Proceedings of Heap Leach Solutions, 2015
September 14-16, 2015, Reno, Nevada, USA
Published by InfoMine, © 2015 InfoMine, ISBN: 978-0-9917905-8-6

# Procedures for estimating seismic-induced permanent displacements on heap leach pads

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## Abstract

The seismic design of heap leach pads in regions with high seismic activity has usually been performed by pseudo-static analysis. However, new methodologies have been studied that focus on determining seismic-induced permanent displacements; the overall seismic design concept of a heap leach project has shifted to specific allowable levels of displacement rather than a factor of safety (FOS). Heap leach pads and their liner system are considered more sensitive to seismic-induced displacements than other mine facilities due to the potential for tearing of the geomembrane during seismic events.

This paper presents a case study of a heap leach pad where seismic-induced displacements were calculated through rigid block, decoupled, and coupled procedures. The authors use a large set of geotechnical information, including state-of-the-art characterization of static and dynamic properties of leached ore, advanced constitutive models, and other geotechnical details.

The analyses suggest that more research is needed to properly assess the dynamic parameters of leached ore, which are critical when calculating seismic-induced permanent displacements of the heap's translational sliding mass. The analyses showed a good correlation between the results of Bray and Travasarou (2007) and the finite-element methods; the latter validated an optimization of the original design of the heap leach studied. This research suggests that the seismic design of heap leach pads should be focused on determining seismic-induced permanent displacements, rather than focused on pseudo-static FOS, unless a rational criterion is used to define the seismic coefficient.

## Introduction

Historically, most civil engineering structures built in Peru are designed to endure the strong seismic events expected in this region. These events are caused by the subduction zone of the Nazca plate beneath the South American plate. Several local studies in Peru, such as Castillo and Alva (1993) and Gamarra and Aguilar (2009), support the probability of strong earthquakes, which leads to the design being heavily oriented towards seismic design. Those researches show isoacceleration maps for different return periods and soil types. However, the Peruvian mining authority typically requests site seismic hazard assessments for each mine site.

During the last decade, most heap leach pad seismic designs were carried out through the pseudo-static approach using a seismic coefficient ranging from one-half to two-thirds of the peak ground acceleration (PGA). Only in particular cases, typically when the pseudo-static factor of safety (FOS) was too low, were seismic-induced permanent displacements (SIPD) calculated, usually using the Newmark (1965) or Makdisi and Seed (1978) methods. However, modern criteria for seismic design of earth structures defines a maximum allowable displacement those structures can sustain. As a consequence, methods such as the Bray and Travasarou (2007) method are being used to estimate SIPD through a simplified coupled procedure. Furthermore, the Bray and Travasarou (2009) method allows the selection of a seismic coefficient based on maximum allowable displacements for any particular structure, thus improving the pseudo-static approach.

Among mining structures, leach heaps, leach pads and their liner systems are considered particularly sensitive to SIPD. In Peru, these liner systems are usually built using compacted low permeability soils and single textured geomembranes. Seismic permanent displacements of 15 to 30 cm (Kavazanjian et al., 2011) can tear the geomembrane and cause both environmental and economic damage. Subsequently, a great deal of effort is put into determining an appropriate seismic coefficient to use on a pseudo-static analysis or estimating reliable values of SIPD.

The objective of this paper is to compare different approaches employed in practice to determine SIPD. A case study is presented in which the Houston et al. (1987), Makdisi and Seed (1978) and Bray and Travasarou (2007) methods were used, as well as a fully-coupled dynamic analysis in PLAXIS.

## 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 heaps and leach pads, since leached ore liquefaction is not generally expected. However, there have been reported cases of this phenomenon (Breitenbach and Thiel, 2005; Castillo et al., 2005), mainly because of high fines content and low permeability of some crushed ore. Inertial stability analysis deals with displacements produced by temporary exceedances of the material’s shear strength by dynamic stresses, assuming that this shear strength remains relatively constant during the seismic event. A FOS calculation by a pseudo-static analysis and SIPD calculation are methodologies used to understand inertial instability.

Following an extensive review of existing methods, Murphy (2010) defined three approaches to estimate seismic-induced displacements: rigid-block, decoupled, and coupled analysis. For the rigid-block analysis, the Newmark (1965) method is the most recognizable. The Makdisi and Seed (1978) is one of the most used decoupled methods, and the Bray and Travasarou (2007) method as well as numerical dynamic analyses performed by software such as PLAXIS or FLAC are part of the coupled methods. The following sections describe the theoretical background of these methods.

### Newmark (1965) and Houston et al. (1987)

Newmark (1965) was the first to formulate the rigid-block analogy, and his methodology has been widely used to calculate SIPD for most geotechnical structures. The Newmark method considers a rigid block mass sliding on an inclined plane, whose SIPD equals the double integration of the difference between earthquake acceleration and a yield acceleration ($k_y$), with the latter concept referring to the overall slope resistance, which in turn depends primarily on the dynamic strength of the material along the critical sliding surface and the structure's geometry and weight (Bray, 2007). Several authors have modified the original Newmark (1965) method to overcome simplifications such as the inclined plane and the rigidity of both the sliding mass and slip surface assumptions.

Houston et al. (1987) modified the Newmark (1965) methodology by introducing a slip layer, whose "softened" properties would prevent accelerations within the sliding mass to exceed $k_y$. Accelerations that surpass $k_y$ within the sliding mass would generate movements in it and then be inconsistent with the original assumption of the rigid-bock method. Typically, the seismic record below the slip layer is used to calculate displacements.

