Seismic analysis of a valley-fill heap leach pad

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Abstract

For a long time, in highly active seismic zones such as Peru and Chile, the seismic design of heap leach pads has been performed mostly by using a pseudo static approach. However, recent researches highlight the calculation of seismic-induced permanent displacements as a more rational concept for seismic design of earth structures. A range of allowable displacements for the structure can be used as design criteria, which should be selected to prevent the tearing of the liner system geomembrane during seismic events. The range of displacements for heap leach pads is usually narrower than for other earth mining structures such as tailings storage facilities or mine waste dumps. Failure of the pad’s liner system can lead to environmental damage, losses of lives and economical detriment. Additionally, in Peru, heap leach pads are usually built within narrow valleys where its three-dimensional nature can influence their seismic behavior.

This paper presents a case study of a valley-fill heap leach pad of a Peruvian mining project where its overall design was defined by its seismic behaviour. A large set of geotechnical information was used for the analyses which included state-of-the-art characterization of static and dynamic properties of leached ore and interface (liner system). The seismic analysis included one-dimensional nonlinear seismic response analysis, the use of simplified procedures to calculate seismic induced permanent displacements and two and three-dimensional dynamic analyses. Several comparisons between the results of multi-dimensional analyses were made to show the influence of the foundation soils, geometry of the valley, geotechnical parameters, among other details.

The results allowed to authors to compare different approaches to calculate seismic induced permanent displacements for a valley-fill heap leach pad. Differences between the methods used was addressed. Also, the seismic response of the heap was evaluated comparing one, two and three-dimensional analysis.
Introduction

The Peruvian mining industry operates at high altitude in the Andes, where its topography is very aggressive and unfavorable for heap leach pad (HLP) design and construction. A standard project can operate at altitudes higher than 3,000 meters above sea level, where the only place available for earth mining structures is usually narrow valleys. The design of earthworks, liner systems, solution collection systems and first lift stacking usually involve special and specific design criteria that differs significantly from the ones used in conventional HLP constructed in almost ideal conditions, such as flat terrains at much lower altitudes.

In countries such as Peru and Chile, which are subjected to strong seismic events, seismic stability analysis of HLP is paramount during design stages and is regularly performed through pseudo-static analysis and less often by the calculation of seismic-induced permanent displacements (SIPD). The approach for SIPD calculation varies from simplified methodologies to fully coupled dynamic analysis (Reyes and Pérez, 2015) and is focused on determining the magnitude of displacements induced by seismic forces in the soil-geomembrane interface of the HLP liner system. The analysis methodologies are whether from one-dimensional (1D) or two-dimensional (2D) nature; however, no previous study has assessed the influence of the three-dimensional nature of valleys for HLP on both the heap and interface dynamic response.

Based on geotechnical site investigations, advance laboratory testing and previous studies related to seismic analyses of HLP, this paper presents the seismic analysis of a valley-fill heap leach pad located in northern Peru. The analysis included 1D seismic response analysis, simplified procedures for the calculation of SIPD and 2D and 3D fully coupled dynamic analyses. The results of the evaluation were compared in order to understand the limitations and advantages of the simplified procedures and the 3D effect associated with valley-fill HLP.

Background and current practice

The process of subduction of the Nazca plate beneath the South American plate is responsible for the high seismicity that Peru experiments. Therefore, most civil and mining structures built in this country are design to endure the strong seismic loads produced by this phenomenon. In Peru, there are regional seismic studies such as the ones developed by Castillo and Alva (1993) and Gamarra and Aguilar (2009), which show the high probability of occurrence of strong seismic events, leading as a consequence to a strongly seismic-influenced overall design. These studies present isoacceleration maps for different soil types and return periods that can be used for seismic design. However, nowadays Peruvian mining
authorities demand to perform a specific seismic hazard assessment for each site, so that this literature is used as reference for conceptual design.

