Landslide Mitigation through Slope Reinforcement, Case Study La Colinas 2001 Landslide

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Abstract

Landslides have been and will be a perpetual and reoccurring hazard worldwide. The extent of damage can range from minor losses in forested areas to complete death and devastation in cities. South and Central American countries are particularly vulnerable to landslides. In 2001 a strong M_w=7.6 earthquake hit the region and devastated El Salvador particularly the Santa Tecla region where thousands of people we buried alive in their homes (Evans and Bent 2004). Mitigation is the key to success for landslides and it has been made possible in different ways. This paper discusses the failure, using information presented in literature and presents a solution using Ischebeck TITAN self-drilling soil nails with a high strength reinforcing mesh on the slope surface.

1 INTRODUCTION

Landslides have been and will be a perpetual and reoccurring hazard worldwide. The extent of damage can range from minor losses in forested areas to complete destruction in built up areas including infrastructure and loss of life. Landslides are a natural hazard that usually occur after triggering events, such as earthquakes and heavy rainfall.

South and Central American countries bordering the Pacific Ocean are particularly vulnerable to deadly hazards. This area forms part of the ring of fire and is unfortunately susceptible to every possible natural disaster. From the beginning of 2019 until October 2019, approximately 200 deaths were recorded due to landslides, flooding events while thousands of persons were injured, and left homeless. In other years, landslides claimed thousands of lives.

The El Niño Southern Oscillation (ENSO) cycle refers to the temperature fluctuations between the ocean and atmosphere of the central and eastern tropical Pacific Ocean (NOAA 2019). Traditionally this weather system occurred every two to twelve years causing heavy and torrential rainfall across the region. Now the effects are felt for a maximum of twelve months at a time with an event frequency of every two to seven years (NOAA 2019).

2 CASE STUDY: LAS COLINAS

On January 13 2001, San Salvador the capital of El Salvador, suffered a major earthquake, considered at that time to be one of the most powerful to strike Central America in the past fifty years (Konagai, Orense and Johansson 2009). The quake occurred at 17:33 (Bommer, et al. 2002) local time with an epicentral distance of approximately 110km SE of San Salvador and moment magnitude, M_w of 7.6 (Evans and Bent 2004). Figure 1 shows the strong motion record.

Hundreds of landslides were triggered across the country, after the earthquake and thousands of people were buried alive in their homes while countless were injured and left homeless (see Figure 2). The most devastating landslide occurred in the region of Santa Tecla near the Cordillera Balsamo. This calamity attracted many foreign aid officials to help in search, rescue and rehabilitative operations in the area.

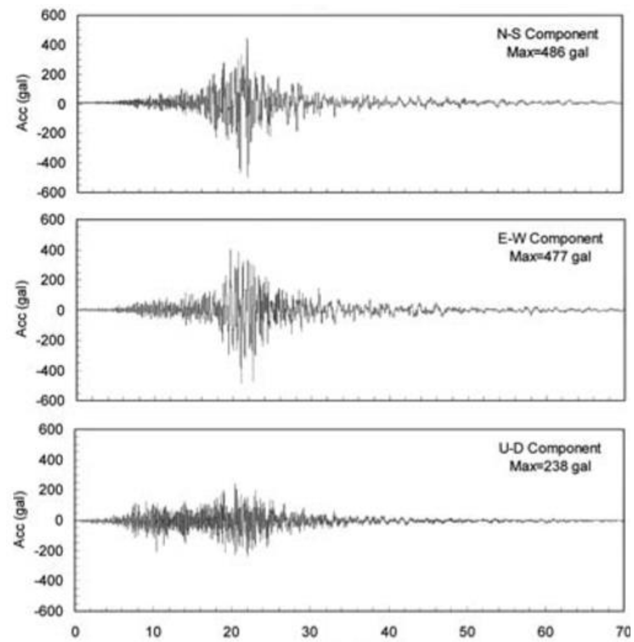

Figure 1: Acceleration records at Santa Tecla (original record (Universidad Centro Americana 2001 n.d.)) recorded in Galileo Galilei where 1gal = 1cm/s2.

A problem such as this usually points to instability due to an accumulation of pore water pressures associated with heavy rainfall; however, records have shown particularly dry conditions corresponding to an abnormally dry rainy season (Konagai, Orense and Johansson 2009). Other reasons for instability have been presented by

Figure 2: The Las Colinas landslide with its huge inpact on the community at the foot of the slope (BBC 2001).

