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Estimation of the Permanent Seismic Displacements of the Costa Verde Cliffs, Lima, Peru

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ABSTRACT

The city of Lima is located in one of the most seismically active regions on Earth. According to seismic history, in the year 1746 off the coast of Lima an earthquake occurred whose seismic intensity was X on the Modified Mercalli scale, a movement of this size could generate instability to the cliffs of the Costa Verde and produce large displacements in the slopes. For this reason, the estimation of slope displacements in the event of a similar earthquake was carried out. The Costa Verde Cliffs are an important part of the city of Lima, so ensuring their physical stability is of interest to many professionals and state and private institutions. These cliffs have a height that varies from 50 m to 60 m and a slope greater than 60° of inclination and are generally formed by the Lima Conglomerate in almost all its longitudinal extension. The strength of this material has been determined by in situ direct shear and large-scale laboratory direct shear tests, giving average values of friction angle of 40° and cohesion of 55 kPa. An extensive collection of information on the physical and mechanical parameters of the conglomerate was undertaken to reevaluated the stability of slopes in areas that were considered at low, medium and high risk in previous studies, to verify if they can still be considered as stable under pseudostatic conditions and the estimated permanent seismic displacements can be consistent for the Lima Conglomerate.

Keywords: conglomerate, frictional angle, slope stability, seismic displacement

1 INTRODUCTION

The cliffs have an extension of approximately 16 km and cover the districts of San Miguel, Magdalena del Mar, San Isidro, Miraflores, Barranco and Chorrillos, in the city of Lima-Peru (Figure 1). The engineering infrastructure near the cliffs of the Costa Verde, such as buildings, recreational areas, and access roads (Figure 2) make these areas important for commerce and tourism in the city of Lima. These cliffs are conformed of the Lima Conglomerate, which have suffered the phenomenon of landslides and rockfall in various sections, so it is of interest to evaluate the static and seismic stability of the slopes.

The landslide on the lower and upper access roads is an old and important problem that has not been completely solved, and considering that the cliff is located in a seismic zone, the problem may have greater consequences than to be regretted. Nowadays, the cliffs present a superficial protection measure against landslides by means of the use of meshes, which in the course of time have proved to be insufficient.

In the past, to evaluate the stability of cliff slopes, researchers generally assumed soil parameters due to the scarcity of laboratory tests and detailed studies (Sanchez, 1975; Garcia, 1982; Carrillo, 1984a, 1984b). In recent years, due to the demand for large investment projects, in-situ direct shear tests and large-scale laboratory shear tests have been carried out, the results of which have allowed a better understanding of the behavior of the Lima Conglomerate, as indicated in the research of Huamán (2000), Cañari (2001), public entities such as the Peruvian-Japanese Center for Seismic Research and Disaster Mitigation (CISMID) and private entities. In addition, recent investigations have characterized the dynamic behavior of the conglomerate through measurements of the S-wave velocity profile and the predominant period of vibration.

Currently, investigations of the Costa Verde Cliffs continue due to improvements in field and laboratory test measurements, as well as simplified techniques for evaluating physical stability and determining permanent seismic displacements. This document offers a briefly evaluation of the Costa Verde, reviewing the contributions made in the last 25 years by various professionals and national institutions.

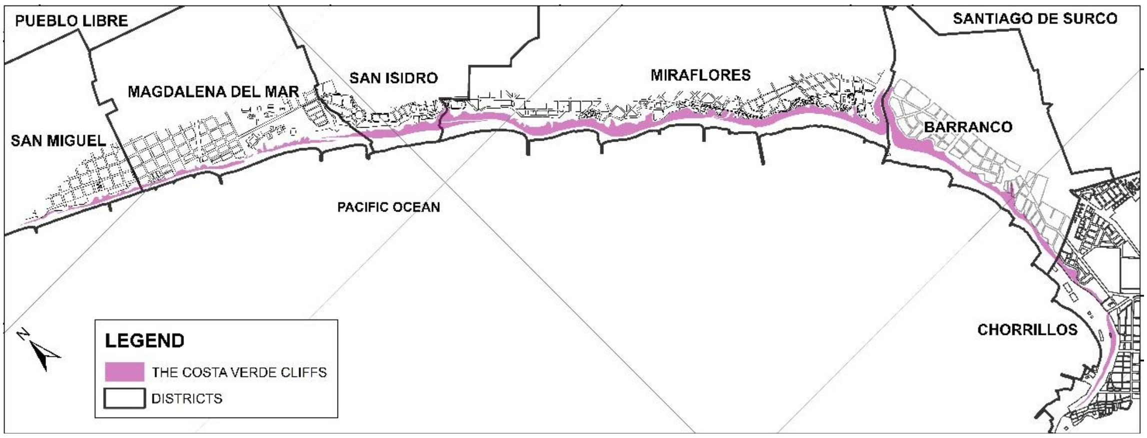

Figure 1. *Plan view of the Costa Verde Cliffs*

Figure 2. *Frontal view of The Costa Verde Cliffs*

2 GEOTECHNICAL CHARACTERISTICS

2.1 Geology and Geomorphology

Lithological described it is conformed of fluvial deposits that were transported by the Rimac River, these constitute the well-known Lima Conglomerate, and this deposit also contains lenses of fine material (conformed by silty sand, clayey silt of low plasticity) of variable thickness. This fluvial deposit naturally has a certain degree of natural cementation and can be observed on the slopes of the Costa Verde cliffs. Geomorphology described that the cliffs are a discontinuity of the alluvial cone of the Rimac River due to marine erosion, and there are also gullies in certain sections due to surface water erosion (Figure 3).