Up until a few years ago, pseudo-static analysis was considered as an standard to evaluate the seismic behavior of HLP design by considering a horizontal seismic coefficient ranging from a third to a half of the peak ground acceleration (PGA), as proposed by Hynes-Griffin (1984). However, this criterion was suggested for earth dams (Newmark, 1965), which are structures less sensible to SIPD than HLP or tailings storage facilities. To overcome this clear limitation, several methods to whether select an appropriate value of pseudo-static coefficient (e.g. Bray and Travasarou, 2009) or to calculate SIPD (e.g. (Makdisi and Seed, 1978; Houston et al., 1987; Bray and Travasarou, 2007) developed in the interface system were summarized and further evaluated by Reyes and Pérez (2015). Based on this study and several others, the current state-of-practice of seismic design of HLP in Peru is currently shifting towards defining a maximum allowable level of SIPD rather than using a simple qualitative pseudo-static factor of safety. SIPD are important for HLP since they tend to develop along its liner system. Recent studies performed by Kavazanjian et al. (2011, 2012) in landfills, which also have a liner system, indicate that the maximum allowable SIPD may vary from 15 to 30 cm, which would be the trigger level of displacements before geomembrane tearing, and subsequent economic and environmental damage, occurs.

While the calculation of SIPD seems a more rational and practical tool for the seismic design of HLP, it is important to understand the difference between the several existing methodologies of analysis, their limitations, advantages, influence of dynamic properties of the materials involved in the evaluations and the limitations of simplifying a 3D structure to 1D or 2D models.

Case study

The case study presented in this paper is a 120-m high HLP located at a mine site in northern Peru with a maximum capacity of almost 55 Mt. While the HLP was already been stacked with a capacity of 6 Mt, the authors were in charge of its stability verification, focusing on its seismic stability condition. In order to accomplish this, a large set of geotechnical field investigations and laboratory tests was carried out to characterize both the static and cyclic behaviour of the materials involved in the HLP design such as the soil foundation, soil-geomembrane interface of the liner system and the leached ore.

To evaluate the seismic stability of this HLP, the authors performed several analysis which included preliminary pseudo-static slope stability analysis, 1D seismic response analysis (SRA), simplified calculations of SIPD and 2D and 3D dynamic analysis; the latter of this is described in detail by Reyes et al. (2016). The following sections describe all of these analyses as well as the reasoning behind them. Figure 1 presents the plan view of the HLP and two cross-sections (1-1’ and 2-2’) which were used for all the analyses.
Field investigation and laboratory testing

The field work was focused on characterizing the foundations soils, soil-geomembrane interface and leached ore. Several samples of soil liner and geomembrane were collected in situ by removing part of the leached ore at the toe of the heap and cutting the geomembrane. On the other hand, leached ore samples were collected directly from the operating heap and their global of field particle-size distribution (PSD) curves, which included particles larger 3 in, were determined through several excavations along existing heap slopes. Additionally, several boreholes were executed at the toe of the heap to evaluate the foundation over-consolidated clayey soils. Standard penetration tests (SPT) were executed and undisturbed samples were collected. No phreatic level was detected. Finally, a complete geophysical survey was completed along the heap and foundation soils.

Using the samples collected from the operating heap, a relatively large set laboratory tests were carried out. Regarding the clayey foundation soils, drained triaxial tests were carried out on undisturbed samples which, in conjunction with geophysical tests results, provide the information necessary for the analysis.

Leached ore was subjected to additional tests, since no database is available particularly for its dynamic properties. The samples collected were reconstituted in laboratory using the parallel gradation technique. This method scaled the field PSD curve to a parallel one considering the maximum particle size allowed by the testing device, which is usually between 10 to 15 times smaller the maximum particle size of standard LO and MW. This technique 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 monotonic triaxial tests on rockfill, crushed rock and alluvial soils, respectively. The PSD curve
of the materials tested in the laboratory maintained the same coefficient of uniformity ($C_u$), PSD shape and relative density as the materials in the field but limiting the fines content to a maximum of 10%. Using this technique, monotonic drained triaxial tests were performed in a local laboratory in Lima, Peru. Additionally, the laboratory program included sets of special tests performed at the University of Texas at Austin using resonant column-torsional shear (RCTS) and cyclic-triaxial (CTX). The RCTS tests were performed in a sequential series on the same specimen with isotropic confining pressures ($\sigma'_0$) ranging from 200 kPa to 800 kPa. For each specimen, nonlinear RCTS tests were conducted at two or three $\sigma'_0$ over a shearing strain ($\gamma$) range from about $10^{-6}$ % to slightly more than 0.1%. CTX tests were conducted on these specimens at a single $\sigma'_0$ of 700 kPa for each specimen and over an estimated shearing strain range from about 0.01% to 1.4%. Further detail on these cyclic tests and others performed exclusively on leached ore and rock mine waste materials are presented by Parra et al. (2016).

