2013 Landslide Failure Mechanism and Back Analysis of Tijuana-Ensenada Scenic Highway

Carlos Chávez(✉), Rafael Soto, and J. Eleazar Arreygue

Universidad Michoacana de San Nicolás de Hidalgo, Morelia Michoacán, México
cachavez@umich.mx

Abstract. The Tijuana-Ensenada highway crosses the "Salsipuedes Bay". This zone has been subject to numerous landslides due mainly to geological conditions. The last great landslide occurred on 28 December 2013. The principal causes that triggered the landslide were: the shale weathering, the groundwater, and the seismicity of the zone. In the present study, a deep sensitivity analysis was performed to reproduce the failure mode to understand the failure mechanisms and the factors that triggered the landslide. The results suggest that the seismic activity and water caused the degradation of the shale. To make a close approximation of the real conditions a finite element analysis was carried out using patterns of joints and adopting different shear strengths for the joints. Finally, the failure mechanism was reproduced in a satisfactory form.

Keywords: Landslide mechanism · Sensitivity analysis · Shale degradation

1 Introduction

On the north-western coast of Baja California near the USA border, a scenic Tijuana-Ensenada Highway was constructed in the 1960s. Since the beginning of construction, the highway had landslides caused by earthworks that affected the critical equilibrium of the geological structures.

In 2000, a detailed study of the zone presented a total of eleven landslides in the scenic highway in Salsipuedes Bay [2]. Through subsequent years, specifically after 2011, signs of movement were detected in the section between 83.0 km and 98.5 km, which prompted a geotechnical study to determine the cause of the movement [1]. A failure, involving 1,440,000 m^3, referred to as the "km 93.0 Landslide" occurred on December 28, 2013, in the highway segment between 92.98 km and 93.24 km. The possible causes that triggered the landslide were the seismic activity and the groundwater observed in the bending planes.

Despite all the information collected and efforts to determine a solution for the slope failure, a detailed numerical simulation representing field conditions was not performed. The purpose of this paper is to present a sensitivity analysis of the case that ultimately replicates the slope failure mechanism, to understand the triggering causes, and be alert of another possible landslide in the zone.

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2 Background and Stratigraphy of the Area

With the local geology [2], it was possible to build a representative geological cross-section [1, 7], as is shown in Fig. 4. The stratigraphy consists of alternating sandstones with thin layers of the shales with an inclination of 6.5 degrees dipping toward the sea.

The 19 November 2013, registered a seismic activity with an intensity of 4.2 Richter scale, this event produces a vertical displacement of 0.15 m to the km 93.0 embankment. So, seismic activity was reported as one of the probable causes of triggering the landslide, in conjunction with the groundwater [8].

3 Mechanical Properties

3.1 Failure Criterion and Mechanical Properties

Modified Hoek and Brown Failure Criterion (MHBFC) was considered for reproducing the mechanical behavior of the sandstone mass rock [3]. Because of a lack of information, typical values of parameters, m_i and compressive strength, were obtained from tables [3], and the GSI was proposed from field observations. The parameters are shown in Table 1.

Rico [6] performed direct and annular shear tests on interfaces of sandstones and weathered shales. The residual angles of internal friction were found to range between 6.5 and 12.4 degrees, and from back-analysis ranged between 7.1 and 20.6 degrees. The variation of peak friction angle ranging from 24 to 34 degrees was derived from stress–strain graphs presented by Rico [6]. The above information is used as a starting point for slope stability analysis presented in the next section.

Table 1 Parameters for the Hoek and Brown failure criterion

Parameter	Value
Compressive strength (kPa)	50,000
Geological strength index, *GSI*	35
Intact rock constant, m_i	17
Disturbance factor, *D*	0

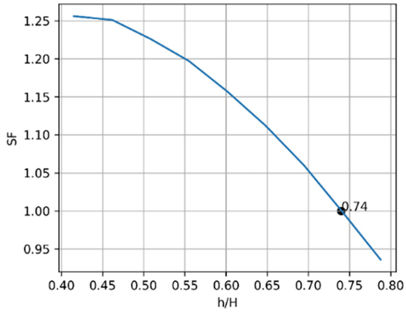

Fig. 1 Safety factor with different water table levels

4 Numerical Analysis

The geological cross-section was taken as a basis to build numerical models. Diverse types of numerical slope stability analyses were carried out for understanding the failure mechanism.

4.1 Limit Equilibrium Analysis

To simplify the analysis a sensitivity analysis was performed, in which the mobilized friction angle of shale was set at 10 degrees and cohesion of zero. The height of the water table level (h) was varied, and the last level of shale was set as critical because, in situ, the failure was located at that level. H is the maximum height of the slope. The relationship between h/H and SF is non-linear, and a value of the relationship of h/H equal to 0.74 gives a critical pore water pressure, that produces SF equal to one (see Fig. 1).

4.2 Seismic Analysis

In the present work, a parametric study is carried out to determine the critical seismic coefficients, k_c, considering three different mobilized friction angles and variations of the water table level. Mobilized friction angles considered are 15, 12.5, and 10 degrees to evaluate different scenarios, in which the shale resistance degrades, assuming zero cohesion.

