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1-D seismic response analysis for seismic permanent displacements estimation on a heap leach pad

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Abstract

The rapid growth of mining operations in Peru has brought many new challenges to the already difficult tasks of designing, building, and operating heap leach pads in the aggressive Andes terrain. Consultants usually deal with delicate issues such as specific construction techniques, complex terrain geometry, limited suitable locations, and – what may be the most important challenge – high seismic activity.

This paper presents a case study of a heap leach pad designed on top of an existing mine waste dump, which itself was built over a heterogeneous and complex deposit of alluvial and residual soft clayey soils. Among other geotechnical issues, it was necessary to assess the long-term stability of both structures. Given the geological and geometrical design complexity and using a large set of geotechnical investigations and laboratory tests, in-depth one-dimensional, non-linear seismic site response analysis was carried out. The resulting seismic response spectrums at the base of the mine waste dump and heap leach pad were used in the determination of seismic coefficients and in the calculation of seismic induced permanent displacements for rotational, compound, and translational failure mechanisms. This evaluation allowed a detailed determination of both seismic coefficients and seismic induced permanent displacements for different foundation conditions, and highlighted its importance. Furthermore, the need to explore the dynamic properties of both mine waste and leached ore was determined.

Introduction

Since the beginning of the 21st century, the mining industry in South America, particularly in Peru and Chile, has experienced rapid growth. Most of the mines in Peru are located in the Andean region, above 2,500 m.a.s.l., where the geological conditions allow mining companies to extract gold, copper and other minerals, making the Peruvian mining industry one of the top gold, silver and copper producers in the world.

Recent development of mining operations in Peru has been closely related to the use of the heap leaching techniques. Since 1998, when the Pierina heap leach project was constructed, major heap leach pads (HLP) have been designed, constructed and operated in the Peruvian Andean region. However, in the last five years several issues have arisen that have complicated the civil and geotechnical design, as well as the construction work. Special construction techniques and criteria for the use of geomembranes, geosynthetics and earthworks have been addressed by César et al. (2013), Garay et al. (2014) and César et al. (2014). Limitations of the two-dimensional (2-D) approach and advantages of three-dimensional (3-D) slope stability analysis due to the aggressive Andes terrain have been introduced by Reyes et al. (2014). However, issues such as the need to design HLP on poor foundation areas given the lack of adequate terrain or even on top of other earth structures have not been extensively analyzed. Moreover, the increasing need for larger and higher HLP have urged geotechnical designers to evaluate the resistance of both leach ore and liner material interface for confining pressures from 1 to 3 MPa, thus highlighting the need for larger and more powerful laboratory devices.

Yet, given the high seismic activity in Peru, which is generated by subduction of the Nazca plate beneath the South American plate, the assessment of the seismic stability is probably the most important geotechnical issue for HLP design. Most of the time it determines its final configuration and implementation of stabilization measures such as dykes, buttresses, precambering, among others. Usually, the pseudo-static approach is employed to assess the seismic stability of earth structures; however simplified methods such as the Bray and Travasarou’s (2007) are being used more often to estimate seismic induced permanent displacements (SIPD) rather than a factor of safety, which has proven to be a more rational approach to this issue. HLP and their liner system are considered particularly sensitive to SIPD. These liner systems are usually built using compacted low permeability soils and single textured geomembrane. SIPD of 15 to 30 cm (Kavazanjian et al., 2011) can tear the geomembrane and cause both environmental and economic damage. In consequence, the need for a precise assessment of the seismic demand and its correct use in the seismic design of HLP is highlighted.

This paper presents the seismic design of a case study of a HLP, which – given the lack of an adequate location – was designed on top of a mine waste dump (MWD), which itself was built over a heterogeneous and complex deposit of alluvial and residual soils. In order to capture the seismic, geological and geometrical complexity of the foundation, over 20 soil columns, consisting of several clayey, sandy and gravelly soils of alluvial and residual origins as well as mine waste, were modeled and used in one-dimensional (1-D) seismic site response analyses. Then, the response spectrums at the base of the MWD and HLP were used for their seismic design.

Theoretical background

Kramer (1996) suggested two approaches to deal with seismic stability analysis: inertial stability and weakening stability analysis. The first one is used for heap leach pads, since leached ore liquefaction is not expected; however, there have been reported cases of this phenomenon (Breitenbach and Thiel, 2005 and Castillo et al., 2005), mainly because of high fines content and low permeability of crushed ore. Inertial stability deals with displacements produced by temporary exceedances of the soil strength by dynamic stresses, assuming that this shear strength remains relatively constant. A factor of safety calculation by a pseudo-static analysis and/or SIPD calculations are ways to deal with inertial instability.

As pointed out before, the calculation of SIPD for HLP is a rational approach to assess its seismic stability. Peruvian practitioners and recent research such as Ayala et al. (2014) and Reyes and Pérez (2015) suggest the use of the Bray and Travasarou (2007) method to estimate SIPD. Equivalently, the Bray and Travasarou (2009) method can be used to rationally select a seismic coefficient, related to a specific maximum SIPD, to use in a pseudo-static analysis. Both of these methods use the same theoretical basis. Finally, the most important input for these approaches is a response spectrum related to the foundation conditions of the sliding mass. Hence, many times seismic response analyses are performed to capture seismic site conditions related to the failure mechanism to be evaluated. The following sections describe the theoretical background of these methods.

1-D and non-linear seismic site response analysis

The surface seismic response of an earthquake is greatly influenced by site soil conditions. In order to quantify this, seismic response analyses are used to determine the dynamic soil behavior due to the shake of the rock immediately beneath it (Kramer, 1996). To quantify the seismic response of a rock, seismic hazard studies are performed. Dynamic behavior of rock is less influenced by the earthquake nature due to its large stiffness. 1-D seismic response analyses are based on the hypothesis that all the soil boundaries are horizontal and that soil response is particularly affected by seismic shear waves, whose propagation turns vertical as it approaches the surface.

The analysis methodology depends on how the soil behavior is modelled. A linear method (LM) analysis relies on the use of transfer functions in the frequency domain. However, the nonlinear behavior of soils, which contrasts with the linear assumption of the LM approach, makes this methodology quiet restricted. In order to account for such restrictions, a simple iterative process involving dynamic equivalent linear properties of soil can be used; this methodology is called the equivalent linear method (ELM). As mentioned, this methodology is still linear up to some extent since it focuses on searching the elastic parameters of the soil. These parameters should be consistent with seismic induced shear strain levels for each soil layer involved in the analysis.

A fully nonlinear analysis (NLM) is capable of modelling the hysteretic behavior of soils due to earthquake loading. It uses a direct numerical integration in the time domain. Through this analysis, a linear or nonlinear stress-strain relationship can be followed by a number of small incremental linear steps. Such relationship is generally modelled by a hyperbolic model.

The load, unload and reload conditions, generally known as the extended 4 Masing (1926) rules, of the soil under cyclic loading was observed and proved by Matasovic (1993b) using the DMOD (Matasovic, 1993a) software. Currently, Hashash et al. (2010) has greatly improved the deficiencies encountered when using the NLM approach (Stewart et al., 2008) by the development of the DeepSoil software (Hashash, 2014).

Bray and Travasarou (2007, 2009)

Bray and Travasarou (2007) presented a simplified coupled semi-empirical predictive model to estimate the SIPD based on the Newmark (1965) rigid-block method and numerical analysis, as a way to update the method developed by Makdisi and Seed (1978). This procedure involves a block failure model sliding over a nonlinear coupled surface (Rathje and Bray, 2000), which can represent the dynamic behavior of structures such as: dams, natural slopes, compacted fill dykes and municipal solid waste fills (MSWF).

Bray and Travasarou (2007) noted that the major uncertainty for the evaluation of an earth structure is the possibility of a seismic event. To overcome this issue, they took advantage of over 688 earthquake records and concluded that the spectral acceleration at a degraded period of the potential sliding mass is the most efficient and sufficient single ground motion intensity measure. The method captures the slope seismic resistance through its k_y and initial fundamental period. Using these parameters as input, Bray and Travasarou (2007) presented formulations to estimate SIPD and to evaluate the probability of negligible SIPD. Finally, they showed that their estimates were generally consistent with 16 documented cases of earth dams and MSWF.