On the other hand, external geodynamics is characterized by rock falls due to wind erosion of the cliffs and erosion from irrigation of the green areas, as well as soil sliding due to its own weight and seismic action, as occurred in the May 1940 earthquake (Valencia, 1940).

2.2 Stratigraphy of Soils

An extensive compilation of various studies in each district of the cliffs and observations at the foot of slope has allowed us to identify the stratigraphy which is largely conformed by conglomerate, and to a lesser extent sandy and clayey soils, a more detailed description of the stratigraphy by each district is shown in Alva and Soto (2022). In summary, conglomerate is found along the entire length of the Costa

Verde Cliffs for almost the entire height of these natural slopes, and soils such as sands and clayey silts are found in Barranco and Chorrillos with a thickness of up to 8 m measured from the ground surface.

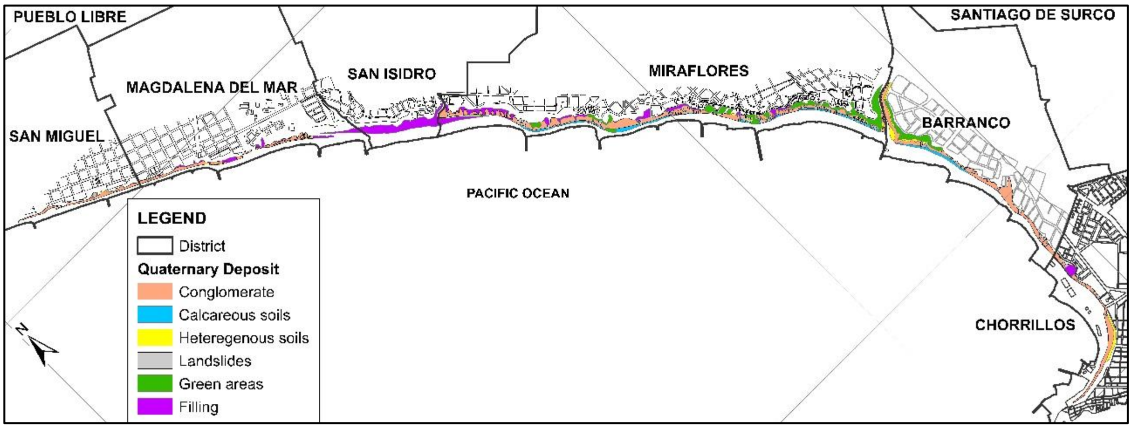

Figure 3. *Geology and Geomorphology of the Costa Verde Cliffs*

2.3 Index Properties

The soils that conformed the cliffs have the following index properties:

- For clayey and silty soils the average dry density is 1.44 g/cm^3, and the moisture content is 20.8% with a Liquid Limit (LL) ranging from 21% to 68% and a Plasticity Index (PI) ranging from 1.7% to 42.0%.
- For the sandy soils, the average dry density is 1.76 g/cm^3, with an average moisture content of 8.47%, the compactness varies from dense to very dense, given that they are in a dry state and a certain degree of cementation observed during the standard penetration tests (SPT).
- The conglomerate is conformed of pebbles (maximum size of 12") and gravels with a matrix of sands and silts, the latter two were considered as fine soils because the conglomerate and gravels are present in 80% of weight percentage. Dry density is 2.21 g/cm^3 and moisture content is 3.65%, values close to those obtained by Sanchez et al, (2016) who considered the variability of these parameters of the Lima Conglomerate from the first 5 m to 30 m depth. Naturally the conglomerate is a granular soil that presents a varied gradation, it has a natural cementation which its bearing capacity is 7 kg/cm^2 for shallow foundations.

2.4 Strength Parameters

The characteristics of the shear strength parameters of the Lima conglomerate was performed by in situ direct shear (DC) tests (Huamán, 2000; Cañari, 2001) or alternatively by the back-calculation method (Pacheco, 2006; Granados, 2006; Díaz, 2006); while for the fine soils (silty sands, silts and low plasticity clays) the strength characteristic was performed by direct shear tests (Huamán, 2000; Cañari, 2001). On the other hand, JAHI (2009) carried out two in situ CD tests to determine the strength parameters of the conglomerate for a shopping center in Miraflores, one at the foot of the slope and the other at 40 m above sea level. Alva and Soto (2022) presented a summary of the resistance parameters obtained from in situ CD and large-scale CD tests of the soils that conformed the cliffs, with these a sensitivity analysis of the variation of the parameters of ϕ' and C' was performed. Table 1 shows the mean values of ϕ' and C' obtained only from the in situ CD tests.