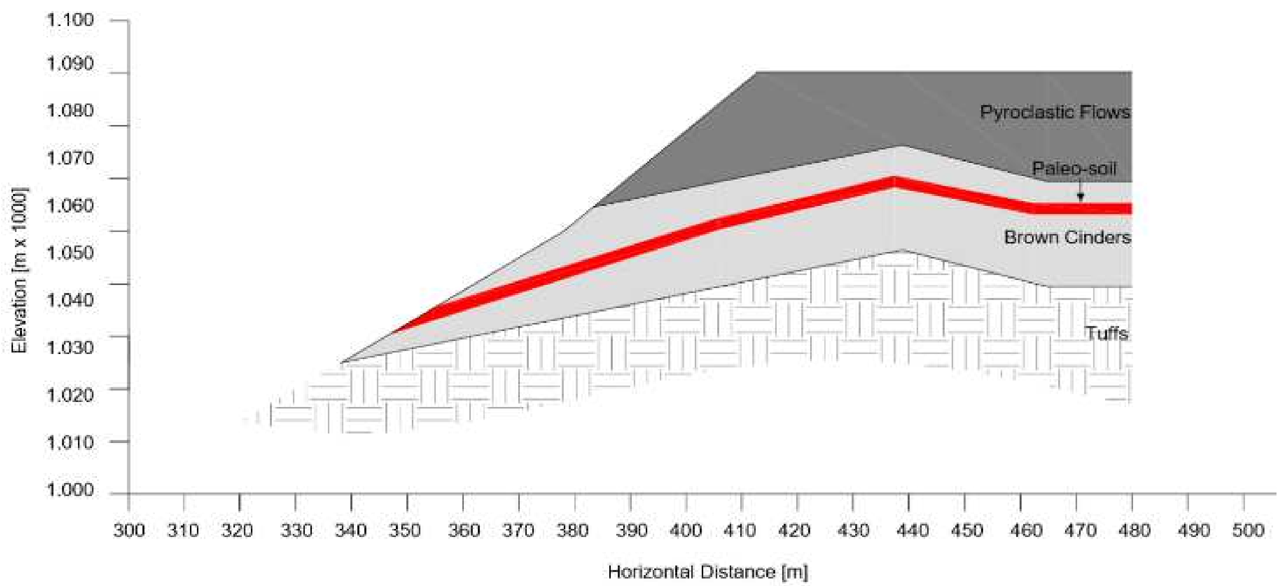

Figure 3: The geological scheme adapted from that of C. Lotti & Associati, 2001.

several authors (C. Lotti & Associati 2001) & (Konagai, Orense and Johansson 2009):

- Low mechanical characteristics of some soil layers.
- The inclination of some layers within the slope in the direction of sliding.
- Site amplification due to the site's topology.
- Possible liquefaction within the slope.
- Development of the slope for residential housing.

However, the exact reason for the failure is still unclear, more than nineteen years after the disaster.

This paper will discuss possible reasons for instability as presented by other authors and the possibility to mitigate against such failures using easy to install methods with minor changes to the natural environment.

Slope reinforcement could have prevented such an event and could be the key to reducing the devastation due to landslides in the future.

3 SITE CONDITIONS

A detailed geotechnical site investigation was performed (C. Lotti & Associati 2001) from which the soil parameters were determined from various soil tests as given in Table 1.

From these site investigations, four main layers were discovered (Figure 3): the uppermost layer contained approximately 1m of lithoid elements. Just below this was the pyroclastic deposits comprising pumice, volcanic ash, sprouts and paleo-soil. Underlain by brown ash/ cinders followed by tuffs formation in which the SPT tests were generally rejected (C. Lotti & Associati 2001).

Table 1: The soil parameters as discovered in the soil investigation (after C. Lotti & Associati, 2001).

Soil	Friction Angle [°]	Cohesion [kPa]	Unit Weight [kN/m^3]
Pyroclasts	30-35	60-80	11
Brown Cinders	30-33	30-40	11
Paleo-soil	20-24	5-10	11
Tuffs	35-38	200	18

Another site investigation (Konagai, Orense and Johansson 2009) was performed after the event, where dark brown coloured stripes of soil that were thinning towards the surface were observed. This indicated that the soils were drying and may have been wet before the event. Another major observation was that all vegetation was removed from the top of the slope in preparation for coffee plantations.