Bray and Travasarou (2009) presented a procedure whereby selecting project-specific allowable level of SIPD, estimating the fundamental period of the sliding mass, site-dependent seismic demand (expressed in terms of spectral acceleration) and based in the original Bray and Travasarou (2007) approach, a rational seismic coefficient can be calculated. This procedure represents a more rational basis for selecting seismic coefficients when compared to the suggestions of Hynes-Griffin and Franklin (1984) who recommended, among other things, the use of half of the peak ground acceleration (PGA) at the site based on their assumption that 1 m of SIPD is acceptable for most earth dams, which were the structures they based their investigation on. In consequence, the Hynes-Griffin and Franklin (1984) approach should not be used for structures with lower values of maximum SIPD, such as HLP.

Case study geotechnical overview and seismic analysis

The case study presented is a HLP design project, located in northern Peru, developed for one of the most important gold mines in the region. A new HLP with a capacity of 40 Mm^3 and maximum height of over 100 m was needed in the short term for the future development of the mine. However, limited suitable locations able to satisfy this capacity forced the designers to place it on top of an existing MWD. This facility was also in design stages: phase 1 was under its last stage of construction and phase 2, with a maximum height of 140 m, was still in design. Previous geotechnical investigation showed that the foundation of phase 2 was composed of large and heterogeneous deposits of alluvial and residuals soils that is over 80 m deep. Clayey, silty, sandy and gravelly soils were distributed all over its area;

consequently, settlements due to consolidation of the clayey soils, settlement caused by the deformation of mine waste and seismic site particularities due to all these materials were expected.

The short-term stability condition of both structures, which included undrained resistance of fine grained soils and settlement calculations due to consolidation of clayey soils and deformation of mine waste, is dealt with by Reyes and van Zyl (2015). The long-term stability conditions included the assessment of the static and seismic stability for operational and closure conditions, which were related to seismic events of 100 and 475 years return period, respectively. These analyses focused on compound and rotational failure mechanisms that crosses both leached ore of the HLP, mine waste of the MWD and foundation soils, and translational failures on the HLP that crossed its liner system, which itself was placed directly on top of the MWD.

Preliminary long-term stability analysis showed that the pseudo-static conditions were much more critical that the static ones. Uniform hazard response spectrums available were calculated for soil type B according to the 2012 version of the International Building Code. With soil strata of over 80 m deep, it classified heterogeneously as types C and D. Hence, it was clear that seismic response analyses were needed to determine response spectrums both on surface and on top of the MWD to use on the seismic design. The first ones would be used to define seismic stability for failure mechanisms that cross foundation soils, mine waste and leached ore and the second ones for translational failure mechanism that cross the HLP liner system. Thus, complementary site investigations as well as laboratory tests were carried out to define in detail the geotechnical properties required for the seismic response analysis, calculation of seismic coefficient and estimation of SIPD.

Geotechnical site investigations

A large complementary geotechnical site investigation program was carried out in order to properly determine the spatial variability and geotechnical properties of the foundation soils. Previous site investigations and stratigraphy were described by Reyes and Parra (2014). A total of 210 test pits, 41 dynamic penetrations tests, 115 standard penetrations tests, 65 large penetrations tests and 26 geotechnical boreholes were included within an area of over 1000 Ha. Laboratory tests included a total of 6 undrained (UU), 14 consolidated undrained (CU) and 7 consolidated drained (CD) triaxial tests, 4 resonant column and torsional shear (RCTS) tests, as well as one-dimensional consolidation and permeability tests on most soils. 775 m of seismic refraction tests, 20 arrays of multichannel analysis of surface waves (MASW), 6 arrays of 2-D MASW and 15 microtremor array measurements (MAM) were also performed. All of this information allowed the authors to describe and discretize in detail all foundation soils, mine waste and leached ore in detail.

Foundation soil stratigraphy and properties for phase 1 of the MWD were described by Reyes and Parra (2014). However, foundation soils of the phase 2 of the MWD proved to be softer and/or looser and more heterogeneous and required more and longer geological-geotechnical cross-sections and additional laboratory tests to properly describe them. For the particular case of residual and alluvial clayey soils, laboratory tests were performed on undisturbed samples obtained from large test pits and boreholes. Since the analyses on this paper do not directly include all geotechnical properties of most of these materials, only field and dynamic laboratory tests on some materials are presented. A detailed review of the other properties is presented by Reyes and van Zyl (2015).

The particle size distribution (PSD) of mine waste and leached ore was determined in the field by several excavations in both the existing MWD and HLP, respectively, determining maximum sizes of over 12 inches for both materials. In each excavation, almost 1 ton of mine waste or leached ore was evaluated, first weighting the whole sample and then separating particles with sizes smaller than 3 inches. Using large sieves, as presented on Figure 1, the PSD of all material larger than 3 inches was determined in situ. Complementary laboratory tests on the smaller size particles completed the global PSD curve. Figure 2 shows the average global PSD of both mine waste and leached ore.

Due to the large size of mine waste and run-of-mine (ROM) leached ore particles, two parallel gradation curves to the originals PSD curves were built for each material, which are shown in Figure 2, using a maximum particle size of 3/4 inches so that no scalping would be needed when tested on the RCTS device built in the University of Texas at Austin. The technique used to develop the curves was first developed by Lowe (1964), and then extensively used by Marachi et al. (1969), Thiers and Donovan (1981) and Varadarajan et al. (2003) to perform drained triaxial tests on rockfill, crushed rock and alluvial soils. In the last decade, many researchers, particularly Gesche (2002), De La Hoz (2007), Dorador (2010) and Ovalle et al. (2014), and practitioners such as Linero et al. (2007) and Palma et al. (2009) have used this technique to test alluvial and waste rock materials. Based upon this background, the tests performed for this case study test would verify the shear resistance properties defined for phase 1 and provide representative stress-strain and dynamic curves for geotechnical models.

Figure 1: In situ determination of the global particle size distribution of coarse granular materials

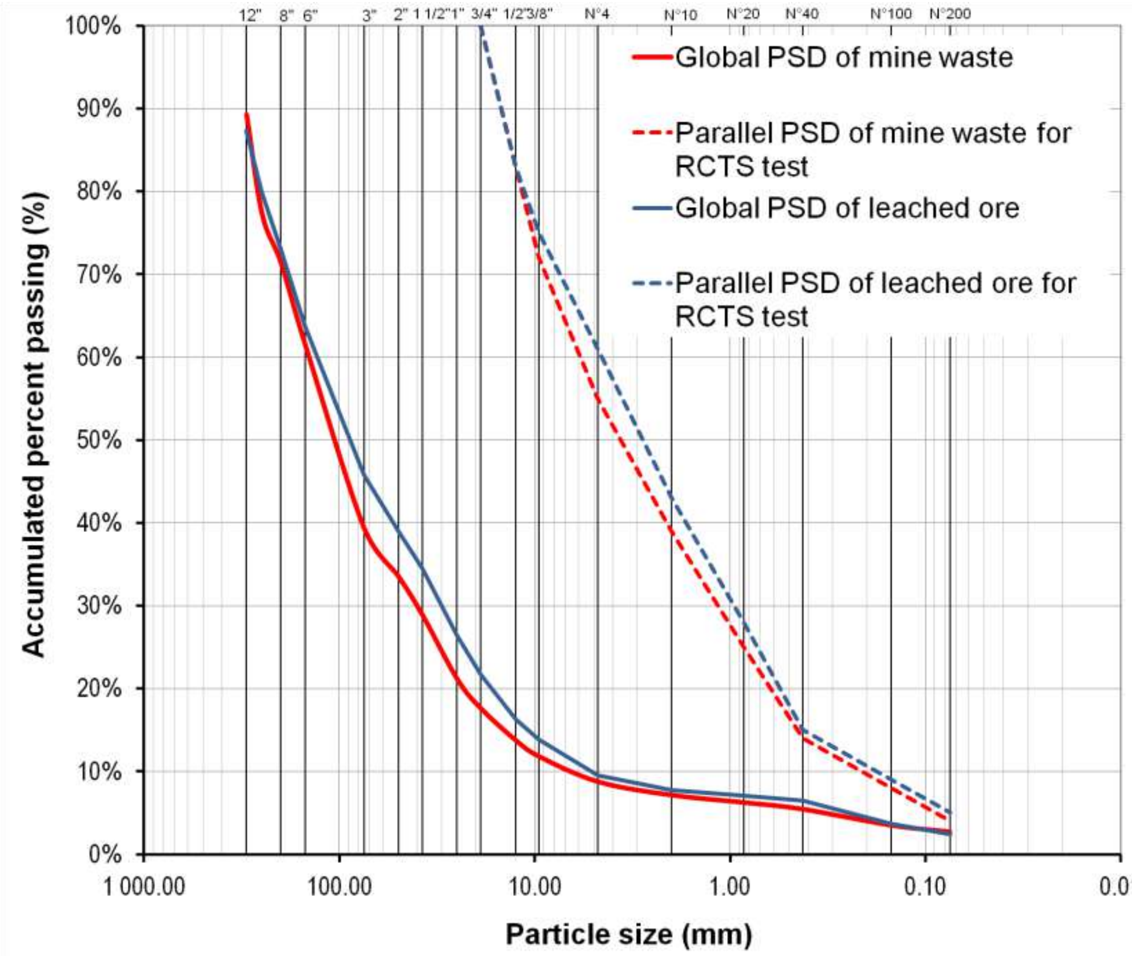


Figure 2: Global and parallel particle size distributions of the mine waste and leached ore

