\alpha\right) - A(1-\alpha)t_f^2 \right\}.
\]

where \( \Omega_f \) is the cumulative displacement before failure.
The abovementioned functions were used for the calibration of landslides on cut slopes covered by spritz beton. In contrast, a dedicated method (which is extensively described in the following section) was developed for the calibration of anchored bulkheads because no information with respect to this type of failure is available.

4.1. Cut slopes

Landslides on cut slopes were analysed using Eq. (4), thus deriving the \( A \) and \( \alpha \) values presented in Table 1. The \( \alpha \) values range from 1.2 to 1.8, which are in agreement with the standard values from the literature for natural slopes \[4,13,14,27,28\]. The \( A \) value ranges from 0.09 to 2.7 and is significantly higher than the values commonly reported in the literature for natural slopes \[28\]. The mean values of \( \alpha \) and \( A \) for all datasets are 1.47 and 0.83, respectively. Value of 0.98 was output from computation of the mean all datasets are 1.47 and 0.83, respectively. Value of 0.98 was found from computation of the literature for landslides occurring on natural slopes, which are in agreement with the standard obtained when using both Eqs. (4) and (5), because no information with respect to this type of failure is available.

4.2. Cut slopes covered by spritz beton

Landslides on cut slopes covered by spritz beton were analysed using both Eqs. (4) and (5) because \( \alpha \) produced values quite close to one. Based on the best \( R^2 \) value achieved, the best \( \alpha \) and \( A \) values were chosen and reported in Table 2. For this landslide category, the \( \alpha \) values range from 0.6 to 1.1 and are significantly lower than those for landslides occurring on natural slopes, as reported in the literature \[4,13,14,27,28\]. The \( A \) values range from 2.6 to 66, i.e., one to two orders higher than values commonly reported in the literature for large landslides \[28\]. The mean values of \( \alpha \) and \( A \) computed for all landslides were 0.86 and 38.5, respectively. A value of 0.97 was found from computation of the mean \( R^2 \) between the real time series and the computed series, thus demonstrating the good quality of the overall analysis.

### Table 1

<table>
<thead>
<tr>
<th>Landslide</th>
<th>Number of data</th>
<th>Time of failure (tf)</th>
<th>Velocity computed (recorded)</th>
<th>Failure displacement computed (recorded)</th>
<th>( \alpha )</th>
<th>( A )</th>
<th>( R^2 )</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>2189</td>
<td>7.60 (7.60)</td>
<td>17.07 (17.07)</td>
<td>142.38 (144.61)</td>
<td>1.4166</td>
<td>0.3944</td>
<td>0.9960</td>
</tr>
<tr>
<td>7</td>
<td>829</td>
<td>2.87 (2.87)</td>
<td>29.96 (29.96)</td>
<td>108.37 (105.13)</td>
<td>1.8059</td>
<td>0.1062</td>
<td>0.9507</td>
</tr>
<tr>
<td>9</td>
<td>4741</td>
<td>16.46 (16.45)</td>
<td>44.37 (44.37)</td>
<td>960.42 (906.33)</td>
<td>1.4209</td>
<td>0.0994</td>
<td>0.9918</td>
</tr>
<tr>
<td>10</td>
<td>332</td>
<td>1.15 (1.15)</td>
<td>64.06 (64.06)</td>
<td>150.92 (147.88)</td>
<td>1.2323</td>
<td>2.7188</td>
<td>0.9928</td>
</tr>
<tr>
<td>Average</td>
<td>2023</td>
<td>7.02 (7.02)</td>
<td>38.86 (38.87)</td>
<td>340.52 (325.99)</td>
<td>1.4689</td>
<td>0.8287</td>
<td>0.9828</td>
</tr>
</tbody>
</table>

### Table 2

<table>
<thead>
<tr>
<th>Landslide</th>
<th>Data</th>
<th>Time of failure (tf)</th>
<th>Velocity computed (recorded)</th>
<th>Failure displacement computed (recorded)</th>
<th>( \alpha )</th>
<th>( A )</th>
<th>( R^2 )</th>
</tr>
</thead>
<tbody>
<tr>
<td>3</td>
<td>109</td>
<td>0.37 (0.37)</td>
<td>10.45 (9.81)</td>
<td>20.29 (20.30)</td>
<td>0.7321</td>
<td>43.7721</td>
<td>0.9876</td>
</tr>
<tr>
<td>4</td>
<td>95</td>
<td>0.32 (0.32)</td>
<td>39.43 (37.32)</td>
<td>23.26 (23.26)</td>
<td>0.9204</td>
<td>66.6634</td>
<td>0.9549</td>
</tr>
<tr>
<td>5</td>
<td>248</td>
<td>0.87 (0.87)</td>
<td>11.29 (8.08)</td>
<td>21.84 (21.85)</td>
<td>0.9064</td>
<td>20.9233</td>
<td>0.9521</td>
</tr>
<tr>
<td>6</td>
<td>598</td>
<td>2.07 (2.07)</td>
<td>10.63 (10.63)</td>
<td>64.03 (63.96)</td>
<td>1.1491</td>
<td>2.6756</td>
<td>0.9920</td>
</tr>
<tr>
<td>8</td>
<td>151</td>
<td>0.52 (0.52)</td>
<td>27.53 (27.21)</td>
<td>99.23 (99.23)</td>
<td>0.6217</td>
<td>58.4589</td>
<td>0.9937</td>
</tr>
<tr>
<td>Average</td>
<td>561</td>
<td>0.83 (0.83)</td>
<td>19.87 (18.61)</td>
<td>45.74 (45.72)</td>
<td>0.8659</td>
<td>38.4987</td>
<td>0.9761</td>
</tr>
</tbody>
</table>