Finally, two sets of large scale direct shear (LSDS) tests were performed on the low permeability soil-textured geomembrane interface: all of them tested on remoulded soil samples considering an interface consisting of the textured side an LLDPE 2.0 mm geomembrane in contact with a low permeability soil. One test was performed under normal stresses ranging from 100 to 800 kPa in a local laboratory and the other one was carried out at the TRI Environmental laboratory at Austin, Texas using normal stresses up to 2000 kPa, since most of the interface in the leach pad is subjected to normal stresses from 1000 to 2000 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.

**Static analyses**

The HLP studied is located over an over-consolidated, unsaturated clayey soil foundation. Hence, only translational failures were of concern, which provided lower factor of security (FOS) than the rotational ones. Thus the foundation soil was represented in all the analyses as a cluster with much higher strength than the interface or leached ore. The following sections briefly describe the geotechnical properties for the static evaluations which were performed before the seismic analyses.

**Static properties**

The CD triaxial tests on leached ore provided nonlinear shear strength envelopes since it was considered cohesionless with a reducing friction angle as confining pressure increases. The logarithmic tendency of developed by Leps (1970) for coarse granular materials was consistent with the results of CD triaxial tests, with a friction angle ranging from 35 to 39°. On the other hand, the nonlinear monotonic stress-strain behavior was modeled using the Hardening Soil (HS) formulation (Brinkgreve et al., 2014). The HS is an advanced model for simulating the behaviour of different types of soil, both soft and stiff
The HS formulation was calibrated with the resulting stress-strain curves of the CD triaxial tests. Another nonlinear shear strength envelope was defined for the interface. The studies published by Ayala and Huallanca (2014) and Parra et al. (2012) evidence the influence of the nonlinear behaviour of the interface for the stability analyses. The LSDS results at high normal stresses demonstrated the shear strength is nonlinear at high stresses. The Figure 2 shows the nonlinear shear strength envelopes for leached ore and interface.

![Figure 2: Nonlinear shear resistance envelope for leached ore and interface](image)

**Static analysis**

The 2D static analysis consisted initially on limit equilibrium method evaluations using the Spencer (1967) procedure on the 5 cross-sections shown in Figure 1. The resulting 2D FOS of the design cross-sections showed stability conditions lower than the permissible (minimum FOS=1.3) for translational failures, as can be seen in the Table 1.

In order to evaluate the static slope stability under a rigorous criterion, the structure was analysed with the computer program FLAC (Itasca, 2011) using cross-sections 1-1’ and 2-2’, which were chosen due to their low FOS and representativeness of the HLP. The heap conformation was simulated by 5 stages consisting of several lifts until the crest is reached. Figure 1 shows the stages defined for each cross-section. The initial stresses (horizontal and vertical) were calculated and then used in the subsequent dynamic analysis. The resulting 2D FOS obtained was very similar to ones calculated by the limit equilibrium evaluation, as can be seen in the Table 1. Figure 3 displays the resemblance of the 2D translation failure surfaces obtained from both limit equilibrium and finite difference analyses. To overcome this apparent static instability, a 3D limit equilibrium slope stability analysis was carried out which allowed the authors to determine that the 3D effect of the valley was favourable for the HLP stability. The obtained 3D FOS were higher than the minimum required for those conditions (1.40-145). Further details on this 3D evaluation are presented by Reyes et al. (2015).
Table 1: 2D FOS calculated by limit equilibrium and finite difference methods

<table>
<thead>
<tr>
<th>Cross-section</th>
<th>Method</th>
<th>Factor of safety Static</th>
<th>Pseudo-static (Tr =100 years)</th>
</tr>
</thead>
<tbody>
<tr>
<td>0-0’</td>
<td>Limit equilibrium</td>
<td>1.19</td>
<td>0.90</td>
</tr>
<tr>
<td>1-1’</td>
<td>Limit equilibrium</td>
<td>1.21</td>
<td>0.91</td>
</tr>
<tr>
<td>2-2’</td>
<td>Finite difference</td>
<td>1.22</td>
<td>0.93</td>
</tr>
<tr>
<td>3-3’</td>
<td>Limit equilibrium</td>
<td>1.12</td>
<td>0.86</td>
</tr>
<tr>
<td>4-4’</td>
<td>Limit equilibrium</td>
<td>1.39</td>
<td>1.07</td>
</tr>
</tbody>
</table>