The RCTS device is capable of performing on the same soil specimen both the torsional resonant column test at high loading frequencies and in the nonlinear range, and the cyclic torsional shear test at much lower frequencies (Liao et al., 2013). This device has been used by Darendeli (2001) and Menq (2003) to test different kinds of fine grained, sandy and gravelly soils, producing shear modulus and damping ratio curves for these materials. Furthermore, Liao et al. (2013) performed several tests in the RCTS device using scalped samples of crushed gravel produced in a rock quarry. However, no previous published work is available on dynamic properties of leached ore and mine waste modeled by the use of the parallel gradation technique on dynamic testing, thus making these test results the first ones to be published.

Geotechnical properties

Table 1 shows all materials involved in the static and seismic analysis of the HLP and their geotechnical properties. Since the seismic design of the HLP is the focus of this paper, only the dynamic properties are presented.

Table 1: Geotechnical properties for the seismic response analysis

Materials		Total unit weight (kN/m³)	Saturated unit weight (kN/m³)	Shear wave velocities (m/s)	Dynamic properties curves
ROM leached ore		20	21	200 – 550	RCTS test results and modified Menq (2003)
Low permeability soil – geomembrane interface		20	21	Yegian et al. (1998)	Based upon Kavazanjian and Matasovic (1995), Yegian et al. (1998) and Arab (2011)
Mine waste		20	21	200 – 600	RCTS test results and modified Menq (2003)
Alluvial soils	Gravelly	19	20	170 – 465	Menq (2003)
	Sandy	18	19	280 – 310	Darendeli (2001)
	Silty and clayey	18	20	170 – 255	Darendeli (2001)
Intrusive residual soils	Gravelly	19	20	340 – 600	Menq (2003)
	Sandy	18	19	280 – 750	Darendeli (2001)
	Silty and clayey	17	19	200 – 750	Darendeli (2001)
Sandstone residual soil		19	20	200 – 580	Darendeli (2001)
Bedrock		22	23	750 – 1000	Idriss (1991)

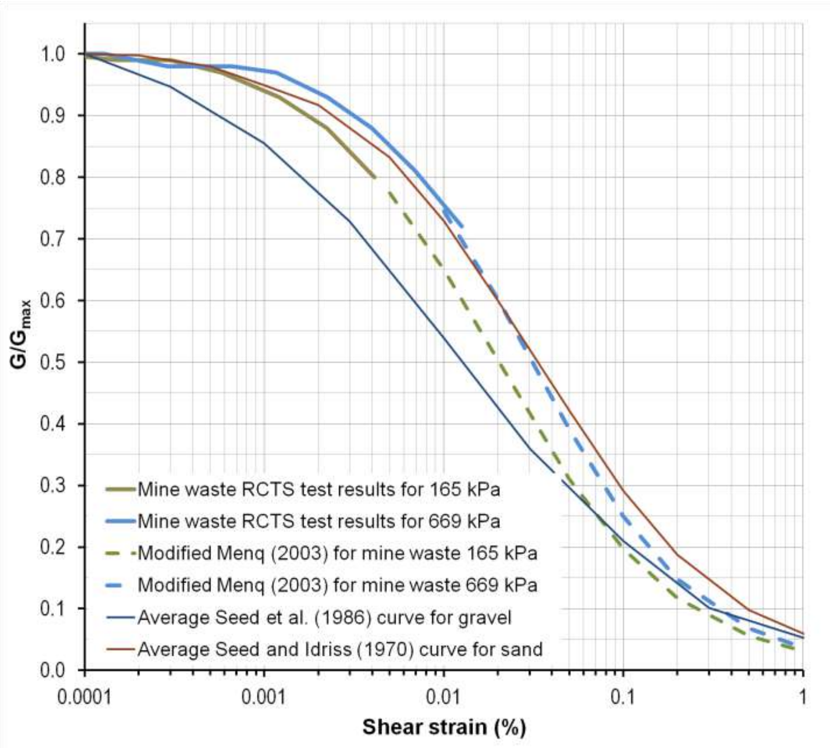
Shear wave velocities for all the materials were obtained from the geophysical surveys carried out on the foundation of phase 2, on top on the mine waste of the existing phase 1 and on top of the leached ore of the existing HLP of the mine, which is located a different area. The resulting shear wave velocities for the foundation soils were very heterogeneous and are presented in Table 1. The shear wave profiles of the existing mine waste and leached ore were compared to the shear wave velocities results of the

RCTS tests, which provided a logarithmic relationships between the shear wave velocity and the mean effective stress; both approaches showed a close fit. Consequently, the definitions of the shear wave velocity profiles of these two materials in the analysis were made using their respective logarithmic relationships.

The RCTS tests also provided two sets of shear modulus and damping ratio curves for representative confining pressures for both mine waste and leached ore, which are shown in Figures 3 and 4. These curves were compared with the average sand curve from Seed and Idriss (1970), average gravel curve from Seed et al. (1986) and the curves predicted by Menq (2003) (using different values for the coefficients for both materials), as done by Liao et al. (2013) and shown in Figures 3 and 4 for the mine waste and leached ore, respectively.

The comparison showed, in general a close fit of these results with the Menq (2003) curves, confirming the change in the nonlinear behavior when affected by confining pressure. For the case of the shear modulus curve, a close fit is achieved when using the real uniformity coefficient (C_u) on the Menq (2003) formulation for both materials. For the case of the damping ratio curves, a fair fit is for the leached ore, showing a particular deviation for the minimum damping ratio. However, for the mine waste damping ratio curve, a clear deviation is shown. The results for the ROM leached ore of this project contrast with the ones found by Reyes and Perez (2015) on his crushed leached ore curves comparison with literature curves including the Menq (2003) formulation, where they showed a clear deviation of these results from the other ones, although it also confirmed the change in the nonlinear behavior when affected by confining pressure.

Menq (2003) sand and gravel relationships effectively show the change in nonlinear behavior of coarse grained soil due to changes in the uniformity coefficient (C_u) value and confining pressure. However, changes in particle angularity, surface texture and exotic material fabrics (such as the one of the leached ore) were not evaluated by Menq (2003). The modified hyperbolic equations used by Menq (2003) can be used to calibrate the leached ore response in the nonlinear strain range and to clearly model its behavior as a function of confining pressure. The equation to calibrate the model should include increased angularity, surface texture, and fabrics of the leached ore and/or mine waste, if needed. More test data from RCTS and cyclic triaxial tests would be needed to have confidence in this material-specific model (Stokoe, 2014). However, only for this particular project, a modification of the original Menq (2003) formulation was made to fit only damping ratio results of the mine waste by changing the parameters of the hyperbolic equations. No change was made for the leached ore. For all the other foundation soils, the dynamic curves resulting for the original formulation of Darendeli (2001) and Menq (2003) were used, as detailed in Table 1.