### Makdisi and Seed (1978)

In their landmark paper, Makdisi and Seed (1978) formulated the decoupled method, which consists of two separate steps: a dynamic response analysis and a sliding response analysis. The first one is performed to quantify the accelerations experienced by the sliding mass. The second one is performed to calculate SIPD through double integration of an earthquake motion. Makdisi and Seed (1978) used average accelerations computed by the procedure of Chopra (1966) and sliding block analyses to compute SIPD of earth dams and embankments (Kramer, 1996).

Makdisi and Seed (1978) were the first to develop a series of calculation charts based on their simplified decoupled method by the analysis of three earthquake records with different magnitudes. One of their charts evaluates the seismic average equivalent accelerations experienced by the sliding mass as a function of the slip surface depth, main body height, and crest peak acceleration of a dam. The other chart is employed to estimate SIPD with respect to the fundamental period of the embankment (Murphy, 2010). The Makdisi and Seed (1978) method is still widely used within the geotechnical community for a broad range of structures, primarily due to its simplicity, despite the fact that it was only developed for dams and embankments.

### Bray and Travasarou (2007)

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). MSWF are usually designed with liner systems, similar to leach pads, which control its failure mechanism, thus limiting the maximum allowable seismic-induced displacements.

Bray and Travasarou (2007) noted that the major uncertainty for the evaluation of an earth structure is the 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.

### Stress deformation analysis

A powerful tool to estimate both static and SIPD is the use of stress-deformation analyses that employ 2-D and/or three 3-D finite elements or finite different models. These analyses include seismically-induced permanent strains in each element of the finite element mesh or zone of the finite difference model (Kramer, 1996). Conceptually, a fully-coupled nonlinear analysis should be able to calculate any SIPD in any slope; however, such analyses are very complex (Duncan and Wright, 2005). Without initial simplifying assumptions, the accuracy of the stress deformation analysis depends on the stress-strain or constitutive model capacity to represent the real soil behavior (Kramer, 1996). Computer programs such as PLAXIS or FLAC are widely used to assess the seismic behavior of most geotechnical structures.

### Pseudo-static analysis

This approach consists of performing a slope stability analysis, usually by the limit equilibrium method, where a 2-D FOS is computed in which a static horizontal inertial force is applied to the potential sliding mass. This force, expressed as the product of a seismic coefficient (k) and the potential sliding mass weight, represents the destabilizing effects of a design earthquake to the analyzed structure. Hence, the validity of this approach is based on a k value representing the seismic loading.

The pseudo-static screening procedure of Hynes-Griffin and Franklin (1984) recommends, among other things, the use of half of PGA at the site, based on their assumption that 1 m of SIPD is acceptable for most earth dams. As a consequence, this approach should not be used for structures with lower values of maximum allowable SIPD, such as heap leach pads. Given the need for an appropriate method to select a seismic coefficient considering the facility-specific maximum allowable SIPD, Bray and Travasarou (2009) presented a procedure, based on the Bray and Travasarou (2007) approach, that permits the selection of a project-specific allowable level of SIPD, and estimates the fundamental period of the sliding mass as well as a site-dependent seismic demand (expressed in terms of spectral acceleration) so that a rational seismic coefficient can be calculated.

## Case study geotechnical overview and analysis

The case study is a 120 m-high heap located at a mine site in southern Peru, with a maximum capacity of 40 Mt. The original seismic design only involved a pseudo-static analysis using a seismic coefficient of 0.19 (50% of the PGA), and resulted in the configuration shown in Figure 1, which included a 100 m-width bench to account for seismic instabilities. The task was to reevaluate the static and dynamic stability of the heap, focusing on updating the geotechnical parameters and accurately estimating SIPD in order to take advantage of the existing ramp and to increase the capacity.

The following sections describe the geotechnical site investigation and laboratory tests carried out, as well as a detailed description of the geotechnical analysis performed. These evaluations included static stability analyses through the limit equilibrium and finite element techniques, followed by pseudo-static analyses and the calculation of SIPD using the Houston et al. (1987), Makdisi and Seed (1978), Bray and Travasarou (2007) and finite element stress deformation methods.

### Geotechnical site investigation and laboratory tests

Given the nature of translational surface failures, which are typically the main cause of failure of heaps, the focus of the geotechnical site investigation was on characterizing both low permeability soil-single textured geomembrane interface and leached ore. The particle size distribution (PSD) of the crushed leached ore was determined in the field by several excavations in the heap. In each excavation, almost 1 tonne of leached ore was evaluated, first weighting the whole sample and then separating particles with sizes smaller than 3". Using large meshes, the PSD of all material larger than 3" was determined at the site. Complementary laboratory tests on the smaller size particles completed the global PSD curve. Figure 2 shows the average global PSD of the leached ore. For the interface, undisturbed samples of low permeability soil and geomembrane were taken from two locations at the heap toe.