Table 1. *In situ strength parameter of soils of the Costa Verde Cliffs*

Soil	**ϕ' (°)**	**C' (kg/cm^2)**
Conglomerate	40.0	0.5
Fine (silty sand)	28.5	0.9

3 SLOPE ESTABILITY

3.1 Previous Research

Several undergraduate and graduate theses (Ayquipa 1995; Huaman, 2000; Cañari, 2001; Silva, 2003; Macazana, 2006; Granados, 2006; Pacheco, 2006; Díaz, 2006; Raygada, 2011), as well as studies conducted by CISMID (2015a, 2015b, 2017) evaluated slope stability in certain areas of the districts of San Miguel, Magdalena del Mar, San Isidro, Miraflores, Barranco and Chorrillos, of the cliffs of the Costa Verde. For this, they used the Method of Limit Equilibrium (MEL) to evaluate the stability of the slopes by means of the Factor of Safety defined as the quotient between the resisting forces of the terrain and the forces acting on the terrain (weight of the terrain, overburden, and earthquake effect). The maximum acceleration for a design earthquake and extreme earthquake was 0.44 g and 0.53 g, respectively, where the effective acceleration was considered between 25% and 30% of the instrumental acceleration, consequently, the seismic coefficient (k) was 0.20. The results obtained indicate that certain zones are stable and others unstable in case of a design seismic event.

After an exhaustive review of the information from the aforementioned authors, it was found that the most extensive slope stability results for the entire coast of the cliffs were those of Huaman (2000) and Cañari (2001) in terms of Stability Risk, which considers the Factor of Safety (FS), geometric aspects (height and slope) that represent potential damage to human life and damage to material property. Figure 4 shows the zones considered to be at very high, high, medium and low risk.

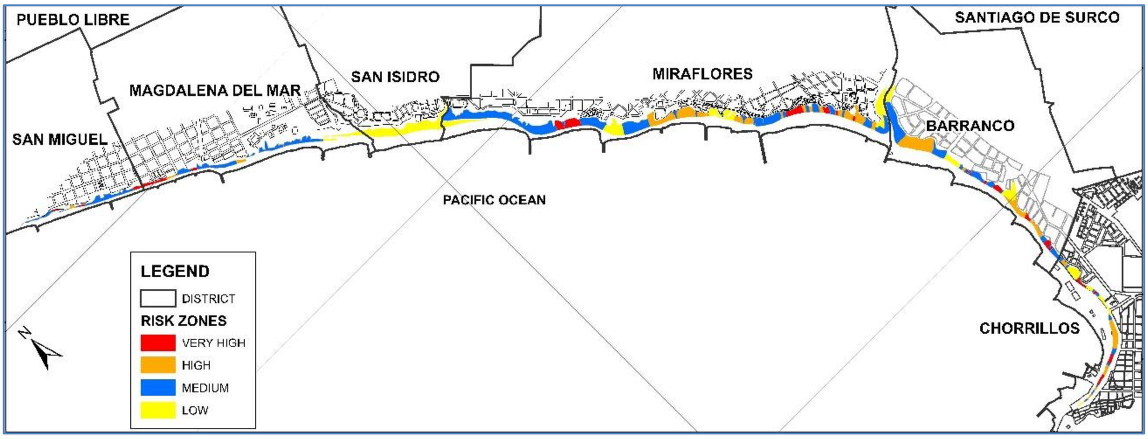

Figure 4. *Risk of landslide of the Costa Verde Cliffs*

The above previous studies did not determine the level of displacement that can be generated by strong ground motion, however, the research carried out by Macazana (2006) and Raygada (2011) estimated the level of displacement by the finite element method.

Macazana (2006) evaluated the seismic response of the slopes that make up the cliffs in the districts of San Miguel, Magdalena del Mar and Miraflores (in high risk zones according to Cañari, 2001), obtaining the permanent seismic displacement and the spectral acceleration at the top of the slopes. The seismic response was evaluated by linear equivalent analysis based on three seismic scenarios. The design seismic motions with peak ground acceleration (PGA) of 0.20 g (base earthquake, OBE) and 0.40 g (maximum earthquake, MDE), and the seismic records used from the Lima earthquake (Mw8.1) of October 3, 1974 (recorded on Lima soil) scaled directly to the PGA of OBE and SMC and the Kobe earthquake (Mw6.9) of 1995 (recorded on Japanese soil) which had a PGA of 0.85 g considered as an extreme earthquake (MCE). As a result of the evaluation, displacements of 5 cm and 8 cm were obtained for the OBE and SMC earthquakes, while for an extreme earthquake the displacement obtained was 35 cm; likewise, the displacements obtained by the Newmark method were 36 cm, 74 cm and 155 cm for the OBE, MDE and MCE earthquakes, respectively

Raygada (2011) determined the physical stability of the slope and performed the dynamic analysis to determine the permanent displacements caused by the seismic movement in a sector of the Miraflores district, specifically in a shopping mall. The strength parameters of soil were those measured by Cañari (2001). The minimum FS stability criterion of 1.5 and 1.1 for the static and pseudostatic condition, respectively, while permanent seismic displacements less than 10 cm as low risk, for displacements between 10 cm to 100 cm as medium risk, and for displacements greater than 100 cm as high to very high risk. For the pseudostatic analysis, a seismic coefficient (k) of 0.22 was used, while for the two-dimensional dynamic analysis, the earthquakes of October 3, 1974 (Mw8.1) and August 15, 2007 (Mw8.0) were used, which were directly scaled to a maximum acceleration of 0.44 g; both synthetic records were considered as design earthquakes. The results for the high-risk sections generated displacements from 20 cm to 90 cm and from 60 cm to 383 cm for the 1974 and 2007 earthquakes, respectively.