Figure 2 displays the results of the parametric analysis using the Morgenstern- Price method, the lines are erratic, and it is due to different critical failure surfaces obtained during the search. Values of critical seismic coefficients until 0.207 g were obtained for a mobilized friction angle of 15 degrees. The largest magnitudes of earthquakes in "Salsipuedes Bay" are of a magnitude of 4.5, with the potential for growth to 6 [5]. Then, according to the seismicity of the zone, the seismic coefficients for design should not be greater than 0.1 g. Considering k_c from 0.0 to 0.05 g values, it is possible to observe in Fig. 2, that the mobilized friction angles of 10 and 12.5° with the correct relationship of h/H for each one can cause failure.

Another more advanced alternative for seismic analysis is the Newmark method [4]. For this case, a seismogram representative of the region, of a magnitude of 4 was considered, and the peak acceleration was modified to match that estimated for the site. Then, a parametric analysis was performed to obtain the corresponding displacements. The peak acceleration was estimated with the United States Geological Survey (USGS) database. The epicenter of the considered event was located 87 km from the Maneadero B. C. Mexico, the date was 02 September 2015 with a magnitude of 4.89. From a peak acceleration map, a peak acceleration of 0.067 g was obtained for the "Salsipuedes Bay" zone.

Figure 3 shows the results of the Newmark analysis; the horizontal axis represents the peak acceleration fraction (PAF) of 0.067 g. The maximum displacement is 324 cm for a mobilized friction angle interval of 7–7.375 degrees, and it decreases with the magnitude of the PAF. Parrish [5] considers that slopes with displacements greater than 100 cm are unstable. Interpreting the numeric simulation results of Fig. 3 regarding the Parrish limits, the natural slope is unstable for the PAF between 0.545 (0.036 g) and 0.72 (0.048 g) with mobilized friction angles of 7.0 to 7.4 degrees. There is a sudden decrement of the displacement for a mobilized friction angle of 7.5 degrees, then the condition to trigger the seismic failure is a frictional angle less than 7.5.

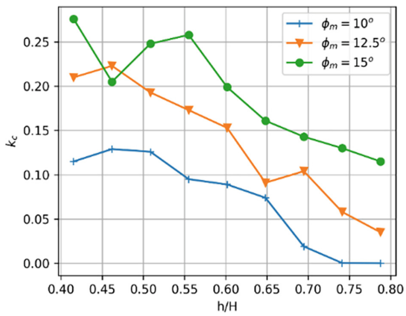

Fig. 2 The critical seismic coefficient for mobilized friction angles

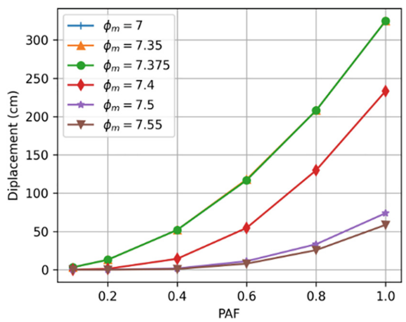

Fig. 3 Newmark Analysis for different peak acceleration fraction of 0.067 g

4.3 Strength Reduction Analysis

One of the most powerful tools to perform slope stability analysis is Shear Strength Reduction (SSR). In this case, the Rocscience RS2 two-dimensional finite-element program was employed.

Field observations helped to conceptualize the representation of the model (Fig. 4). The idea is to adopt a joint set for representing the bedding planes in sandstone, like the mass rock in situ. Two main joint patterns were used: cross-jointed and Voronoi. The sandstone cross-jointed pattern is used and has two principal joint sets: the bedding set that is dipping 6.5 degrees out of the slope face and is the principal potential plane of failure; the set perpendicular to bedding, where tension cracks can develop. Additionally, there are three levels where the bedding planes are filled with shale and its mean thickness is 0.2 m. In Fig. 4 two types of shale are represented: a degraded shale (shale M) and non-degraded (shale Inter); these two types of shale were necessary for reproducing the landslide mechanism seen in situ. Talus deposit is simulated like a rockfill, with a Voronoi pattern that is a simplified form of a deposit, the number of Voronoi cells per square meter was 0.4. In the upper right side in the cross-section of Fig. 4, a cross-jointed pattern represents in situ jointed sandstone that is distinct from the talus. From the above analysis, a suitable groundwater level was chosen and remained consistent in the subsequent analysis.

The properties used are presented in Table 2. A special case is the shale layer, to produce the failure mechanism, were proposed different properties in the function of the zone, because of the brittle behavior of the shale. Different properties of shale were used within the same shale layer to reflect differential weathering relative to the slope face.