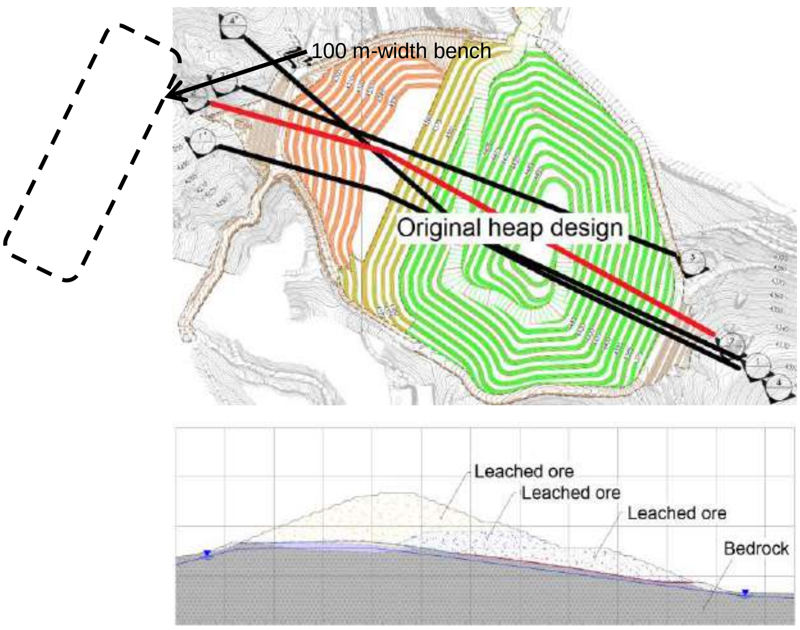



**Figure 1: Plan view of the original design of the leach pad and heap critical cross-section**

Due to the large particle size of the crushed ore and using 1” as maximum particle size, a parallel gradation curve to the original PSD was developed (see Figure 2). By using this curve, scalping would not be needed when testing on a 150-mm diameter triaxial consolidated-drained test. The technique used to build the parallel gradation curve was first developed by Lowe (1964), and then extensively used by Marachi et al. (1969), Thiers and Donovan (1981), and Varadarajan et al. (2003) in order 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 when testing alluvial and waste rock materials. The laboratory tests performed for this case study were used to obtain shear strength properties for leached ore and provided representative stress-strain curves for the numerical models.

Another parallel gradation curve was built in order to perform cyclic tests on leached ore and obtain shear modulus degradation and damping ratio increment curves (see Figure 2). Two sets of cyclic tests on this sample, with a maximum particle size of ¾", were performed in a resonant column and cyclic torsional shear device (RCTS) built at the University of Texas at Austin. 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 modeled by the use of the parallel gradation technique on dynamic testing, thus making these tests results the first to be published.

For the current case study, three sets of large scale direct shear (LSDS) tests were performed on the low permeability soil-textured geomembrane interface: two of them tested undisturbed and remolded soil samples with normal pressures up to 800 kPa in a local laboratory. One extra set of tests was carried out on remolded samples using normal pressures up to 2,000 kPa, since most of the interface in the leach pad is subjected to normal stresses from 1,000 to 2,000 kPa. Along with the tests above described, a detailed review of all previous field and laboratory tests was executed that allowed to properly define both static and dynamic properties of all materials involved in the geotechnical design.

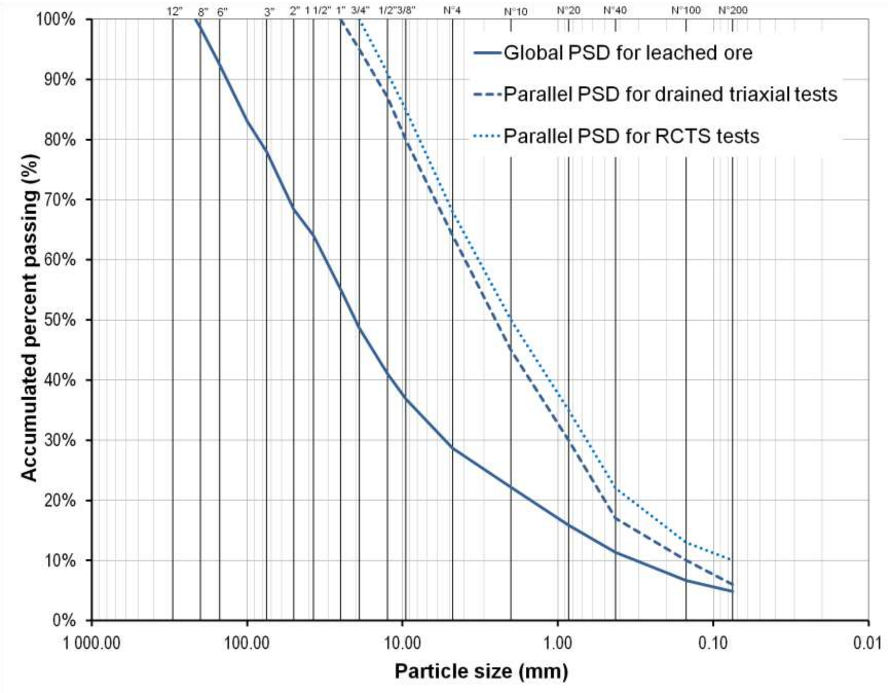



**Figure 2: Particle size distributions of the original material and two sets of scaled samples**

### Static analyses

Since the leach pad was located over fair to good quality rock, only translational failures were of concern, which provided lower FOS than the rotational ones. Therefore, a lot of effort was made to properly determine and model both shear resistance and deformational parameters of leached ore and interface. The drained triaxial test on the leached ore provided a nonlinear shear strength envelop; this material was considered cohesionless with a reducing friction angle as confining pressure increases. The range of variation of the friction angle was defined from 40° to 35°. Another nonlinear envelope of shear strength was defined for the interface. Since nonlinearity of soil-geomembrane interfaces deeply influences its stability (Parra et al., 2012 and Ayala and Huallanca, 2014), the LSDS results at high normal stresses confirmed the shear strength decay at high loadings. Figure 3 shows the nonlinear resistance envelopes for both materials.