It is believed that these displacement values obtained are acceptable from a numerical point of view, however, if we would like to consider them realistic for the conglomerate, it must be taken into account that these are rigid, so it is necessary to evaluate a range of admissible displacement values.

4 Assessment of Slope Stability

4.1 General Information

The slope stability was analyzed for 13 geotechnical profiles named P-01 to P-13 (Figure 5), which correspond to previous high, medium and low risk zones. In most of the profiles the slope has a height ranging from 50 m to 60 m and a slope steeper than 60°. The stratigraphy of these terrain profiles consists mainly of conglomerate, in some cases there is the presence of fine material (silty sands, silts and low plasticity clays) and the presence of fillings. Some profiles have buildings on the upper part, where it was assumed that each floor of the building transmits a uniform overburden of 10 kN/m per meter of width, for example, for section P-04 it was considered that the 6-story building on the slope has an overburden of 60 kN/m (Figure 6). The presence of the water table does not directly affect the slope stability analysis, so it was not considered.

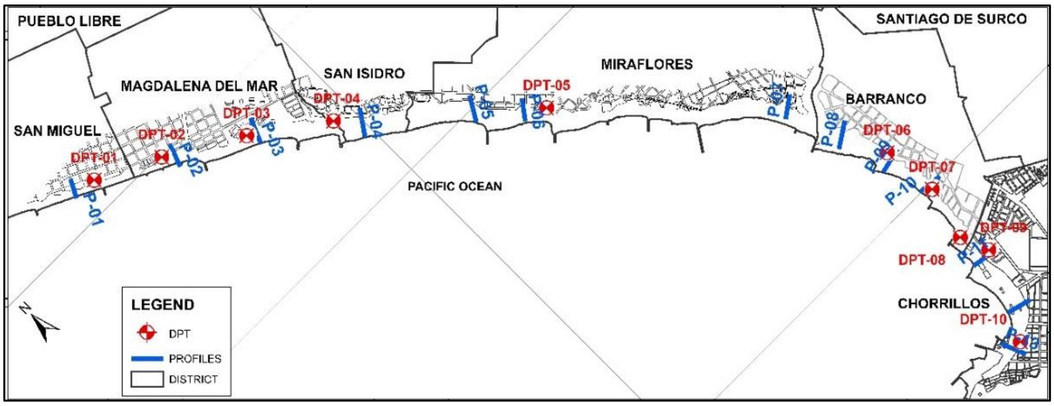

Figure 5. Locations of DPT and geotechnical profiles for slope stability analysis

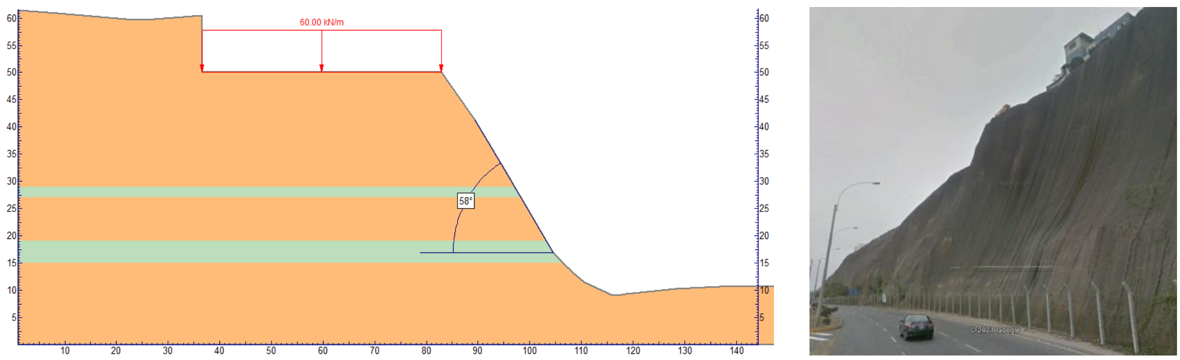

Figure 6. Cross-section of P-04 for slope stability analysis (left). Real view on site (right)

4.2 Seismic Coefficient (k)

Peruvian code CE.020 (2012) mentions that the seismic coefficient corresponds to a 475-year return period earthquake, the equivalent to a 10% probability of being exceeded in 50 years, considered as a design earthquake (MDE). Then, the criterion proposed by Hynes-Griffin and Franklin (1984) was used, which establishes that the seismic coefficient (k) is 0.5 times the maximum acceleration (PGA).