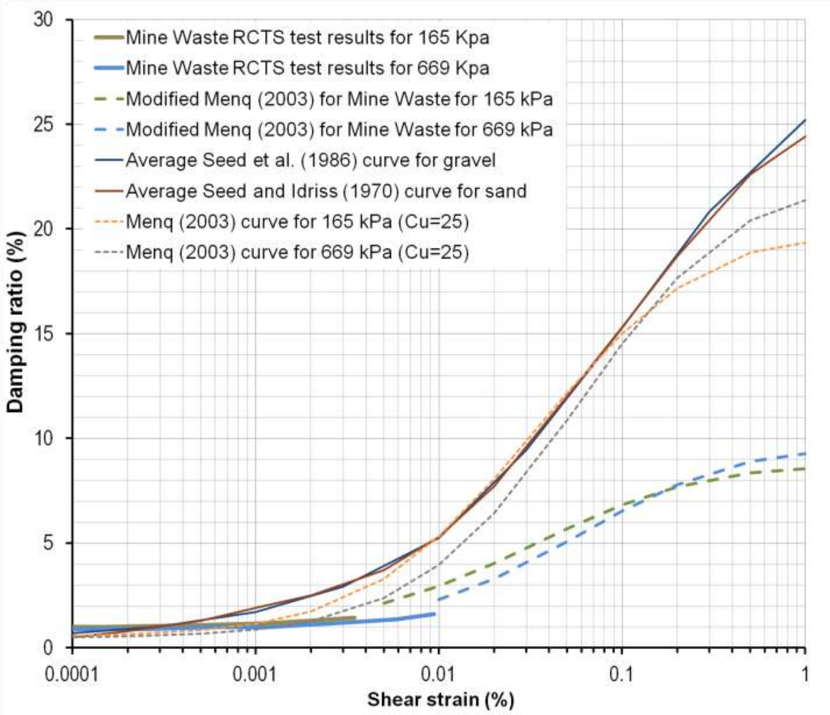


Figure 3: Shear modulus degradation and damping ratio curves for mine waste and comparison with published research

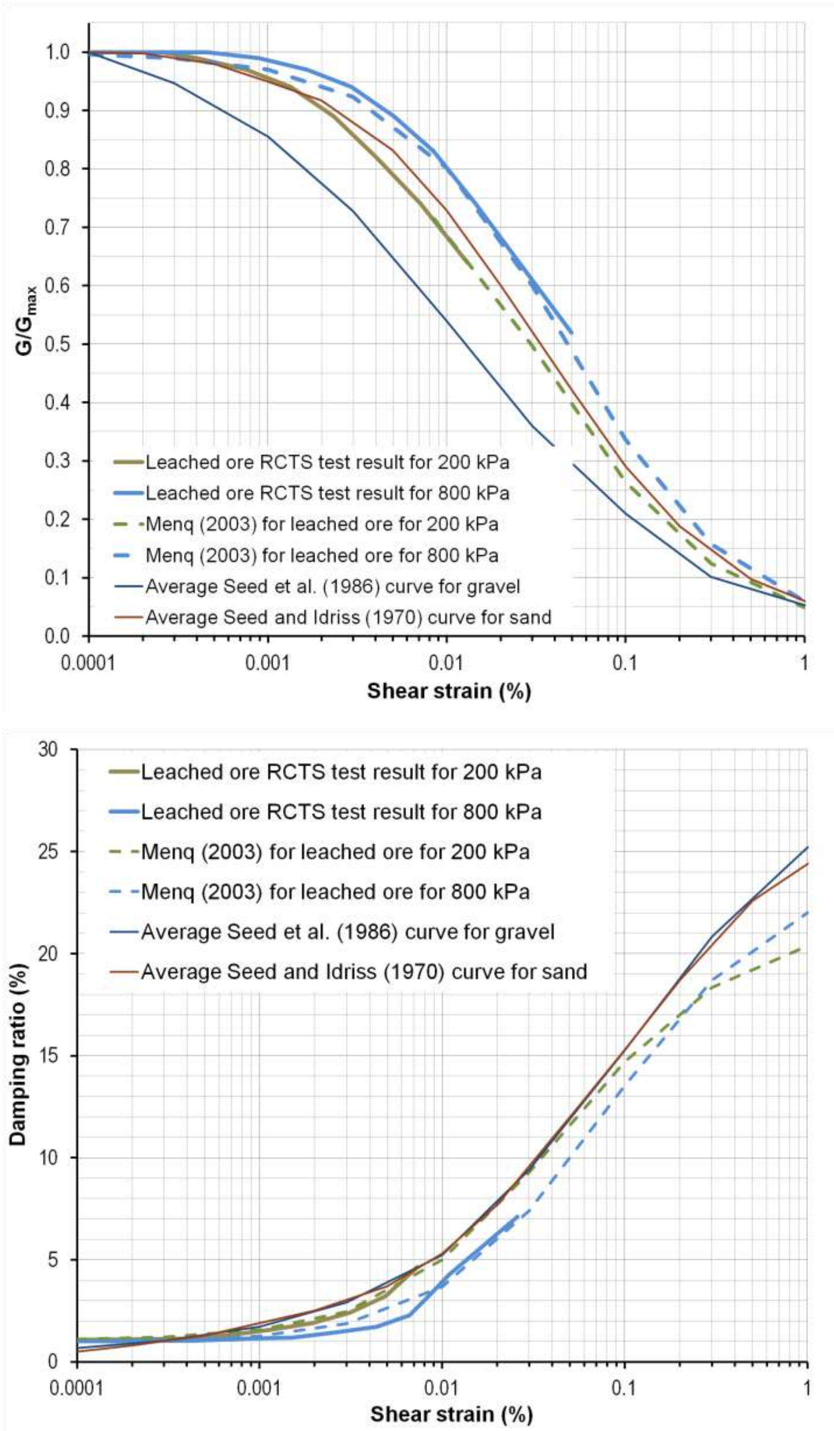


Figure 4: Shear modulus degradation and damping ratio curves for leached ore and comparison with published research

To model the non-linear seismic behavior of the interface of the HLP, project-specific parameters of the DeepSoil's software (Hashash, 2014) hyperbolic and non-lineal model were selected. The back-bone curve for the interface was modelled based on the static shear strength of this material, according to the conclusions and recommendations of Kavazanjian and Matasovic (1995) and Arab (2011). The damping ratio was modelled based on the cyclic shear tests shown by Arab (2011) which showed a relatively constant damping ratio value. The constant nature of the interface damping ratio is similar to the findings of Yegian et al. (1998). It is important to mention that no cyclic shear test on the interface was developed for this paper; however, sensibility analyses were carried out to analyze the inherent uncertainties of this model. These analyses showed similar results for all cases.

Geotechnical analysis

As mentioned before, this paper deals with the seismic stability of both HLP and MWD. The short-term stability condition of these structures, which includes undrained resistance of fine grained soils and deformation due to consolidation, is dealt with by Reyes and van Zyl (2015). Nine geological-geotechnical cross-sections were developed using the geotechnical site investigations, all needed to capture the heterogeneous stratigraphy and the HLP and MWD layouts complexity. Figure 5 shows a plan view of the layout of the HLP on top of the phase 2 of the MWD and its cross-sections.