A study performed on volcanic soils suggested that in dry conditions these soils tended to be rather lightweight. With low stress states, the material is brittle and becomes increasingly more ductile with increasing confining pressures. As saturation increases however, these soils exhibit a proportional decrease in shear strength resulting in landslides and failure during periods of intense rainfall (Bommer, et al. 2002).

The undrained shear strength (c_u) of these volcanic soils tend to be significantly dependent on the clay mineral content and structure with values ranging between 60 and 125kPa (Bommer, et al. 2002). This paper also made mention of the *Tierra Blanca* (White Soil) found in San Salvador, described as a poorly consolidated soil that developed from ash after several volcanic eruptions. This type of soil has characteristically negative pore water pressures and low cementitious properties (Bommer, et al. 2002).

As shown in Table 1 the paleo-soil was identified as that with the weakest shear strength parameters and a zone for potential failure.

A finite element model was developed (Dat Vu Khoa and Jostad 2010), using the information from the site investigations (C. Lotti & Associati 2001). In this work, the slope seemed relatively stable according to calculation and failure was only achieved by implementing the Hill's sufficient condition of stability. The paper concluded that employing the second-order work, the landslide was better visualized as that experienced in reality.

Few persons who studied this failure have accounted extensively for the possible site effects due to the amplification of the seismic action because of the geological make-up of the soil. When the seismic waves move from the tuffs to the pyroclasts and brown ash layers, the waves are amplified and even resonance can occur when the dominant frequencies collide and result in greater damage (Kattan, Lopez and Menjivar 2017).

4 PSEUDO-STATIC CALCULATIONS

In this paper, a pseudo-static analysis was performed considering the geometry and soil layers presented in Figure 3 to determine a solution and the calculations will be described below. This analysis is effectively done through the application of limiting equilibrium practices that considers a force or moment of a particular soil above the potential failure surface (Akhlaghi and Nikkar 2014). The values used for the seismic coefficient have a grave impact on the slope calculation in determining the possible factor of safety and sliding mechanism.

This seismic coefficient controls the failure mass, related to the measure of amplitude of inertial forces induced during instability (Gurudeo and Rajani D. 2017). In the case of a rigid body, the inertial force is simply calculated as the product of the actual horizontal acceleration and sliding mass. This inertial force will reach its maximum value when the horizontal acceleration is maximum. However, since in most cases, the problem is not considering a rigid body, the maximum acceleration would only last for a short period and is not a representative seismic coefficient. In practice, these pseudo-static coefficients are significantly below the maximum acceleration and Terzaghi originally outlined some guidelines (Table 2).

Table 2: The estimated horizontal seismic coefficients according to Terzaghi. *(Tergzaghi 1950)*

k_h	Earthquake Intensity
0.1	severe earthquakes
0.2	violent and destructive earthquakes
0.5	catastrophic earthquakes

A study performed, using shear beam models indicated that deformations of earth dams comprising ductile soils with a maximum acceleration less than 0.75g, would allow for considerably small pseudo-static factors of safety and further with k_h = 0.1 (M = 6.5) to k_h = 0.15 (M = 8.25) (Seed and Martin 1996). From this study, they concluded that the pseudo-static accelerations are between 13 to 20% of the peak crest acceleration. Applying the Newmark sliding block analysis, again for earth dams with a pseudo-static FOS greater than 1.0, the following equation can be derived (Hynes-Griffin and Franklin 1984).

$$0.5 \cdot a_{max}/g \quad (1)$$

The system was analysed in GGU Stability with a Mohr-Coulomb constitutive model using Bishop's Method to determine the factor of safety and the risk of sliding. All the slope parameters, given in Table 1 were inserted in the program including the seismic action via horizontal and vertical seismic coefficients.

To include the seismic action in GGU, a seismic coefficient must be determined since it is only a fraction of the PGA. As previously described, it is somewhat complicated to determine the seismic coefficient k_h or k_v and they should be an accurate representation of the vibration effect due to a given earthquake (Papadimitriou, et al. 2010).

The reports all established a peak ground acceleration (PGA) of 0.5g but for the determination of these seismic coefficients, the strong motion data was acquired from the nearby stations (Table 2).

Table 3: excerpt of strong motion records of 13 January 2001, San Salvador earthquake (Bommer, et al. 2002).

Network	Station	PGA (g)		
		N-S	E-W	V
CIG	San Salvador DB	0.225	0.250	0.160
CIG	San Salvador RE	0.304	0.324	0.329

Considering these measured values at the CIG San Salvador RE Station given in Table 3 above and Equation (1), the values used in the model were $k_h = 0.162$ and $k_v = 0.165$ respectively. These values are consistent with those presented by Terzaghi for a severe to violent and destructive earthquake (Table 2).