2-D slope stability analyses by the limit equilibrium method using Spencer's (1967) procedure were performed on all 4 cross-sections, shown in plan view in Figure 1. The resulting 2-D FOS of the original design cross-sections, as well as the SIPD estimated, showed adequate stability conditions (see Table 1). However, it should be noted that the nonlinear resistance envelope of the interface for high normal stresses reduced the FOS when compared to the ones calculated in the original design, primarily because the envelope was only defined based on tests up to 800 kPa.

To optimize the heap leach capacity, a redesign was proposed by reducing the width of the existing ramp from 100 to 25 m. This configuration was analyzed; the resulting FOS are shown in Table 1. Figure 4 shows the optimized design.

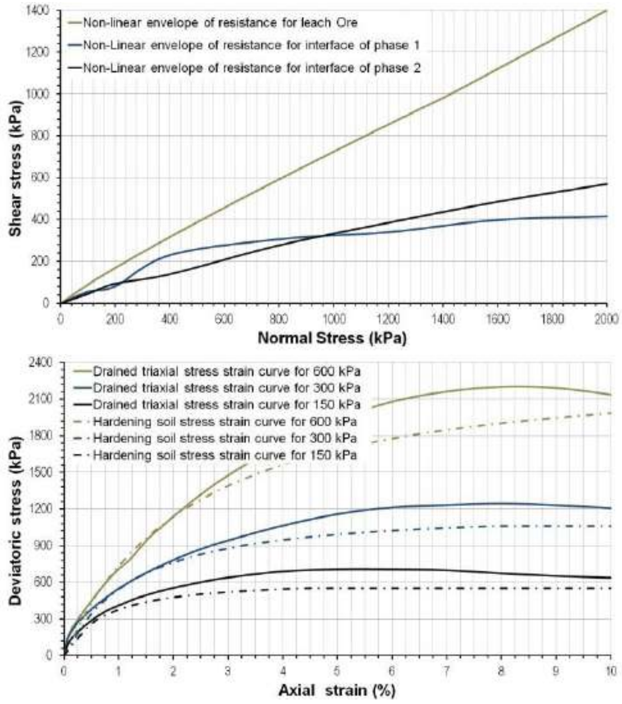



**Figure 3: Non-linear shear strength envelopes for leached ore and low permeability soil-geomembrane interfaces (left) and calibrated stress strain curves from HS model for leached ore (right)**

**Table 1: 2-D FOS calculated from the static analysis of the optimized design**

| Cross-section | Method | Factor of safety | | |
|---|---|---|---|---|
| | | Static | Pseudo-static | |
| | | | 100 years return-period | 475 years return-period |
| 1–1' | Limit equilibrium | 1.43 | 1.04 | 0.93 |
| 2–2' | Limit equilibrium | 1.41 | 1.01 | 0.90 |
| | Finite element | 1.41 | 0.96 | 0.87 |
| 3–3' | Limit equilibrium | 1.43 | 1.03 | 0.92 |
| 4–4' | Limit equilibrium | 1.41 | 1.02 | 0.91 |

Given the need of a finite element model for the dynamic analysis, the computer program PLAXIS was used to analyze the critical cross-section (section 2-2') of the heap leach pad. The authors compared the 2-D limit equilibrium analysis FOS to the 2-D finite element model FOS to verify the static properties of the materials used in the latter, since no available constitutive model in PLAXIS allows for an easy representation of nonlinear envelopes of resistance. The Hardening Soil (HS) model (Brinkgreve et al., 2014) was employed for the numerical modeling of the deformational behavior of the leached ore, calibrating it with the resulting stress-strain curves from the triaxial test. The HS is an advanced model for simulating the behavior of different types of soil, both soft and stiff (Schanz, 1998). It is based on the Duncan and Chang (1970) hyperbolic model, introducing the plastic theoretical approach rather than elastic theory and including soil dilatancy, a yield cap and a Mohr-Coulomb failure envelope (Brinkgreve et al., 2014). Another feature of the HS model is the stress dependency of the stiffness, which allowed performing a proper staged construction model of the heap. Figure 3 shows the original triaxial stress-strain curves and the ones generated by the calibrated HS model of the leached ore.

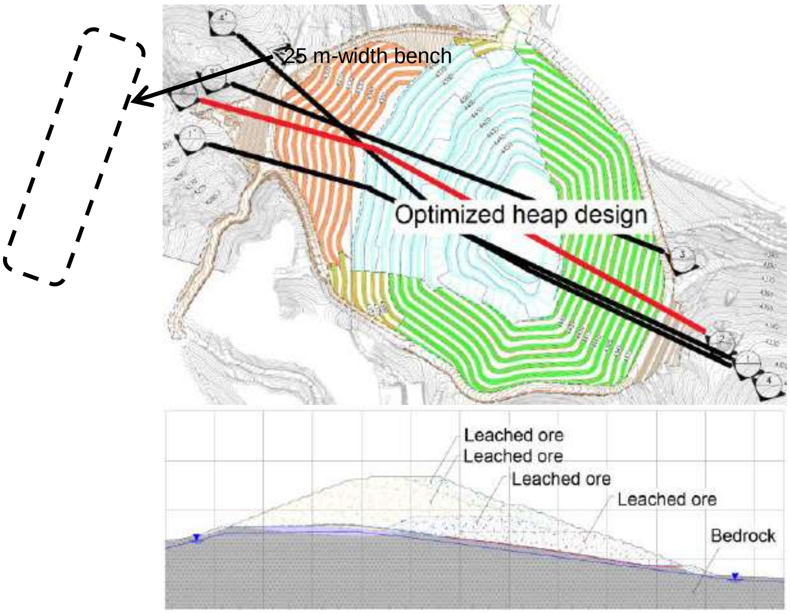
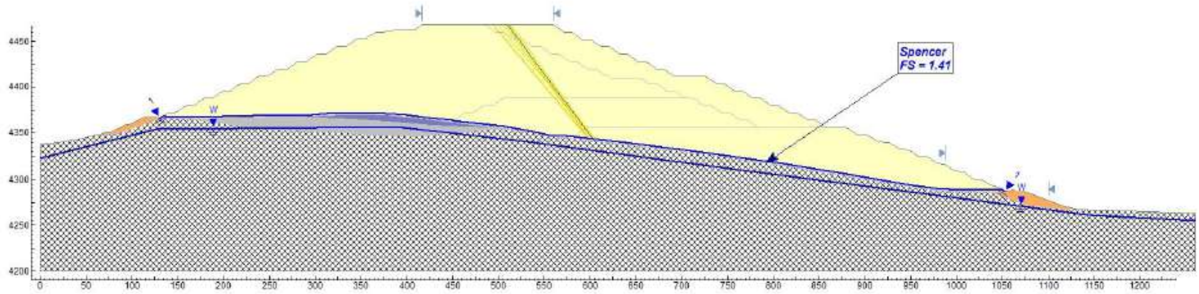