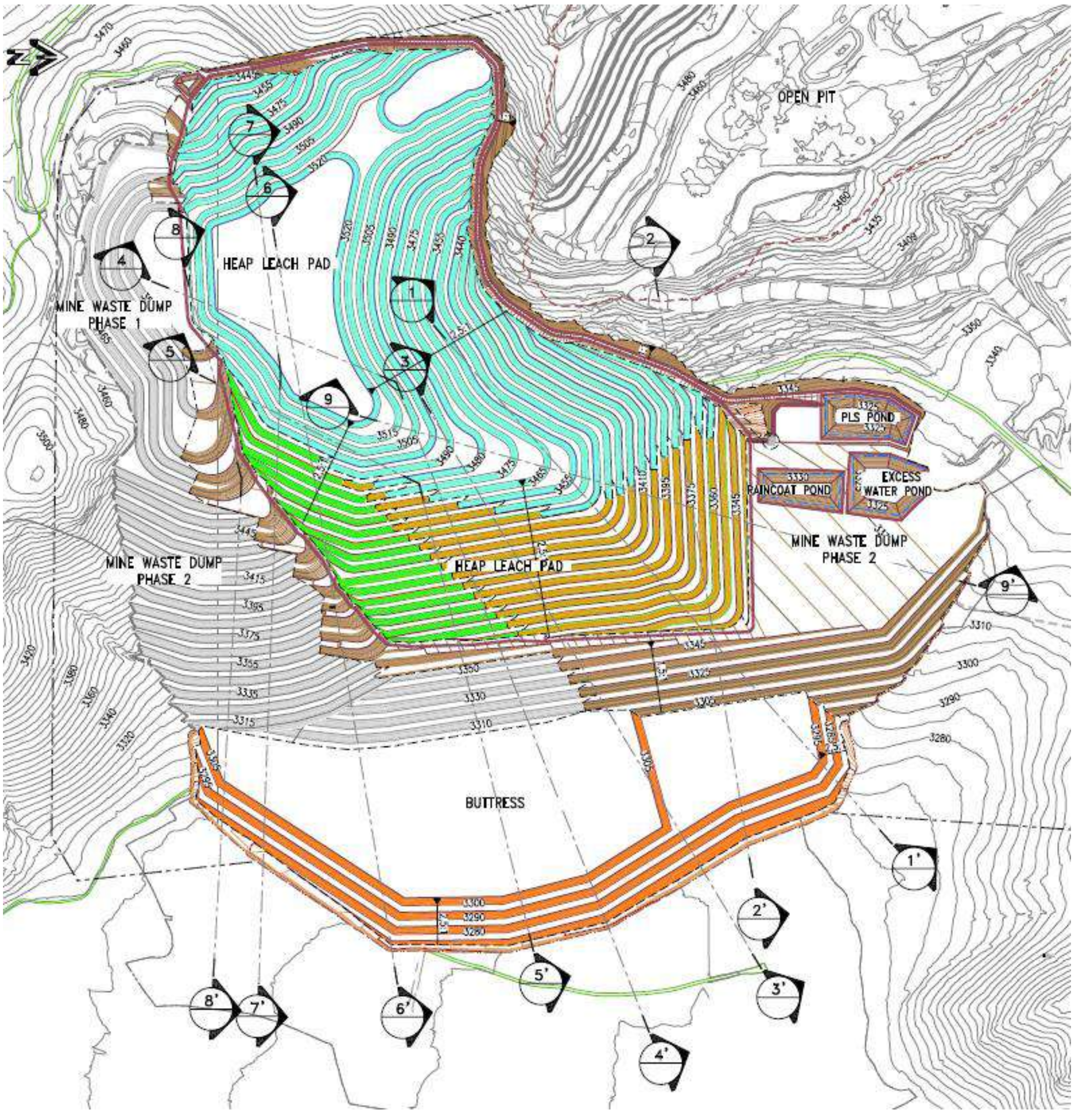


Figure 5: Plan view of the heap leach pad and mine waste dump

Seismicity

As mentioned previously, PGA values and uniform hazard response spectrums available were calculated for soil type B according to the 2012 version of the International Building Code. Additionally, the foundation classified heterogeneously as types C and D. It was clear, then, that seismic response analyses were needed to properly determine the correct seismic accelerations that the potential sliding mass would be subjected. Moreover, the translational failure of the HLP on top of the MWD meant that accelerations for this particular case had to be assessed to account for the seismic response of the mine waste and its influence on this failure mechanism.

The uniform hazard response spectrums for 100 and 475 year return periods (operation and closure conditions, respectively), from the site seismic hazard assessment, were employed in all seismic evaluations. Seismic records from both horizontal components used as input for the site response analyses were obtained from published motions from Peruvian and Chilean subduction earthquakes recorded in Peru. The earthquake motions from the 1974 Lima, 2001 Atico, 2005 Tarapacá and 2014 Iquique earthquakes were chosen. It is important to mention that the Lima and Atico earthquake motions were recorded near the epicenter of the event, capturing their high energy content; however, the Tarapacá and Iquique motions employed were recorded far from their epicenters, as a consequence low values of PGA and energy content were registered. No other earthquake motions were selected due to the limited database available for Peru. All 8 seismic records were spectral matched to the 100 and 475 years return period response spectrums using the SeismoMatch software, which is based in the pulse wave algorithm proposed by Abrahamson (1992) and Hancock et al. (2006).

Soil columns

Two cases of analysis were defined. The first one, Case 1, focused on determining response spectrums to use for rotational and compound failures that cross foundation soils. 11 columns were developed that included only foundation soils and bedrock (without considering both MWD and HLP), which are shown in plan view in Figure 6 along with the geology of the area. Case 2 consisted on determining response spectrums on top of the MWD (without considering the HLP) to use for translational failures along HLP interface. 6 columns were developed that included bedrock, foundation soils and mine waste; these are shown in Figure 7 in plan view along with the design of only the MWD. Additionally and only for this paper, a Case 3 was created consisting of determining response spectrums on top of the HLP. 6 columns were developed, all including bedrock, foundation soils, mine waste, low permeability soil/geomembrane interface and leached ore.

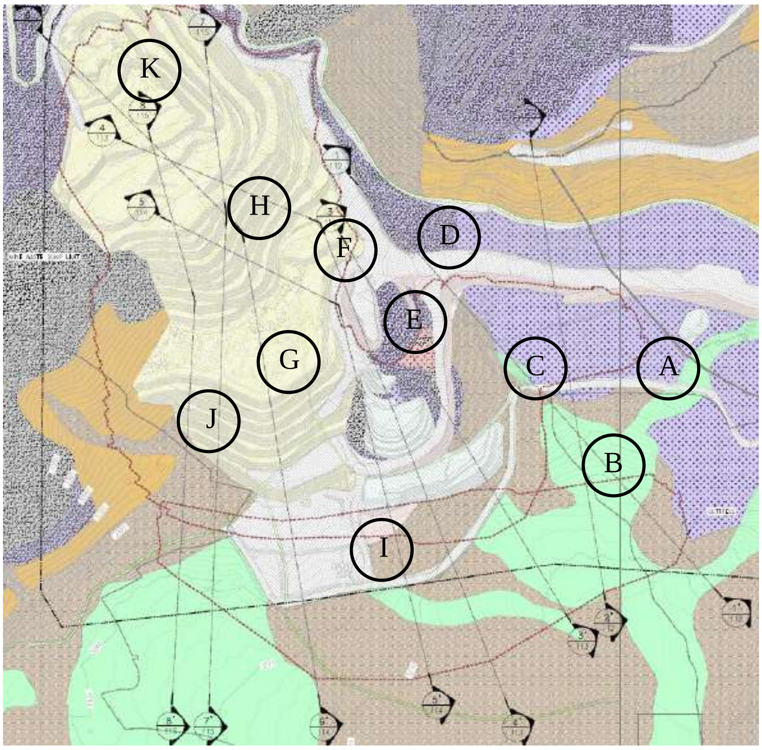


Figure 6: Soil columns' location for Case 1

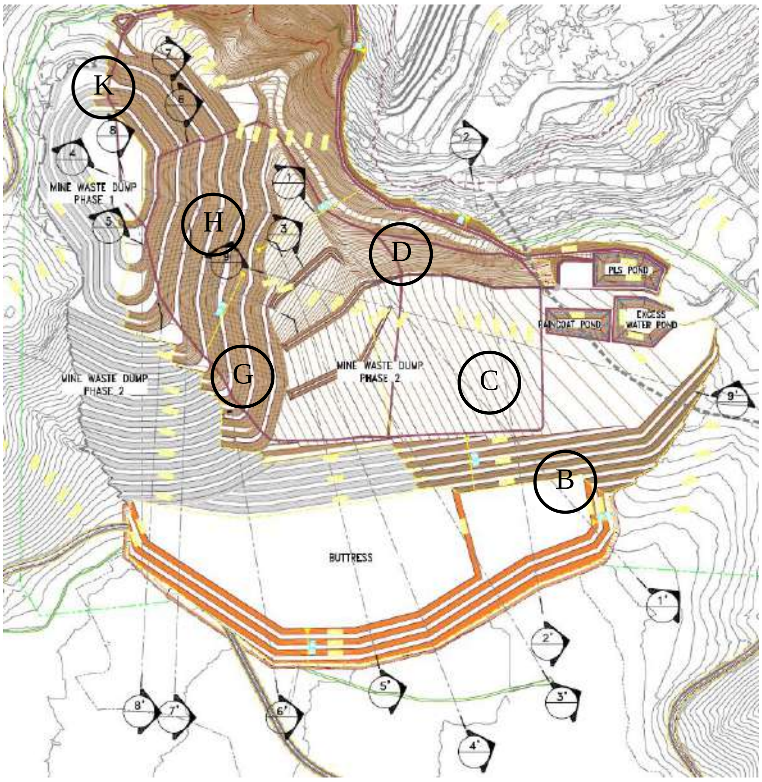


Figure 7: Soil columns' location for Case 2

The locations of all columns were chosen based on stability, seismic and geological criteria in order to capture the complexity of the case study. Cases, columns and their heights, shear wave velocity range and natural period are presented in Table 2. Case 1 columns were built using geophysical information. Case 2 columns were in the same positions as their correlatives of Case 1 and were built using both geophysical information and the resonant column results. It is important to point that in order to estimate the increase of shear wave velocities of the foundation soils due to the loading of the mine waste, several geophysical tests were performed on the toe, middle and crest of the existing MWD. These tests results allow the authors to estimate the overburden effect of the mine waste weight and its influence on the shear wave velocity profile of the foundation soils. Case 3 columns were built in a similar way. Table shows the cases of analysis and columns denominations, heights, shear wave velocity range and natural period.