An accumulation of pore water pressure at the paleo-soil layer was assumed, consistent with the findings of the geotechnical reports (Konagai, Orense and Johansson 2009) and the calculations were performed as such.

As depicted in Figure 4, the instability of the unreinforced slope was confirmed (minimum factor of safety less than 1).

5 SOIL REINFORCEMENT

To increase the factor of safety of the slope, soil nails were introduced in the model as reinforcement. These would be designed and installed such that the environmental impact is minimal and the safety of the slope is realized. Like this slide in Santa Tecla, the possibility of such a failure can occur on the adjacent slopes, sharing similar properties and incidence angles, as well as any other susceptible slopes in the region.

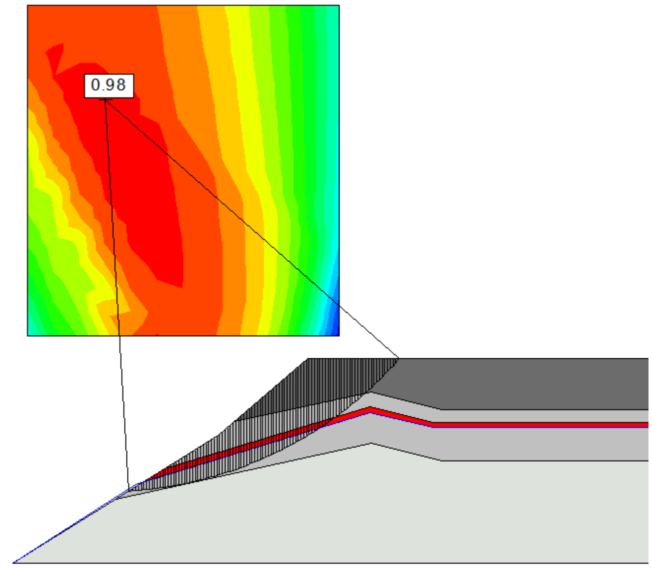

Figure 4: The fcator of safety after the analysis of the slope including pore water.

Soil nails allow tensile forces to be accommodated, which would otherwise be impossible. The load is transferred to the surrounding soil via skin friction. This specifically refers to the bond between the grout - soil interface and in this particular case; the bond in the volcanic tuffs defines the design. These values were determined from several tests on different volcanic tuffs performed in various studies.

These studies were summarized (Covassi, Zeballos and Gorosito 2015) correlating the unconfined compressive strength and skin friction. Based on the correlations a value of 0.5MPa was assumed for the tuffs layer and used to design the soil nails.

5.1 SOIL NAIL DESIGN

In this particular study, Ischebeck TITAN, self-drilling soil nails were used for the calculations. Advantages of this solution include:

- Suitable for use in areas where working space is limited or challenging.
- Varying soil conditions do not pose a threat for this system and the soil nails can be adapted for every possible scenario.
- Permanent corrosion protection by means of grout cover is provided.
- They can be installed in both permanent and temporary situations.

Several calculations were performed until a minimum factor of safety of 1.2 was achieved. Figure 5 shows the slope reinforced with 17 soil nails and details are presented in Table 4. The nails are labelled from the base of the slope upwards. From this table the soil nails labelled one (1) through nine (9) are very long since this was found to be the most critical part of the slope. The upper, labelled ten (10) through seventeen (17) are considerably shorter to prevent local failure that may occur. The longer nails are all inclined 45° to the horizontal and the shorter, 40°:

Table 4: The details of each soil nail including the inner and outer diameters of the hollow bars (d), length (L), diameter of grouted soil nail (D) and characteristic bearing capacity R_k.

Nr	**Diameter [d,mm]**		**L [m]**	**D [mm]**	**R_k [kN]**
	Outer	Inner			
1-4	73	35	15	150	1390
5-7	73	35	27	150	1390
8-9	73	35	30	150	1390
10-17	30	16	4.5	110	190

These soil nails should be installed in accordance with the best engineering practices (e.g. FHWA 2003, EN14490).