A sensitivity analysis was performed to understand the dependence of the seismic coefficient on the stability analysis. Horizontal seismic coefficients of 0.01, 0.02, 0.03, 0.04, and 0.05 g were supposed for the analysis. Additionally, the peak friction angle is varied, and the residual friction angle was fixed at 6.5 degrees according to Rico et al. (1976). The results of the analysis are shown in Fig. 5, on the vertical axis, the safety factor, SF, and on the horizontal axis the ϕ_p is for pseudo-static SSR analysis. The first thing to note is that there is a combination of the peak friction angle and the seismic coefficient which results in a factor equal to or less than one. It should be noted that failure occurs with peak friction higher than the residual angles, as large as 20 degrees.

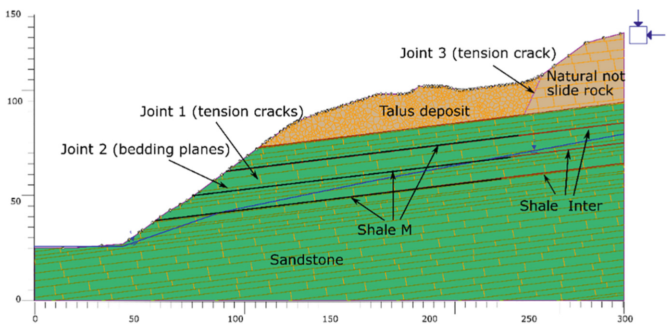

Fig. 4 The pattern of Joints used for the simulation

Table 2 Mechanical properties of joints

Material	c peak (kPa)	ϕ peak (^{o})	c residual (kPa)	ϕ residual (^{o})
Joint 1 and rockfill (vertical)	200	35	0	35
Joint 2 (horizontal)	200	35	0	20
Shale B	10	25	0	15
Shale M	10	10	0	6.5
Shale Inter	10	20	0	12

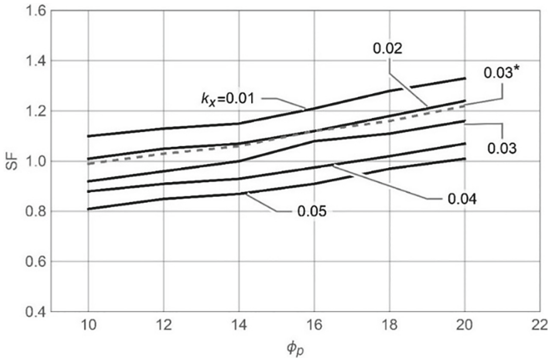

Fig. 5 Safety factor vs peak friction angle for pseudo-static SSR analysis

In this case, the in situ peak friction angle is not known, the previous analysis reveals that the failure occurred with a mobilized peak friction angle between the in-situ peak (24^{o}) and residual angles.

The mobilized friction angle depends on the factors that trigger the sliding, such as seismic activity and the water table level. It's viewed in Fig. 5 that the seismic force overcomes the shear strength of the soil taking it to residual strength, with values as low as 0.02 g. The case of k_x equal to 0.03* in Fig. 5 corresponds to the analysis without the water table level, in this analysis, the SF is increased by about 7%.

5 Conclusions

The 2013 landslide that occurred in "Salsipuedes Bay" are among the biggest (approx. 1,440,000 m^3) that occurred in Mexico, so a detailed back stability analysis should be conducted, due to the lack of mechanical properties. In the present research, a sensitivity analysis was carried out that considers the geological-geotechnical conditions of the zone.

In the case of the variation of the water table, a nonlinear dependence was found, and the water table level of 0.75 of the maximum height was high enough to intersect the shale beds and lubricate them to produce the failure. There is another factor that contributes to triggering the landslides, seismic forces. A pseudo-static limit equilibrium analysis was done to determine the critical seismic coefficients that cause the failure considering different mobilized friction angles. The critical pseudo-static factor for mobilized friction angles of 10 and 12.5 degrees is as low as 0.05 g and decreases with increasing groundwater levels.

An analysis of displacements was carried out using the Newmark method with a peak acceleration of 0.067 g obtained from the USGS database, which resembles the 0.05 g deduced previously. According to the Newmark method, a displacement between 15 and 100 cm degrades the shale and higher than 100 cm can cause failure. The results showed that the failure can occur with a 0.048 g peak ground acceleration and mobilized a friction angle of 7.4 degrees. Both seismic analyses arrive at a similar seismic coefficient, but the friction angle is different, it can conclude that the range of variation for the mobilized friction angle is 7.4 to 12.5 degrees, obtain with the limit equilibrium technique.

The finite element method allows the making of a complex numerical model that considers discontinuities named joint networks. Based on the field reports, it was proposed joint patterns representing rock structures in different zones. It is shown that the landslide was triggered with a mobilized friction angle between 14 to 16.4°, additionally, low cohesion is considered. With the help of seismic events and groundwater, the landslide is presented. The most feasible seismic coefficient is 0.04 g, it can initiate the movement, and a typical water table can decrease the factor of safety by about 7%. In conclusion, the low seismic activity in the zone is strong enough to accelerate the shale failure, and the presence of the groundwater aggravates the stability.

A more detailed study of the deterioration of shale in Salsipuedes Bay is necessary to understand the presence and evolution of landslides over time.

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