Table 2: Geotechnical properties for the seismic response analysis

Cases	Columns	Heights (m)	Shear wave velocities range (m/s)	Natural period (sec)
Case 1	A1	43	200 – 770	0.40
	B1	60	170 – 600	0.66
	C1	41	200 – 750	0.40
	D1	24	200 – 600	0.26
	E1	10	450 – 600	0.08
	F1	15	400 – 600	0.12
	G1	45	200 – 580	0.53
	H1	31	200 – 580	0.39
	I1	53	170 – 550	0.61
	J1	53	170 – 600	0.59
	K1	15	400 – 600	0.12
Case 2	B2	120	175 – 685	1.11
	C2	86	175 – 790	0.82
	D2	44	175 – 630	0.49
	G2	165	175 – 700	1.14
	H2	131	175 – 710	1.21
	K2	125	175 – 700	1.17
Case 3	C3	182	220 – 900	1.32
	D3	154	220 – 695	1.19
	E3	100	220 – 710	0.83
	F3	140	220 – 705	1.09
	H3	236	220 – 830	1.64
	K3	150	220 – 750	1.15

Results

All seismic records were used for all analyses. For each and every column, results of the seismic records were averaged since no important variation was found. For Case 1, columns showed very different results, consequently, these were grouped in 4 zones, called "foundation", given the geological and spatial variation of each of them. Table 3 shows the characteristics of these foundations types for Case

1. Figure 8, 9 and 10 show the average results and its standard deviation of Foundation 1, 2 and 3, respectively, for both return period seismic events. Figure 11 shows the average values for Case 1.

Table 3: Geotechnical properties for the seismic response analysis

Cases	Groups	Columns	Natural period range (sec)
Case 1	Foundation 1	B1, I1 and J1	0.59 – 0.66
	Foundation 2	A1 and C1	0.40
	Foundation 3	D1, F1, G1 and H1	0.12 – 0.53
	Foundation 4	E1 and K1	0.08 – 0.12
Case 2	Zone 1	E1 and F1	0.08 – 0.12
	Zone 2	G2, H2 and K2	1.14 – 1.21
	Zone 3	B2, C2 and D2	0.49 – 1.11