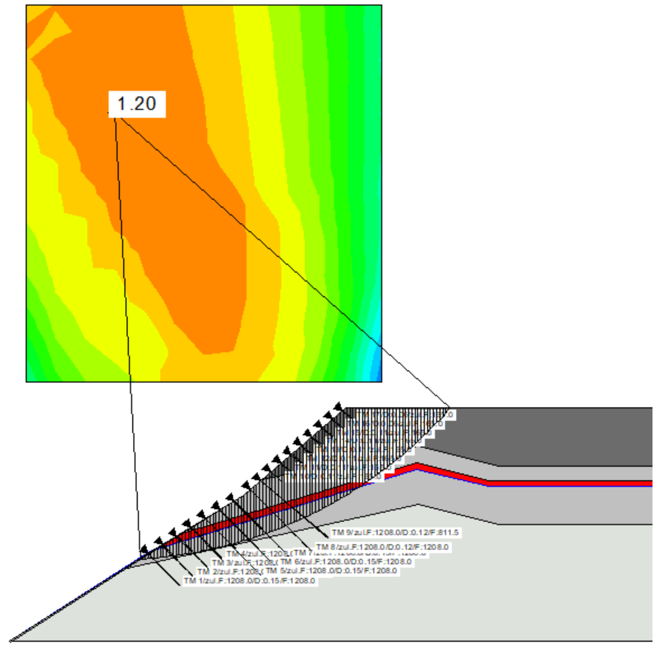

Figure 5: The final design showing the configuration of nails spaced 2m in the vertical and 3m in the horizontal directions.

Figure 6 below, shows a section through a typical soil nail configuration including the head construction, couplers to connect the hollow bars and a sacrificial drill bit.

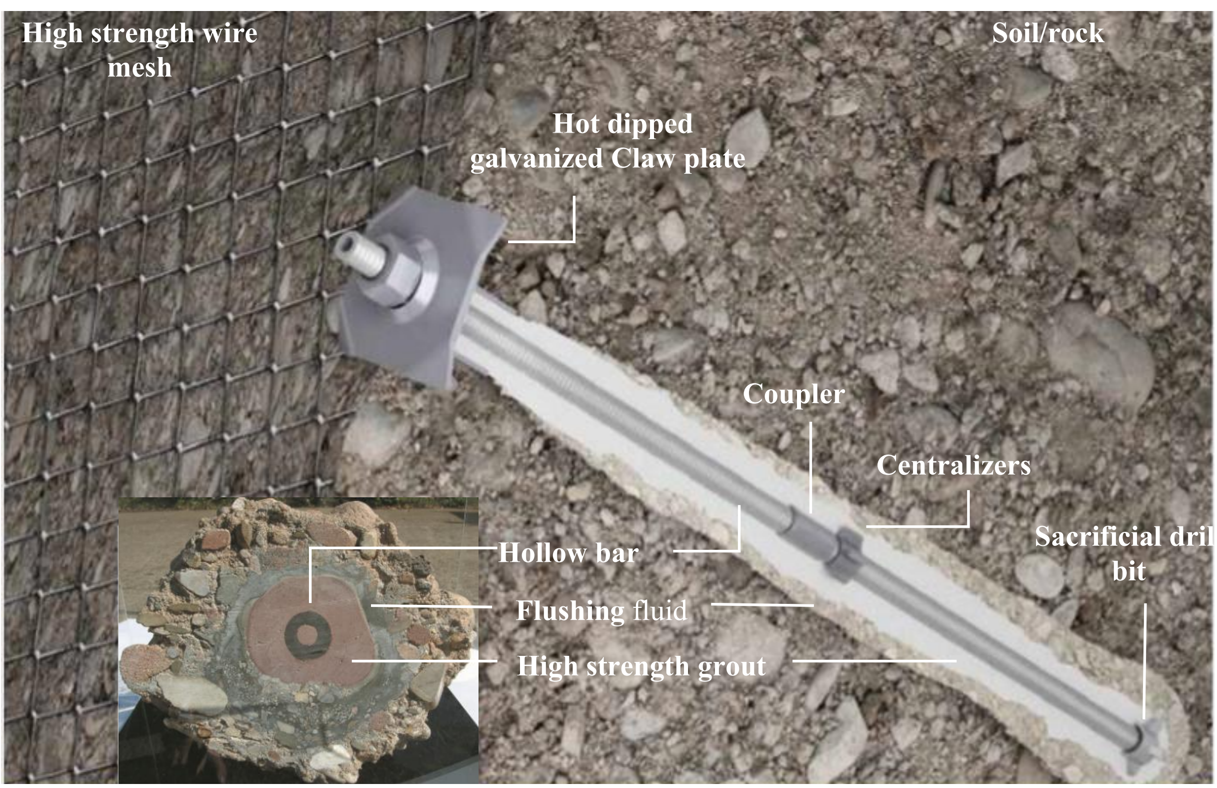

Figure 6: cross-section through a soil nail (Ischebeck TITAN) where the zoomed image (bottom left) shows the cross-section of the grouted body (Friedrich Ischebeck GmbH 2019).