**Figure 4: Plan view and critical cross-section of the optimized design of leach heap**

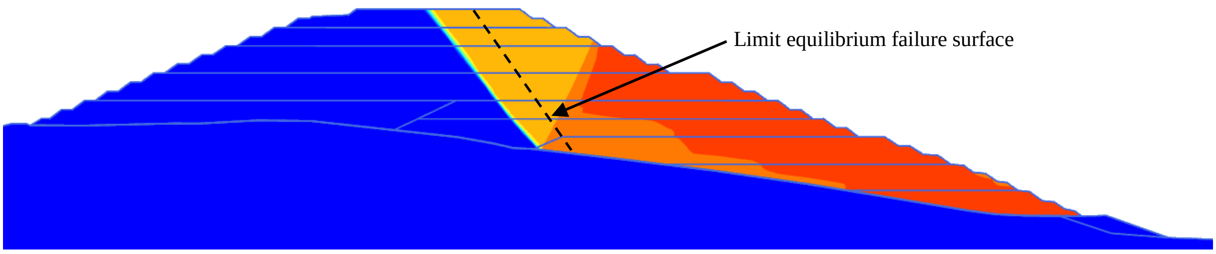
To model the nonlinear resistance behavior of the interface in PLAXIS, the heap was previously built and analyzed in the software. These preliminary results allowed the authors to discretize the interface in PLAXIS according to the normal stresses applied to this material at the final stage of construction. Each discretized cluster was modeled by the linear elastic and perfectly plastic Mohr Coulomb model and assigned different shear resistance parameters based on the nonlinear resistance envelope used in the limit equilibrium analysis. As for the deformational parameters, since the results of the LSDS tests are usually presented in terms of displacements rather than shear strains, the authors used the shear box length of the LSDS device to obtain pseudo shear strain curves. These curves provided the shear modulus and shear resistance parameters that were later calibrated for a direct shear test simulated in PLAXIS. The resulting static 2-D FOS calculated by PLAXIS using the shear strength reduction approach for section 2-2' is presented in Table 1. As it can be seen, the FOS from the finite element model of the section 2-2' is very close to the FOS obtained by the limit equilibrium method. Figure 5 shows the resemblance of the 2-D translation failure surfaces obtained from both limit equilibrium and finite element analysis.

### Seismicity

The uniform hazard response spectrums for 100 and 475 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 site response analysis 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 (two horizontal components per earthquake) 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).




**(a)**




**(b)**

**Figure 5: 2-D failure surfaces from the (a) limit equilibrium and (b) finite element analyses for the section 2-2'**

## Seismic analyses

The pseudo-static analysis carried out in the original design report considered the use of 50% of the PGA as the seismic coefficient (0.10 and 0.19 for operation and closure conditions, respectively), which resulted in FOS slightly above 1. The authors recalculated the seismic coefficients with the Bray and Travasarou (2009) method, using 15 and 50 cm as maximum allowable SIPD for operation and closure conditions, resulting in coefficient values of 0.10 and 0.14, respectively. The recalculated FOS from the pseudo-static analysis resulted far above 1. So, as previously stated, an optimization of the design by reducing the width of the ramp from 100 to 25 m was proposed and seismically analyzed. The coupled Bray and Travasarou (2007) method was used as basis for the optimization, the rigid block method of

Houston et al. (1987) and coupled finite element dynamic analysis using PLAXIS were performed to further verify those results. The decoupled Makdisi and Seed (1978) method was used as a means of comparison only for this paper. The following sections describe the dynamic parameters of the material involved and the details and results of each analysis.

### Dynamic properties

Shear wave velocities for the leached ore were obtained from previous geophysical surveys on top of the heap leach pad. These shear wave velocity profiles were compared to the shear wave velocity results of the RCTS tests, which provided a logarithmic relationship between the velocity and the mean effective stress, both approaches showed a close fit. RCTS tests also provided two sets of shear modulus and damping ratio curves for representative confining pressures, which are shown in Figure 6. 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) and Liao et al. (2013).

The comparison showed a clear deviation of these results from all previous literature curves, although it confirmed the change in the nonlinear behavior affected by confining pressure. The 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 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. More test data 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. This modification was exclusively made for this particular RCTS results for both shear modulus and damping ratio curves, as shown in Figure 6.