Peruvian code E030 (2018) establishes a Seismic Zone Map (Z) in the Peruvian territory that basically provides the maximum ground acceleration for a Rigid Soil with a 10 % probability of being exceeded in 50 years, considered as MDE. According to the map of seismic zones, a ground acceleration of 0.45 g can occur in the city of Lima. The obtained value of k was 0.225.

4.3 Geotechnical parameters

The physical parameters and strength such as density (γ), angle of drained friction (ϕ') and drained cohesion (C') of the conglomerate and fine soils used were those discussed in items 2.3 and 2.4. For the fills, the parameter ϕ' was obtained using the number of blows of the Chinese Dynamic Penetration Test (DPT). A further extension of this test used in Costa Verde Cliffs is described in Alva and Soto (2022), regarding parameter C' a value of 10 kPa was assumed to avoid obtaining FS values less than unity since the slopes of this fill are currently stable. Table 2 shows a summary of the geotechnical parameters of the soils for the slope stability analysis.

Table 2. *Strength parameters for Costa Verde Cliffs soils*

Type of soil	γ (kN/m^3)	ϕ' (°)	C' (kPa)
Conglomerate	21.5	40.0	55
Fines (silty sand)	19.0	28.5	90
Filling	18.0	30.0	10

4.4 Slope Stability Results

Static and pseudostatic conditions were used for the analysis of Slope Stability. Limit Equilibrium Analysis was used applying the methodology of Spencer through non-circular surface failure surfaces. For each profile, the top quarter (h/4), the top half (h/2), the top three quarters (3h/4) and the entire slope (h) were analyzed, to then estimate the level of displacement that can occur in pseudostatic condition. Tables 3 and 4 show the factors of safety (FS) obtained in static and pseudo-static conditions, respectively.

In static conditions it is observed that almost all the profiles present a FS greater than 1.5, so they are statically satisfactory, however, profiles P-02, P-03 and P-04 present a FS greater than 1.0 and less than 1.5, so they are not necessarily unstable and are also in equilibrium, being consistent with what was observed at the site.

Table 3. *Factor of safety obtained from static condition analysis*

Profile	Factor of Safety (FS)				District
	¼ h	½ h	¾ h	h	
P-01	3.26	2.49	1.92	1.71	San Miguel
P-02	2.11	1.67	**1.39**	**1.34**	Magdalena del Mar
P-03	1.84	1.45	**1.21**	**1.04**	
P-04	1.57	**1.31**	**1.19**	**1.10**	San Isidro
P-05	4.16	2.63	2.19	1.86	Miraflores
P-06	1.55	1.25	1.15	1.06	
P-07	3.54	2.62	2.13	1.87	
P-08	5.41	2.78	1.99	1.73	Barranco
P-09	3.87	2.35	1.75	1.53	
P-10	4.25	2.43	2.00	1.69	
P-11	4.03	2.71	2.14	1.91	Chorrillos
P-12	1.56	**1.40**	1.88	2.42	
P-13	2.66	2.12	1.80	1.68	

In pseudostatic conditions, it is observed that several profiles present a FS greater than 1.25 (profiles P-01, P-05, P-07, P-08, P-09, P-10, P-11, P-13), so they are considered stable during an earthquake of great intensity, on the contrary, these results differ from the results of the very high and high risk zones shown by Huaman (2000) and Cañari (2001), this is possibly because the topography of geotechnical profiles differs from the actual topography. On the other hand, profile P-02 has a FS slightly higher than 1.0 so it can be considered that it still maintains its equilibrium, while profiles P-03, P-04 (they were considered as an embankment without stabilization elements) and profile P-06 present a FS lower than 1.0 so the whole slope will fail, and in the case of profile P-12 the upper half of the slope presents a FS lower than 1.0 so this part of the slope will fail.

Table 4. *Factor of safety obtained from pseudostatic condition analysis*

Profile	Factor of Safety (FS)				District
	¼ h	½ h	¾ h	h	
P-01	2.36	1.83	1.42	1.27	San Miguel
P-02	1.68	1.30	**1.07**	**1.02**	Magdalena del Mar
P-03	1.18	**0.96**	**0.82**	**0.70**	
P-04	**1.06**	**0.89**	**0.81**	**0.74**	San Isidro
P-05	2.64	1.78	1.50	1.29	Miraflores
P-06	**1.02**	**0.84**	**0.77**	**0.71**	
P-07	2.29	1.76	1.45	1.28	
P-08	3.16	1.79	1.42	1.23	Barranco
P-09	2.50	1.63	1.25	1.12	
P-10	2.55	1.64	1.39	1.18	
P-11	2.67	1.90	1.51	1.37	Chorrillos
P-12	**1.04**	**0.93**	1.20	1.56	
P-13	1.82	1.52	1.31	1.22	

5 ESTIMATION OF PERMANENT SEISMIC DISPLACEMENTS

The permanent seismic displacements were determined using the simplified methods of Newmark (1965), Makdisi and Seed (1978) and Bray et al. (2018). The Newmark method aims at predicting the accumulation of displacement of the slope during a series of acceleration cycles (as in an earthquake) we suggest the reader review the original reference for further details. Makdisi and Seed (1978) method performed a parametric analysis using real and hypothetical dams and embankments, this method use the simplified Newmark's method and presented it in the form of a chart, details about use of this, we suggest to review the original reference. Bray et al. proposed a simplified method to estimate the seismic displacement of slopes for earth structures (dams, embankments, among others) and slopes that are subject to subduction earthquakes in interface zones, it take in account the nonlinear model of soil, seismic coefficient, and period of vibrations of slope, more details about method we suggest the reader review the original reference for further details.