The installation process is very simple and easy and many different sizes of equipment can be used to drill these bars.

Self-drilling soil nails consists of continuously threaded hollow bars, made out of seamless steel pipes, installed via rotary percussive drilling. During the drilling process, the soil nails are continuously grouted (dynamic injection), building a rough interlocking at the interface grout-soil, increasing the skin friction (see Figures 8 and 9).

Figure 7: An example of soil nail installation in a forested area (Friedrich Ischebeck GmbH 2019).

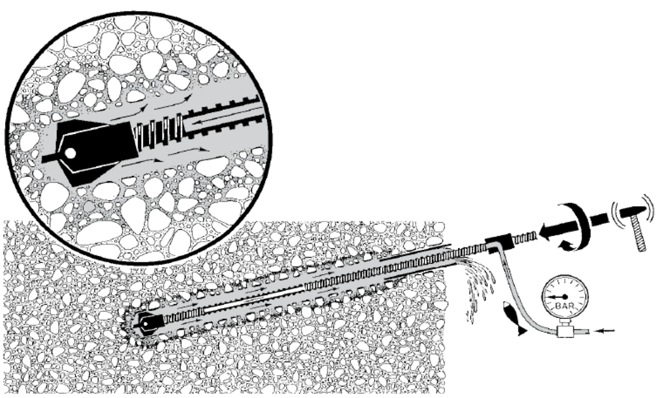

Figure 8: Rotary percussive drilling with flushing grout (w/c = 0.7-0.8) (Friedrich Ischebeck GmbH 2019).

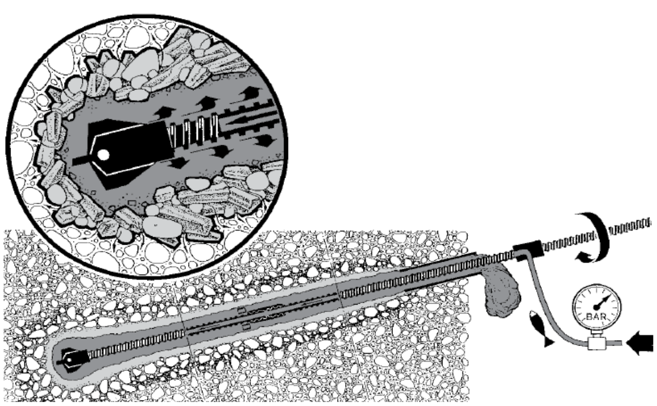

Figure 9: Dynamic pressure grouting (w/c = 0.4-0.5) (Friedrich Ischebeck GmbH 2019).

As a composite material, the permanent use is limited by the corrosion protection of the steel elements. According to the European Standards that regulate the use of composite materials for geotechnical applications, such as soil nails (EN 14490 2010), the corrosion protection can be provided, among other measures, by an efficient encapsulation in grout. Research has shown that crack widths controlled to less than 0.1mm can be considered to be self-healing, therefore, the cement grout is considered acceptable as an impermeable protective encapsulation, provided that the crack width within the grout body can be demonstrated not to exceed 0.1mm (EN 14490 2010). In Germany, this requirement has been adopted by the German Institute of Building Technology to assess the structural behavior of self-drilling soil nails, as highlighted in the National Technical Approval Z.34.14-209 (DIBt 2018).

6 CONCLUSION

Control measures can be implemented in this region since landslides frequently occur and the effects can be minimal or extreme. As such, preventative measures should be adopted to limit or eliminate the risk to life and infrastructure.

This paper presents the analysis of one event that occurred in El Salvador in 2001, triggered by a moment magnitude 7.6 earthquake.

The results of the analysis have shown that the implementation of reinforcement along the slope considerably increased the overall stability. The reinforcement selected consisted of self-drilling soil nails by which are particularly suited for this application. These soil nails are lightweight, with effective installation practices and very little modifications to the forested environment.

The implementation of control measures can prevent events like the Las Colinas landslide, saving many lives and protect infrastructure from destruction.

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