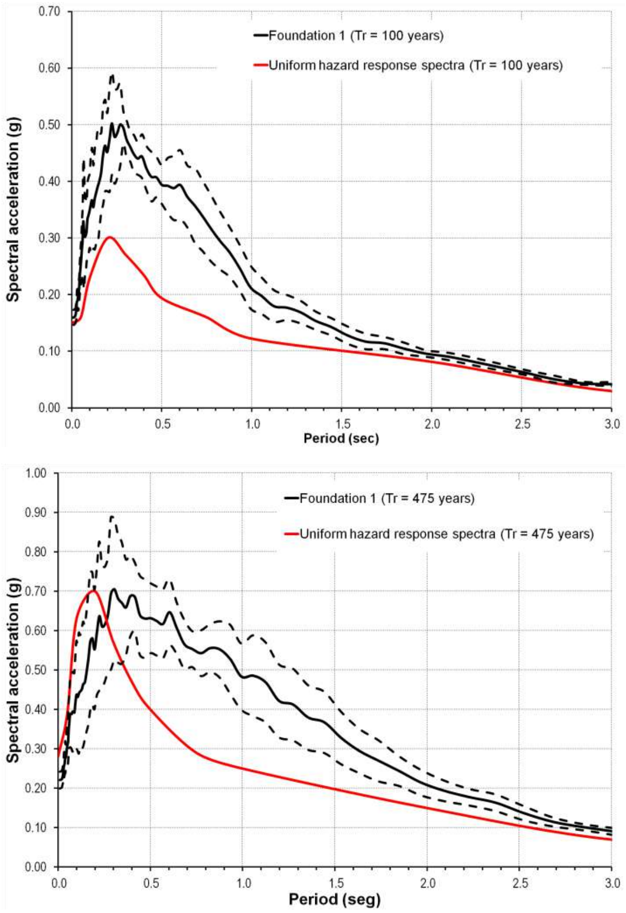


Figure 8: Response spectrums for Foundation 1 type of Case 1

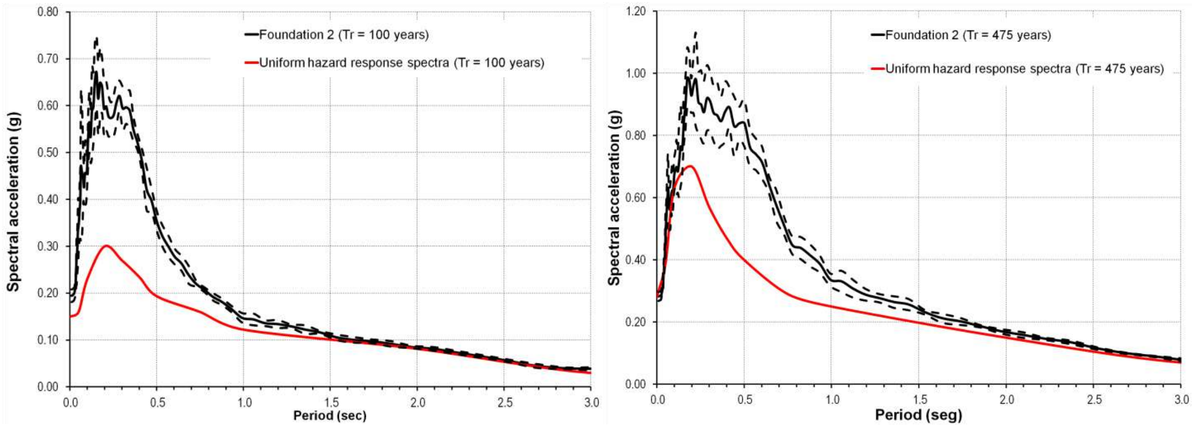


Figure 9: Response spectrums for Foundation 2 type of Case 1

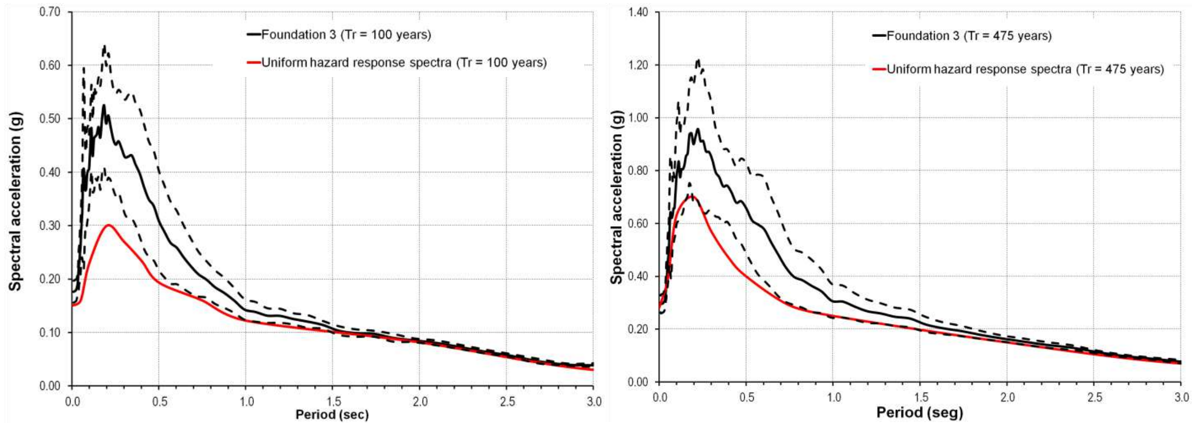


Figure 10: Response spectrums for Foundation 3 type of Case 1

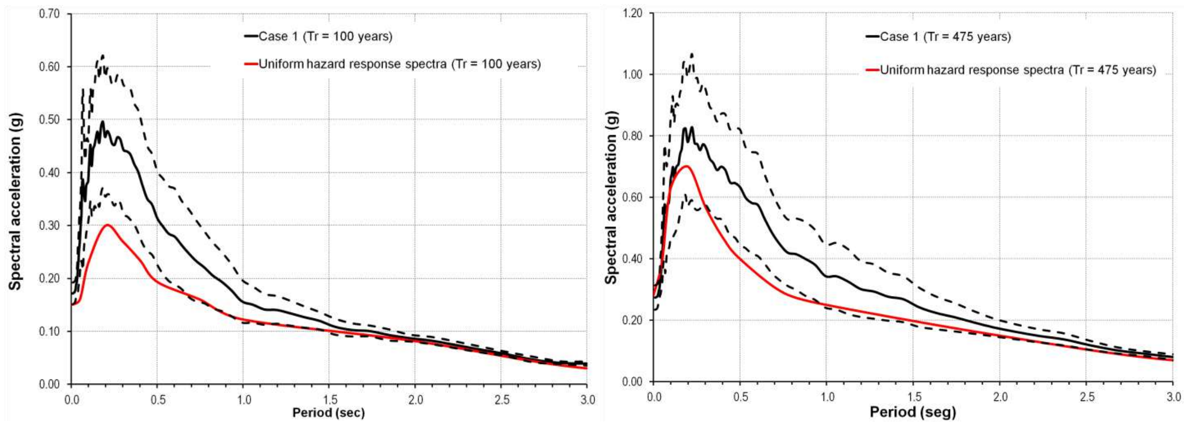


Figure 11: Average response spectrums for Case 1

As can be seen in Figure 8 for Foundation 1, the 100 years return period event results in amplification up 100% around natural periods of the earthquakes and the columns. However, for the case of the 475 years return period event, amplification from 50 to 100% is shown only around the columns' natural period; deamplification happened around and before the earthquake natural period. Figure 9 presents the results for Foundation 2 and shows, for both seismic events, amplification from 50 to 100% around the earthquake natural period. A wider range of amplification resulted in the 475 years results, as it included the column's natural period. For Foundation 3, Figure 10 shows in average amplification from 50 to 100% around the earthquake natural period; however, the variation of results is high due to the different geology and configurations of the columns of this Foundation. Finally, Foundation 4 exhibited almost non-existent amplification due its small size and period. Figure 11, shows the average results from all Foundation types. From all the results, it can be concluded that for the 100 years return period event, in average the amplification occurs around the natural period the design earthquake which, in general, is similar to the ones of the columns. The amplification for the 475 years return period event is relatively lower than the other one, however, it is more pronounced around the natural period of the columns. It can also be seen that the variation of the results for the 475 years return period is higher than the other one, even showing deamplification for some cases. This different behavior can be attributed to the non-linear nature of soils, which is captured in the NLM analysis, for high seismic demands and medium to high shear strain levels.

For Case 2, columns were also grouped in 4 zones due to the geological and spatial variation related to the foundation soils that composed each of them. Table 3 shows the characteristics of these zones. Figure 12 and 13 show the average results and its standard deviation of Zones 2 and 3, respectively, for both return period seismic events.

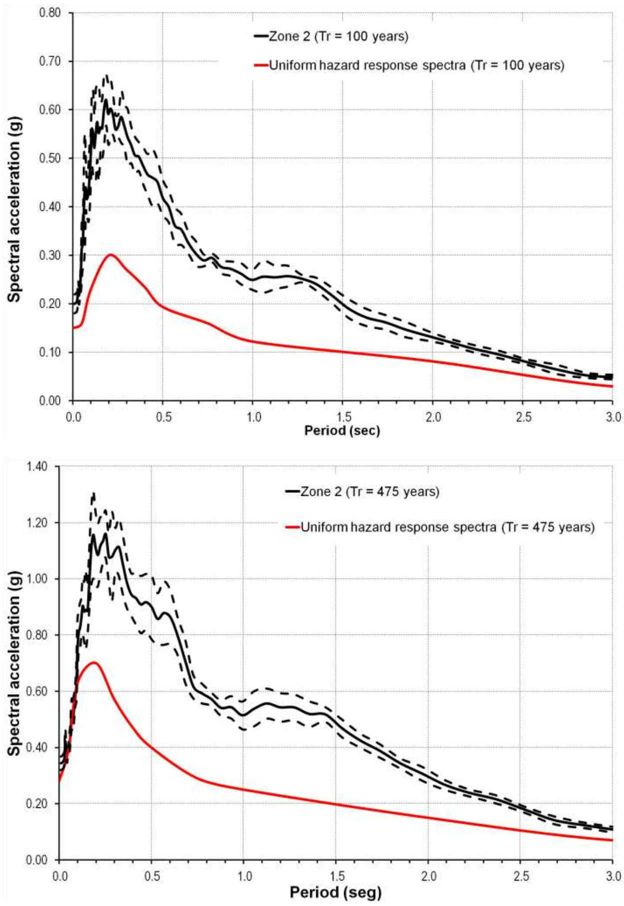


Figure 12: Response spectrums for Zone 2 of Case 2

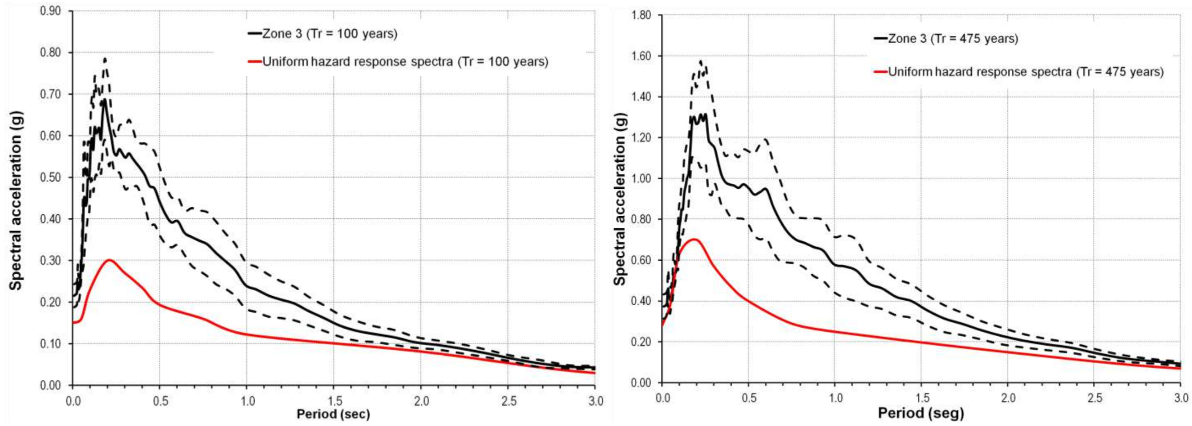


Figure 13: Response spectrums for Zone 3 of Case 2

Figure 14 shows the average values for Case 2. Zone 1 exhibited almost non-existent amplification due its small size and period; it is important to mention that this zone was selected because the HLP in some zones is placed directly on top of rock or shallow soil deposits. As can be seen in Figures 12 and 13 for Zones 2 and 3, respectively, for both seismic events the results showed amplification up 100% around natural periods of the earthquake and columns. Figure 13 also shows how the variation of the natural periods of the columns of Zone 3 results in a high standard deviation of the results when compared to the average value. Figure 14 shows the average values of the columns for Case 2, where the deviation of the results is higher for the 475 years return period for similar reasons as Case 1.