4.3. Slopes stabilised by anchored bulkheads

A different approach was used for the slopes stabilised by anchored bulkheads because they did not fail during the analysis; a complete time series of displacements and the time of failure were thus not available for calibration purposes. Specifically, the following values were not available: \( t_f, \Omega_1, \) and \( \Omega_2 \). With the intention of deriving the \( t_f \) value by assuming a constant excavation rate, we first applied the Fukuzono \[4\] method for \( \alpha=2 \), i.e., a linear relationship between the time and the inverse of velocity. As shown in Fig. 8., the predicted \( t_f \) was 2.8 days after the beginning of the analysis. However, failure did not occur at the predicted time regardless of the constant excavation rate. The predicted \( t_f \) was significantly anticipated using the linear Fukuzono model, thus demonstrating that the velocity vs. time line was clearly nonlinear, and therefore \( \alpha \) is significantly less than two (Fig. 8).

To estimate reliable \( \alpha \) and \( A \) values for the investigated slope, we computed the logarithm of the velocity vs. the logarithm of the acceleration, as previously suggested \[4,18\], thus achieving \( \alpha \) value close to 1.1 (Fig. 9). Next, a new analysis based on the Fukuzono \[4\] approach fixing \( \alpha \) close to 1.1 was performed, thus achieving a new value for the \( t_f \) of 3.9 days (Fig. 8).
Then, the \( t_f \) and \( \alpha \) values were used to constrain the analysis of Eqs. (4) (for \( \alpha > 1 \) and \( \alpha \neq 2 \)), which was performed using a suitable minimisation algorithm to define the \( A \), \( \Omega_f \) and \( \Omega_t \) parameters (depending of the use of Eqs. (4) or (5)) and to confirm the other parameters (\( \alpha \) and \( t_0 \)). The values \( \alpha=1.01 \), \( t_f=3.95 \) days, \( A=1.43 \) and \( \Omega_t=2.67 \) mm/h were obtained, thus allowing us to compute the typical time-velocity curve using the following equation for \( \alpha > 1 \)

\[
\dot{\Omega} = \left[A(\alpha - 1)(t_f - t) + \Omega_t^\alpha \right]^{1/(\alpha - 1)}
\]

(6)

or the following equation for \( \alpha < 1 \), where \( t_0 \) is the initial time and the velocity at \( t_0 \) (Fig. 11):

\[
\dot{\Omega} = \left[A(1 - \alpha)(t - t_0) + \Omega_0^{1-\alpha} \right]^{1/(1-\alpha)}
\]

(7)

The same procedure was used for the second excavation phase and produced the results shown in Table 3 and Fig. 10. For the first excavation event, only topographical data were available, and the temporal resolution and displacement accuracy were insufficient to perform such analysis.

### 5. Discussion

To improve the calibration of the existing semi-empirical methods for predicting landslide time-of-failure and evaluating possible extensions to slopes modified by anthropic intervention, four years of continuous monitoring were carried out on a slope affected by tunnelling excavation and various stabilisation activities using an integrated monitoring platform. Data collected during the occurrence of certain landslides on the slope were analysed via back analysis to derive the \( A \) and \( \Omega \) parameters that control previously described equations \([4,10]\). The following represent the main original features of this work: (i) Several landslides have been observed and measured on the same slope; (ii) Pre-failure displacement data have been collected with a high sampling rate (up to 5 min) and high accuracy (up to 1 mm); (iii) Landslides characterised by the presence of stabilisation activities have been considered. To the best of the authors’ knowledge, the last point represents the main challenge of this work before prior of this work Fukuzono and Voight models were used only to predict landslides that affect natural slopes.

Structural stabilisations like coverage by spritz beton, walls and anchored bulkheads may significantly modify the mechanical behaviour of the natural slopes, especially at the scale of small volume landslides. More specifically, all of the abovementioned structural interventions are intended to reduce the deformational pattern of a slope by increasing their strength but also increasing their rigidity. Therefore, the soil-structure interactions in landslide issues represent a complex process that require advanced rheological treatment and stress-strain analysis.