On the other hand, dynamic properties of the interface were modeled based on the technique recommended for linear equivalent analysis by Yegian et al. (1998), which considers an equivalent soil layer of 1 m of thickness. For the finite element dynamic analyses, the HS small strain model was used to model leached ore dynamic response. The HS small strain model in PLAXIS is based on the HS model and uses almost entirely the same parameters, adding only two that allow hysteresis in cyclic loading and damping. However, as its name suggests, stiffness degradation is bounded by a low limit of strain. For this reason, one dimensional (1-D) seismic response analyses were included to determine the maximum strains the leached ore would be subjected to. It was verified that these strains would not surpass the strain limit of the HS small strain model in PLAXIS.

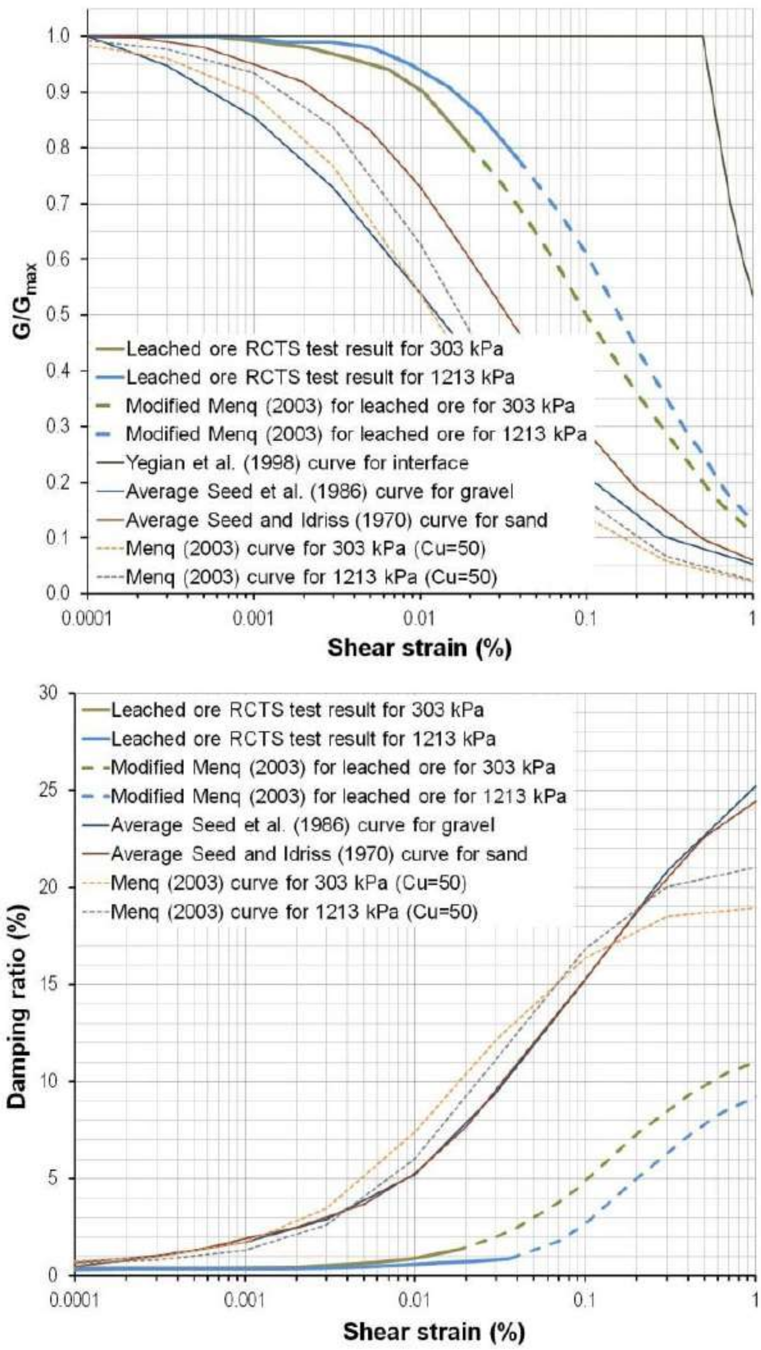



**Figure 6: Shear modulus degradation and damping ratio curves for leached ore and comparison**

The interface in PLAXIS was modeled with the Mohr-Coulomb model. Their dynamic parameters were defined by checking the strains developed in the 1-D seismic response analysis. Using these strains, degraded shear moduli were assigned for each discretized cluster, based on the curves suggested by Yegian et al. (1998). After all these simplifications, response spectrums for all materials resulting from the 1-D seismic response analysis and the PLAXIS model were compared, showing a good general agreement of both approaches.

### Seismic analyses and results

SIPD for operation and closure conditions, associated with 100 and 475 years return period seismic events, were calculated using the Houston et al. (1987), Bray and Travasarou (2007) and stress-deformation methods. The Makdisi and Seed (1978) method was used for this paper for comparison purposes, by using their original charts.

The linear equivalent seismic response analyses for the Houston et al. (1987) method were executed with the software DeepSoil (Hashash, 2014) and using all 8 seismic records. The D-MOD (Matasovic, 1993) software was used to calculate the SIPD based upon the seismic records from the response analysis. The average SIPD values of all records are presented in Table 2. The Makdisi and Seed (1978) and Bray and Travasarou (2007) SIPD values are also presented in Table 2. All 8 seismic records were used in the finite element PLAXIS analysis. Figure 7 shows the cumulative displacements of the Houston et al. (1987) analysis and Figure 8 presents the horizontal displacements of the PLAXIS model.