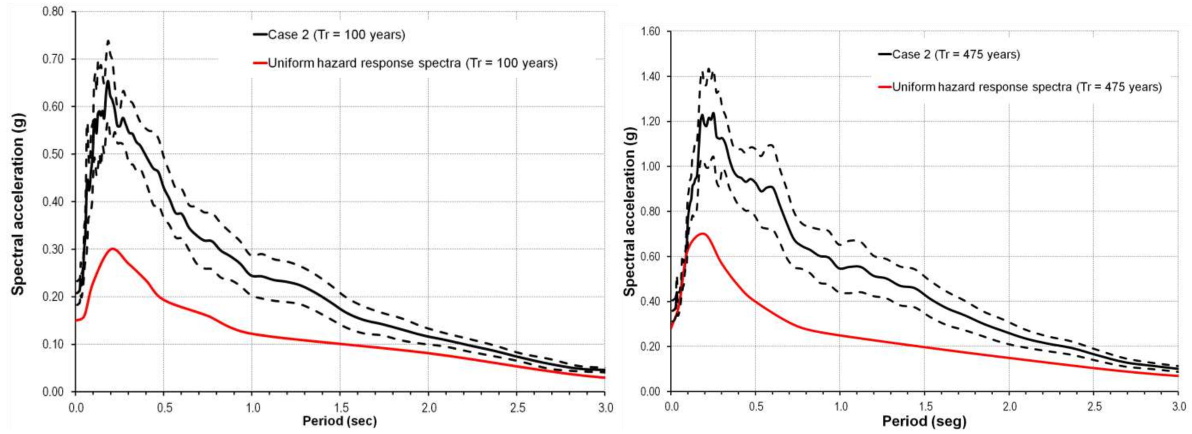


Figure 14: Average response spectrums for Case 2

Since Case 3 was not analyzed for design, only comparison for columns K and H were made for this paper based on the overall results. Figure 15 shows results for 100 and 475 years return period events and presents a comparison for column K of the response spectrums resulting from the uniform hazard study, Case 2 analysis (top of the MWD without the HLP), top of HLP and an alternative case of the top of the HLP without considering the presence of the interface. As can be seen, the interface deamplifies the response for small periods up to 30% and a slightly amplifies for larger periods when comparing only results of response spectrums on top of the HLP. This observation agrees with most of the research done when comparing the effect of the interface; however, it is important to note that this column considers a leached ore height of 24 m. It can also be seen that the presence of the leached ore results in deamplification up to 50% when comparing the results of Case 2 (without the leached ore of the HLP) and the ones on top of HLP (whether is considered the interface or not). Figure 16 shows a similar comparison for column H. Despite the results of column K, column H, with 104 m of leached ore, shows that interface has little effect on its response spectrum. However, the deamplification effect resulting from the presence leached ore on top of the MWD is again seen. This deamplification could be explained given the higher damping of the leached ore relative to the mine waste and higher degradation due to the higher shear strains developed on these granular materials.

The resulting response spectrums for Case 1 and Case 2, and their zones, were used as input for the seismic design of the both MWD and HLP. The Bray and Travasarou (2007, 2009) methods need response spectrums relative to the failure mechanism to be evaluated. In general, these considerations where taken for the calculation of the seismic coefficient (Bray and Travasarou, 2009) and calculation of SIPD (Bray and Travasarou, 2007):

- For rotational and compound failures, which are related to failure mechanisms that cross foundation soils, mine waste and leached ore, response spectrums from Case 1 were used. Since geological and geometrical variation were of concern and response spectrums from the Foundations area evaluated exhibited different results, for the different cross-sections used in the stability analyses different response spectrums were used. The response spectrums of the Foundations areas were selected due to their influence on and for each cross-section.
- For translational failures that occur only for the HLP and cross leached ore on its back scarp and the interface on its base, response spectrums from Case 2 were used. Similarly to the rotational and compound failures, different response spectrums related to the different Zones of Case 2 were employed in the analysis of each cross-section.

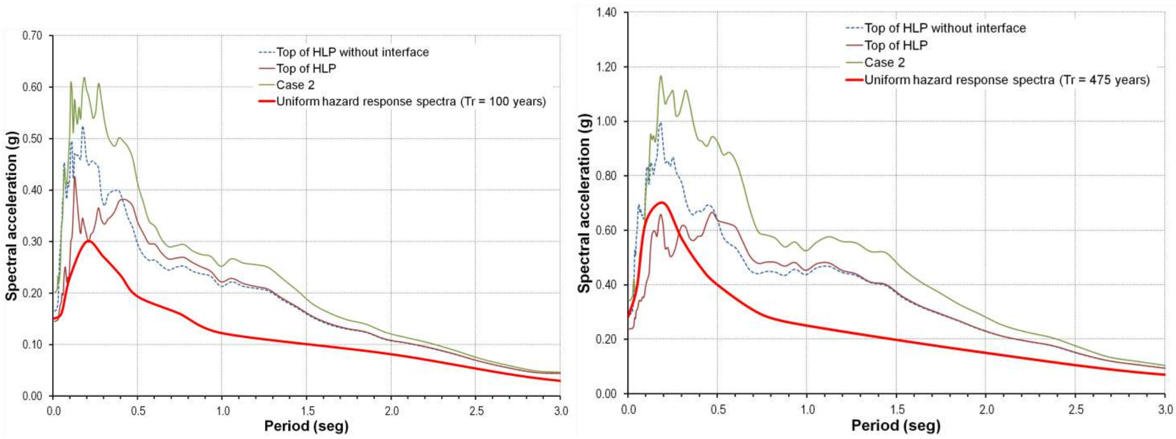


Figure 15: Response spectrums for column K for Case 3

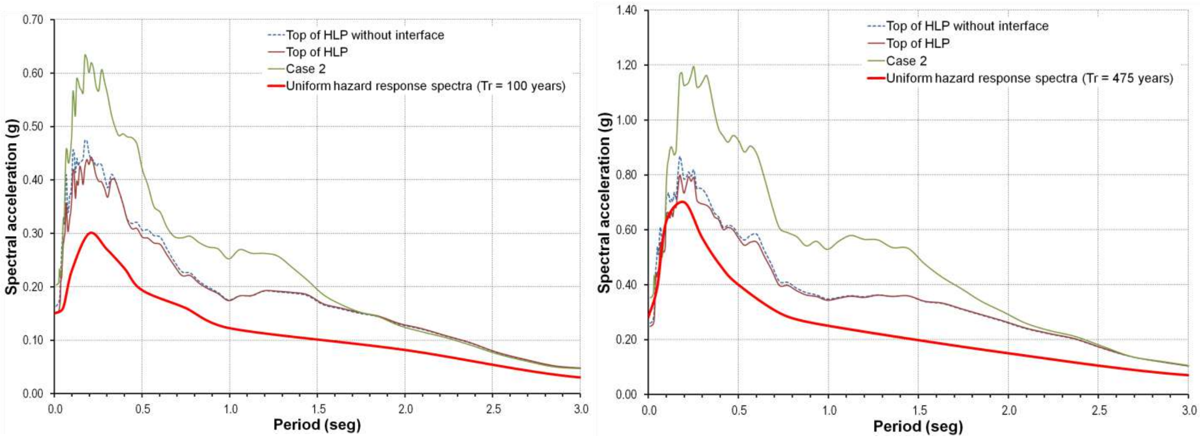


Figure 16: Response spectrums for column H for Case 3

Conclusions

Current state of practice of the design of HLP in the Andean region has many challenges that have complicated its civil and geotechnical design, as well as its operation. The high seismic activity, geological complexity and aggressive terrain in the region can be considered as the most important issues for HLP geotechnical design.

A case study was presented where a HLP was designed, given the lack of an adequate location, on top of a MWD, which itself was built over a heterogeneous and complex deposit of alluvial and residual clayey soils. The seismic design of both structures was dealt with by determining response spectrums for different failure mechanism to use in the calculation of seismic coefficients and SIPD. 1-D non-linear site response analysis were based on state of art techniques such as parallel gradation for the testing of large-sized mine waste and leached ore, RCTS tests and the use of accepted dynamic properties of fine grained soils. The geological complexity and spatial variability of the foundation demanded the use of several soil columns, each one related to a rotational and compound failures that cross foundation soils, mine waste and leached and translational failures that cross leached ore at the back and the HLP interface in its base. Results provided detailed information about the influence of foundation soils, mine waste and leached ore dynamic properties and geometry on the response spectrums and related spectral accelerations for design and for different foundation conditions.

The results highlighted the importance of site conditions on seismic design, particularly in the calculation of SIPD. Important amplification effects were determined around the natural period of the sliding mass, consequently deeply influencing the displacements calculation both for rotational, compound and translational failures. Bray and Travasarou (2007, 2009) used the resulting response spectrums in their formulation to calculate seismic coefficients and SIPD, which resulted in significantly higher values than the ones previously calculated using response spectrums for dense or soft soils from site seismic hazard studies. It is important to mention that SIPD are sensitive to the fundamental period of the sliding mass and correspondent spectral acceleration. Therefore, the determination of the dynamic characteristics of leached ore and interface, and a correct selection of response spectra for design are critical. This research suggests that the seismic design of heap leach pads should be focused on determining SIPD rather than focused on pseudo-static factors of safety unless a rational criterion is used to define the seismic coefficient, such as the one presented by Bray and Travasarou (2009). Finally, the importance of more research is needed to properly define shear modulus and damping ratio curves for mine waste, leached ore and low permeability soil-textured geomembrane interface, given their influence in the overall results.

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