In order to estimate the seismic displacement, some necessary parameters required by each method were used as follows: for Newmark's method, the lowest value of "ky" of each profile was considered, and the acceleration record of the earthquake of October 03, 1974 was used, this was modified and adjusted spectrally to a design spectrum with a return period of 475 years; for Makdisi and Seed's method, the acceleration record mentioned above was used and was scaled directly to the maximum acceleration of 0.45 g, the nonlinear behavior of the conglomerate was assumed as the modulus and damping reduction curves for gravels proposed by Seed et al (1986), and the earthquake magnitude M8.25 as the maximum allowable value; for the Bray et al. method, a design spectrum mentioned above was used and the maximum magnitude of the earthquake was Mw 8.8.

The permanent seismic displacements of the profiles are shown in Figure 7, which shows the profile number and the estimated seismic displacement of the upper quarter of the slope (1/4h), the upper half (1/2h), the upper three quarters (3/4h) and the entire slope (h) obtained by the methods of Newmark (N), Makdisi and Seed (MS) and Bray et al (BT).

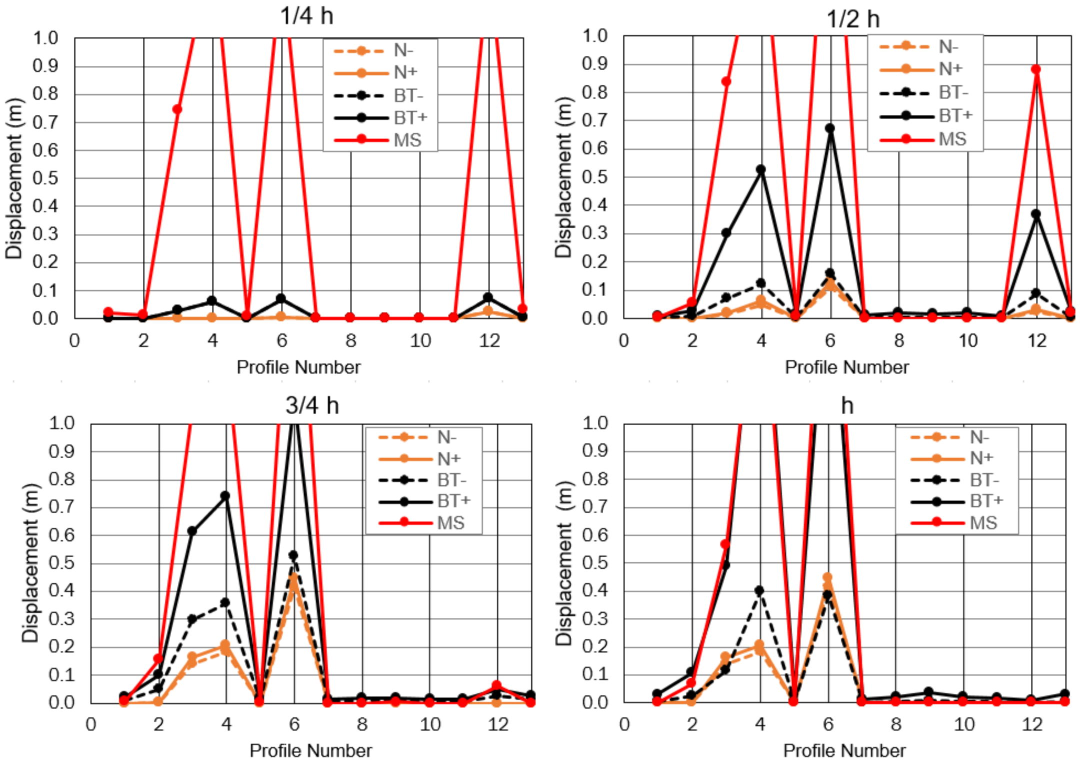

Figure 7. *The estimated seismic displacement for each profile using Newmark (N), Makdisi and Seed (MS) and Bray et al. (BT)*

The results obtained by the method of Makdisi and Seed (MS) provide higher values in all cases since they are obtained considering that the slope is similar to that of an earth dam; the results by the method of Newmark (N) in the negative sense (N-) and in the positive sense as (N+) give lower values; and the results by the method of Bray et al. (BT) give intermediate values with respect to the method (N) and (MS), the results being a range of values between mean minus one standard deviation (BT-) and mean plus one standard deviation (BT+).