Table 2 presents the average and range of values of horizontal displacements developed along the interface for the 100 year return-period event. The fully coupled PLAXIS analysis showed an average displacement of 2.5 cm, ranging from 0.3 to 3.0 cm for a failure mechanism similar to the one showed in Figure 7. Using these results as basis for comparison, the Houston et al. (1987) and Bray and Travasarou (2007) resulted in much lower values. However, these three procedures resulted in negligible displacements when compared to the 15 cm defined as a limit for operation conditions. On the other hand, the Makdisi and Seed (1978) method resulted in a wider range of displacements, much higher than the PLAXIS results and almost half of the maximum allowable displacement (15 cm). It is important to mention that the Bray and Travasarou (2007) predicts a probability of negligible displacements of 100%, which is confirmed by the Houston et al. (1987) results and partially by the PLAXIS model.

**Table 2: SIPD values for 100 year return-period**

| Column / Cross-section | Seismic permanent displacements (cm) | | | | | | | |
|---|---|---|---|---|---|---|---|---|
| | Makdisi and Seed (1978) | Houston et al. (1987) | | Bray and Travasarou (2007) | | | PLAXIS | |
| | Range | Average | Range | Probability of negligible displacements | Average | Range | Average | Range |
| 1 | – | 0.0 | 0.0–0.1 | – | – | – | – | – |
| 2 | – | 0.1 | 0.1–0.2 | – | – | – | – | – |
| 3 | – | 0.1 | 0.1–0.2 | – | – | – | – | – |
| 2–2' | 0.8–8.0 | 0.1 | 0.0–0.2 | 100% | 0.3 | 0.2–0.6 | 2.5 | 0.3–3.0 |

**Table 3: SIPD values for 475 year return-period**

| Column / cross-section | Makdisi and Seed (1978) | Houston et al. (1987) | | Bray and Travasarou (2007) | | | PLAXIS | |
|---|---|---|---|---|---|---|---|---|
| | Range | Average | Range | Probability of negligible displacements | Average | Range | Average | Range |
| 1 | – | 6.6 | 4.2–7.5 | – | – | – | – | – |
| 2 | – | 6.8 | 4.3–7.7 | – | – | – | – | – |
| 3 | – | 7.4 | 4.7–8.8 | – | – | – | – | – |
| 2–2' | 40–150 | 6.9 | 4.2–8.8 | 5% | 9.1 | 4.6–18.2 | 7.4 | 2.0–12.6 |

Seismic permanent displacements (cm)

Table 3 shows the average and range of values of horizontal displacements developed along the interface for the 475-year return-period event. The PLAXIS model results in an average displacement of 7.4 cm, ranging from 2.0 and 12.6 cm; the failure mechanism is shown in Figure 8. Stress deformation results were compared again to the Houston et al. (1987) and Bray and Travasarou (2007), it resulted in similar values of average displacements. However, the Bray and Travasarou (2007) average value is slightly more conservative. Furthermore, the maximum value of displacements calculated from the PLAXIS model is within the Bray and Travasarou (2007) range; in contrast, the Houston et al. (1987) range of results underestimates displacement. Finally, the Makdisi and Seed (1978) method resulted in much higher displacements than the one obtained by PLAXIS, being almost equal to three times the maximum allowable displacement (50 cm).

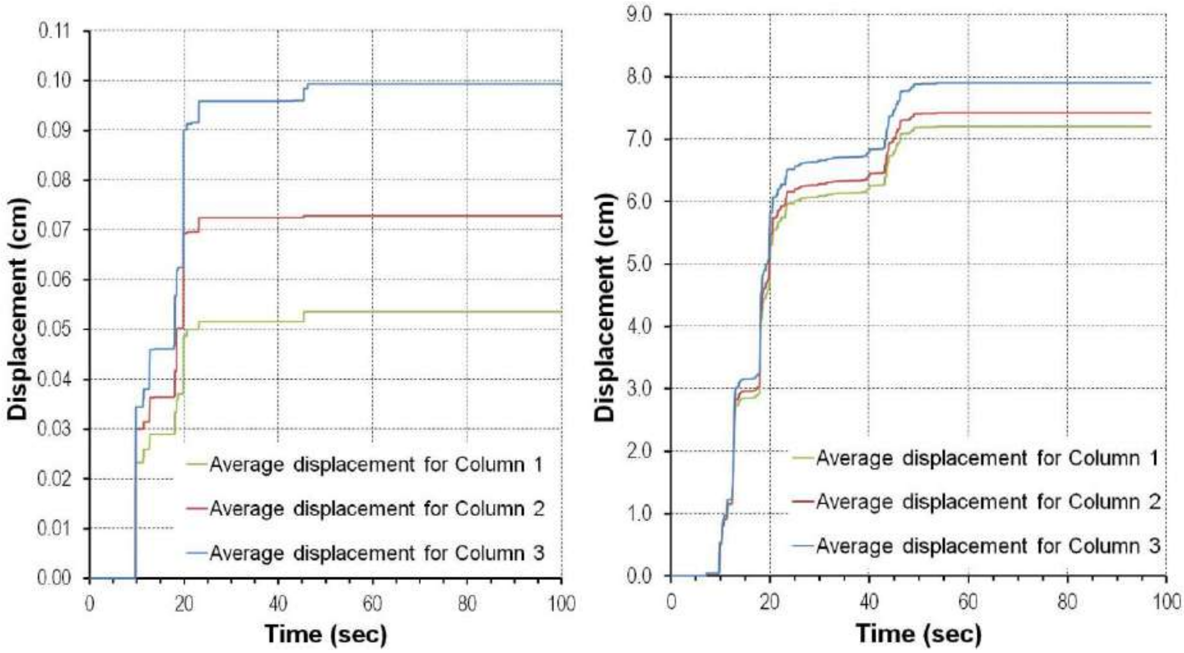



**Figure 7: Cumulative seismic permanent displacements of the Houston et al. (1987) method for 100- (L) and 475-year return-period, considering the 2001 Atico earthquake**