In the framework of the presented study, it was possible to analyse the failure behaviour of slopes characterised by full spritz beton coverage and slopes stabilised by anchored bulkheads and to compare their behaviour with that of cut slopes in natural terrain.

The \( \alpha \) and \( A \) values of Eq. (4) were found by back-analysing the landslides characterised by different anthropogenic structural features. It has been observed that the \( \alpha \) value was significantly lower in certain cases than those values reported in the literature for natural slopes. Because Eq. (4) is not effective for \( \alpha \) values lower than one, a new function has been derived in Eq. (5), thus allowing the best fit to the Fukuzono equation in the cases where \( \alpha \) is less than 1. The achieved results show that lower values of \( A \) (sometimes less than 1) and higher values of \( A \) (up to two orders of magnitude) are common for landslides characterised by structural interventions. In Fig. 11, the \( \alpha \) and \( A \) values derived for all of the analysed landslides are plotted together with values from large landslides derived by \([28]\).

As shown in the figure, the \( A \) and \( \alpha \) values for landslides on cut slopes are similar to those from the literature for landslides on natural terrains, but landslides on slopes covered by spritz beton are characterised by the highest values of \( A \) and lower values of \( \alpha \) (Fig. 11).

A novel approach has been implemented to derive the parametric values for landslides characterised by the anchored

![Fig. 9. Linear interpolation of the logarithm of velocity versus the logarithm of acceleration for the data in Fig. 7c and d. The slope of the straight line is equal to \( \alpha = 11 \).](image)

![Fig. 10. Diagram showing the predicted displacement (red dashed lines) and velocity (green dashed lines) vs. time during the tunnelling excavation. (For interpretation of the references to color in this figure legend, the reader is referred to the web version of this article.)](image)

### Table 3

Parameters of the first order for the anchored bulkhead (Figs. 2, 3, 10, 11).

<table>
<thead>
<tr>
<th>Excavation phase</th>
<th>Period of excavation</th>
<th>Excavation meter</th>
<th>Excavation rate ( \text{meter/day} )</th>
<th>( \Omega_t ) mm/h</th>
<th>( \Omega_0 ) mm</th>
<th>( \alpha )</th>
<th>( A )</th>
<th>( t_f ) day</th>
<th>( R^2 )</th>
<th>Monitoring</th>
</tr>
</thead>
<tbody>
<tr>
<td>Second</td>
<td>11/01/10–15/01/10</td>
<td>12–17</td>
<td>1.2</td>
<td>5.00</td>
<td>38.1</td>
<td>0.87</td>
<td>4.96</td>
<td>2.91</td>
<td>0.9524</td>
<td>TInSAR</td>
</tr>
<tr>
<td>Third</td>
<td>21/01/10–29/01/10</td>
<td>17–28</td>
<td>1.4</td>
<td>2.67</td>
<td>44.6</td>
<td>1.01</td>
<td>1.43</td>
<td>3.95</td>
<td>0.9864</td>
<td>TInSAR</td>
</tr>
</tbody>
</table>
bulkheads. This effort was necessary because these landslides did not fail, so a complete dataset was not available (i.e., \( t_f \) and \( \Omega \) were not available). This work may now be considered a reference for this type of analysis.

The \( A \) and \( \alpha \) values derived for the anchored bulkheads lie in an intermediate region of the diagram located between the spritz beton and natural slope values. Furthermore, in analysing the two subsequent phases of excavation, a trend was observed towards a reduction of the \( A \) value (considering the first event as normalised) and an increase of the \( \alpha \) value. This observation is consistent with the expected stiffness decrease of the slope/bulkhead system due to the increase of the total amount of displacement (i.e., moving from the first to the third excavation phase).

6. Conclusions

The efficacy of the time of failure prediction semi-empirical methods by Fukuzono and Voight for landslides characterized by structural stabilization works has been demonstrated by the back analysis of well constrained events occurring during 4 years experiment on a large unstable slope affected by tunnelling activities. Furthermore, a preliminary calibration of \( A \) and \( \alpha \) empirical parameters of the Voight function has been done for both landslides on slope covered by spritz beton and on slope stabilized by anchored bulkheads. For these events, the derived \( \alpha \) values are lower than the standard values for landslides that occur on natural terrains (0.6 to 1.2 instead of 1.3 to 2.2) while the \( A \) values are significantly higher (2.5 to 66 instead of 0.001 to 0.1). Hence, the application of these methods is feasible for predicting landslides that interact with stabilisation structures, but additional calibration by back-analyses of real events is required. The increased rigidity of the slope (which is necessary for reducing the deformations) determines the reduction of the time between the beginning of the displacement and the failure and therefore a reduction in the prediction capability.

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