5.1 Discussion of results

The N method underestimates the displacements while the MS method overestimates the displacement values, the latter because it was developed for free-boundary structures such as earth dams, and the BT method provides reasonable values of permanent seismic displacement. To establish the limit of displacements of rigid soils, previous researches such as Makdisi and Seed (1978), Hynes-Griffin and Franklin (1984), Kavazanjian et al. (1997) mentioned that earth structures subjected to severe earthquakes support allowable permanent displacements of up to 1.0 m, while Bray et al. (1998) established an allowable displacement of 0.15 m to 0.30 m for municipal clearing structures. Considering the nature of the Lima conglomerate (a rigid material), a permanent seismic displacement of 0.3 m is estimated. A larger displacement will be considered as a massive landslide.

The results of the permanent displacements for the profiles are described below:

- For profiles of the upper quarter of the slope (1/4h), it is observed that the seismic displacements do not generate a slip of the upper quarter of the slope, since they do not exceed 0.3 m.

- For the upper half of the slope (1/2h), profiles P-03, P-04, P-06 and P-12 (corresponding to backfill areas) it is estimated that seismic displacements of more than 0.3 m can be obtained, so that this material can slide and cause damage to any element on the slope, interrupting traffic on the road at the foot of the slope.

- For the upper three quarters of the slope (3/4h) and the entire slope (h), of profiles P-03, P-04 and P-06 (corresponding to fill zones), it is estimated that the seismic displacement may vary from 0.3 m to 1.5 m, so that a landslide of the entire slope may occur, which will interrupt road traffic. In the case of profile P-12, below this part of the slope, there is natural soil, which is more resistant than the fill.

A comparison between the FS results under pseudostatic conditions and the seismic displacements, allows identifying if the FS value of 1.0 can be considered as appropriate to determine to generate a maximum displacement of 0.3 m. This could be observed for profiles P-04, P-06 and P12 with a height of ¼ h obtained a FS varying from 1.02 to 1.06 with a displacement less than 0.1 m, the same profiles for heights from ½ h to h, obtained a FS from 0.71 to 0.96 with a displacement greater than 0.3 m these values that were to be expected due to the reasonable criterion of FS less than unity. While the profiles presenting a FS greater than 1.1, the permanent displacements obtained are less than 5 cm. Therefore, for slopes formed by conglomerate that obtain a pseudostatic FS greater than or equal to 1.1 possibly show permanent seismic displacements less than 0.3 m.

On the other hand, the estimated displacements are larger than the displacements obtained by Macazana (2006), but are smaller than the displacements obtained by Raygada (2011). This shows that the analysis performed by Macazana has underestimated the displacements, indicating that no sliding will occur, while analysis of Raygada has overestimated the deformations, inferring that landslide will occur.

5.2 Mitigation of landslide risk

To reduce the risk of landslides there are several methods such as surface erosion control, retaining walls, anchors, inter-ramp slope, among others, each of these methods has its advantages and disadvantages from the technical and economic point of view. Currently, the surface of the Costa Verde cliffs have an erosion control mesh to contain rockfall; however, this measure is not sufficient to reduce landslides or increase slope stability.

To improve the stability of the slopes of the Costa Verde cliffs, it is recommended that the terrain be cut into inter-ramp slope(in the sections or areas where they are required) and an embankment be built at the foot of the slope. The inter-ramp slope have a double function: to improve the overall stability of the slope and to reduce the effects of rockfall, in order to avoid accidents during its useful life and during the occurrence of a severe earthquake. The wedge-shaped embankment reduces the trajectory and energy potential of rockfall. In addition, this system has the advantage of maintenance over time (Alva & Soto, 2022).

6 CONCLUSIONS

The estimation of the permanent seismic displacement is important to guarantee the physical stability of the slopes when performing the pseudostatic analysis. Some profiles of the Costa Verde Cliffs showed that the pseudostatic FS varied between 1.02 and 1.10, obtaining permanent displacements less than 0.1 m, while those with FS less than 1.0 obtained displacements greater than 0.3 m. Therefore, the estimation of displacements by simplified methods should be reasonably applied to other slopes conformed by other types of soil to establish an allowable seismic displacement.

REFERENCES

Ayquipa, C. (1995). Microzonificación Sísmica de Chorrillos y Barranco *[Seismic microzoning of Chorrillos and Barranco districts]* (Degree Dissertation). National University of Engineering, Lima, Peru.

Alva, J.E., & Soto, J. (2022). *Análisis de Estabilidad ed la Costa Verde, Lima, Peru* [*Slope Stability Analysis of the Costa Verde, Lima, Peru]*. Proceeding XXVII Reunión Nacional de Profesores de Ingeniería Geotécnica, Cuarta Conferencia Eulalio Juárez Badillo. Mexico.

Bray, J., Macedo, J., & Travasarou, T. (2018). Simplified Procedure for Estimating Seismic Slope Displacements for Subduction Zones. J. Geotech. Geoenviron. Eng., 2018, 144(3): 04017124.

Cañari, M. (2001). A*nálisis de la Estabilidad de Taludes de la Costa Verde [Slope Stability Analysis of the Costa Verde]* (Degree Dissertation). National University of Engineering, Peru.