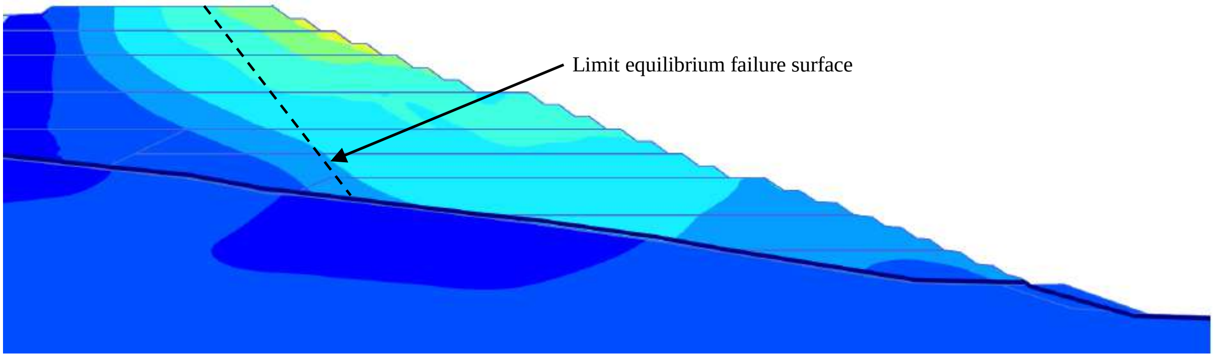



**Figure 8: Horizontal displacements in the finite element model after the 475-year return period analysis**

These results allowed verifying the optimization design of the heap leach pad, which ultimately resulted in a capacity increment of 4 $Mm^3$ with no construction cost. The Bray and Travasarou (2007) was used to further analyze other options of optimization, given that this procedure resulted in similar average values to the ones calculated from PLAXIS with a range of displacements rationally conservative.

## Conclusions

Several procedures for estimating SIPD on heap leach pads were evaluated. The rigid-block Houston et al. (1987), Makdisi and Seed (1978) decoupled, Bray and Travasarou (2007) coupled and finite-element fully-coupled procedures were reviewed and compared.

A case study was presented of a dynamic analysis of a heap leach pad, in which the calculation of SIPD was critical to optimize its design. The authors employed a large set of geotechnical information and state of art characterization of static and dynamic properties of leached ore. The parallel gradation technique, which was used to scale the large size particle ore to fit standard-size laboratory equipment, was employed to test crushed leached ore on triaxial and RCTS devices. LSDS tests were performed on low permeability soil-textured geomembrane interface with normal pressures ranging from 100 to 2,000 kPa. Non-linear shear strength envelopes were used to characterize the shear strength of both leached ore and interface.

The 2-D limit equilibrium static method was compared to the finite-element method in terms of FOS and failure mechanism geometry giving a good correlation. It was determined that the nonlinearity of the interface shear strength was fundamental to the results because of its asymptotic nature for normal stresses of 800 to 2,000 kPa, which corresponds to the normal stresses that are mainly formed at the leach pad block failure mechanism. The HS model was used to model the leached ore and several discretized cluster of Mohr-Coulomb materials were employed to model the nonlinear features of the interface. Eight seismic records were used to calculate SIPD for Houston et al. (1987) for both 100- and 475-year return period earthquakes. As for the PLAXIS analyses, two seismic records (Lima, 1974; Atico, 2001) were used that were spectral matched to the rock spectrum. In the case of the Bray and

Travasarou (2007) coupled procedure, the spectral acceleration was calculated with the site spectral acceleration for rock. Finally, all those results were compared to each other and also a Makdisi and Seed (1978) SIPD was introduced as a reference.

The results showed, in general, a good agreement and were rationally conservative for the Bray and Travasarou (2007) method when compared to the PLAXIS model. The Houston et al. (1987) results were within the range estimated with the Bray and Travasarou (2007) procedure, but in some cases underestimated the PLAXIS results. In contrast, the Makdisi and Seed (1978) results were generally much larger than the other ones. Bray and Travasarou (2007) showed consistency between the results of their method when compared to observed seismic displacements on earth dams and MSWF and concluded that the Makdisi and Seed (1978) method can yield both conservative and unconservative displacements. The results of this paper support their conclusions. Given these results, the Bray and Travasarou (2007) procedure was validated for this case study and was further extensively used in this analysis. The authors recommend the use of this method to estimate SIPD for seismic design of heap leach pads, since it involves relatively simple calculations in comparison the numerical complexity of Newmark (1965) type analysis or the use finite-element finite-difference models. However, it is important to mention that SIPD are sensitive to the fundamental period of the sliding mass and correspondent spectral acceleration, which are inputs for the Bray and Travasarou (2007) procedure. 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 FOS 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 testing interfaces for high normal stresses is highlighted due to the asymptotic behaviour of its shearing strength, which is critical for the stability assessment of heaps of over 80 m height. Similarly, more research is needed to properly define shear modulus and damping ratio curves for leached ore and low permeability soil-textured geomembrane interface, since in the case of the first one, a comparison of project-specific results showed disagreement with literature curves of gravel and sand.

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