Carrillo, A. (1984a). *Estabilidad Estática Dinámica de los Acantilados de Lima [Dynamic and Static Stability of the Cliffs of Lima]*. Proceeding 5th National Congress of Civil Engineering. Lima,Perú.

Carrillo, A. (1984b*). Recientes Experiencias en el Análisis de Estabilidad de los Acantilados en la Costa Verde [Recent Experiences in the Stability Analysis in Costa Verde cliff]*. Journal of Geotechnical Engineering. National University of Engineering, Peru.

CE020 (2012). *Estabilización de Suelos y Taludes [Slope and Soil Stabilization]*. Norma Técnica Peruana

CISMID (2015a). *Microzonificación del distrito de Barranco - Apéndice C Caracterización Geotécnica del Suelo [Microzonation of the Barranco district - Appendix C Geotechnical Characterization].* Technical Report.

CISMID (2015b). Microzonificación del distrito de San Miguel - Apéndice C Caracterización Geotécnica del Suelo *[Microzonation of the San Miguel district - Appendix C Geotechnical Characterization].* Technical Report.

CISMID (2017). Microzonificación de los Distrito de Magdalena del Mar - Apéndice C Caracterización Geotécnica del Suelo *[Microzonation of the Magdalena del Mar district - Appendix C Geotechnical Characterization].*Technical Report

Díaz, J. (2008). *Estabilización del Talud de la Costa Verde en la Zona del Distrito de Magdalena [Stabilization of the Costa Verde cliffs in the Magdalena District].* (Degree Dissertation). Pontificia Universidad Católica del Peru.

García, E. (1982). *Análisis de Estabilidad Estática y Dinámica de los Acantilados de Lima [Static and Dynamic Stability Analysis of Lima Cliffs].* (Degree Dissertation). National University of Engineering, Peru.

Granados, A. (2006). *Estabilización del Talud de la Costa Verde en la Zona del Distrito de Barranco [Stabilization of the Costa Verde cliffs in the Barranco District]*. (Degree Dissertation). Pontificia Universidad Católica del Peru.

Hynes-Griffin, M. E., & Franklin, A.G. (1984). Rationalizing the Seismic Coefficient Method. Final Report, Miscellaneous Paper GL-84-13, Department of the Army, U.S. Army Corps of Engineers

Huamán, M. (2000). *Características del Acantilado de Barranco [Characteristics of cliffs of Barranco district]*. (Degree Dissertation). National University of Engineering, Peru.

JAHI (2009). *Ensayo de Corte Directo In-Situ para Hotel Larcomar, Miraflores [Insitu direct shear test at Larcomar Hotel]*. Technical report.

Kavazanjian E. Jr., Matasovic N., Hadj-Hamou T., & Sabatini P. J. (1997). Design Guidance: Geotechnical Earthquake Engineering for Highways, Vol. 1, Design Principles, Geotechnical Engineering Circular 3, Publication FHWA-SA-97-076, Federal Highways Administration, U.S. Department of Transportation, Washington

Macazana, R. (2006). *Análisis dinámico de los acantilados de Lima [Dynamic Analysis of the cliffs of Lima].* (Master Dissertation). National University of Engineering, Peru.

Makdisi, F.I. & Seed HB. (1978). Simplified procedure for estimating dam and embankment earthquake-induced deformations. Journal of Geotechnical Engineering, 104(7), 849–867

Newmark, N. (1965). Effects of earthquakes on dams and embankments. Geotechnique, 15(2), 139–160

Pacheco, A. (2006). *Estabilización del Talud de la Costa Verde en la Zona del Distrito de San Isidro [Stabilization of the Costa Verde cliffs in the San Isidro District]*. (Degree Dissertation). Pontificia Universidad Católica del Peru.

Raygada, L. (2011). *Análisis de Estabilidad de Taludes y Desplazamientos en el acantilado de la Costa Verde [Slope stability analysis and Displacement of the Costa Verde cliffs]*. (Degree Dissertation). National University of Engineering, Peru.

Sánchez, L. (1975). *Análisis y Diseño en la Estabilidad de Taludes [Análisis y Diseño en la Estabilidad de Taludes]*. (Degree Dissertation). National University of Engineering, Peru.

Sanchez, S., Rodriguez, J.M., López, J.D., Laina, C., & Jiménez, A. (2016). Caracterización de Suelos Granulares Gruesos. El caso de la Grava de Lima [Characterization of Coarse Granular Soils, the case of Lima Gravel]. Proceeding of the 10th National Symposium on Geotechnical Engineering. Spanish Society of Soil Mechanics and Geotechnical Engineering.

Silva, R. (2003). *Análise da estabilidade dinâmica de taludes de solo [Análisis de Estabilidad Dinámico de taludes de Suelo]*. (Master Dissertation). Pontificia Universidad Católica de Rio de Janeiro, Brasil.

Valencia, R. (1940). *El terremoto del 24 de mayo de 1940, sus efectos y enseñanzas [The May 24, 1940 earthquake, its effects ans learning]*. Journal of Engineering, Universidad Católica del Peru.