Figure 3: Maximum shear strain-rate for the cross-section 1-1’ (above) and 2-2’ (below)

Seismic analyses

The seismic analyses of HLP initially consisted on limit equilibrium pseudo-static analysis. However, due to the instability suggested by its results, 1D SRA and simplified calculation of SIPD were completed to determine the effect of the seismic forces on the liner system. Additionally, to verify these calculations and include the effect of the HLP valley in the seismic stability, dynamic analysis in 2D and in 3D using FLAC and FLAC 3D (Itasca, 2012), respectively, were performed. Below, it is described the dynamic parameters of the materials involved, details and results of each type of analysis.
Seismicity

The seismic analysis used the uniform hazard response spectrum for 100 years return period and defined for Class B soil (rock) as a design criterion. Seismic records from both horizontal components used as input for site response analysis were obtained from published motions from Peruvian subduction earthquakes recorded in Peru. The earthquake motions from the 1970 Lima and 2001 Atico were chosen to perform the dynamic the 2D and 3D analyses. It is important to mention that the both Lima and Atico earthquake motions were recorded near the epicenter of the event, capturing their high energy content. No other earthquake motions were selected due to the limited database available for Peru. These two seismic records were rotated to the most critical direction before any processing was done. Then, they were spectral matched to the 100 years return period uniform hazard response spectrum using the SeismoMatch software, which is based in the pulse wave algorithm proposed by Abrahamson (1992) and Hancock et al. (2006).

Dynamic properties

First, based on the curves obtained by the both RCTS and CTX tests on leached ore, the normalized shear modulus and damping ratio curves for this material were determined for confining pressures of 200 and 700 kPa. These proposed curves were compared with the Menq (2003) formulation, observing a good agreement from the small-strain range up to 0.01% of shear strain. Detailed discussion of these tests results is presented by Parra et al. (2016). Additionally, geophysics survey results, performed above leached ore, were compared with RCTS report test obtaining a good agreement between the in-situ measurements and the predictions of the RCTS device. Figure 4 present the dynamic properties of the leached ore as tested in laboratory and as proposed curves for the seismic analysis.

![Figure 4: Normalized modulus reduction and damping ratio curves for leached ore.](image-url)

The dynamic properties of the interface were defined by reviewing existing information on this matter. The backbone curve for the interface was modeled based on its static shear strength, according to
the conclusion of Kavazanjian and Matasovic (1995) and Arab (2011). The damping ratio was modeled based on the cyclic shear tests on interfaces performed by Arab (2011) which show a relatively constant damping ratio value. This constant nature of the interface damping ratio is similar to the findings of Yegian et al. (1998), which only was used to determine the maximum shear modulus ($G_{\text{max}}$). It is important to mention that no cyclic shear test on the interface was developed for this paper; however, sensibility SRA analyses were carried out to analyse the inherent uncertainties of this modelling; these evaluations showed similar results for all cases.

For the foundation soil, the modulus reduction and damping ratio curves were represented by means of the Darendeli (2001) formulation, which use several parameters such as plasticity index (PI), over-consolidation ratio (OCR) and confining stress, among others. The Darendeli (2001) formulation was proposed by clayey and silty soils with a low percent of coarse grained soil. Additionally, the geophysical survey’s shear wave velocity profiles of the foundation were used in the seismic analysis.

Finally, the dynamic properties of the bedrock were assigned considering an elastic material and only as a medium to broadcast waves. Because of the translational failure does not occur through this material, the shear strains induced by the earthquake were not of importance. Table 2 presents the main properties used in the seismic analysis.

### Table 2: Main geotechnical parameters for seismic analysis

<table>
<thead>
<tr>
<th>Material</th>
<th>Static properties</th>
<th>Dynamic properties</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Cohesion (kPa)</td>
<td>Friction angle (°)</td>
</tr>
<tr>
<td>Leached ore</td>
<td>Nonlinear envelope</td>
<td>Defined based on HS model calibration</td>
</tr>
<tr>
<td>Foundation soil</td>
<td>150</td>
<td>32</td>
</tr>
</tbody>
</table>

#### 1D seismic response analysis

In order to determine a seismic response spectrum in free field conditions for the HLP, a 1D equivalent linear SRA was carried out using several soil columns and the spectral matched seismic records defined before. The computer program DeepSoil (Hashash et al., 2016) was used to perform the SRA. The analyses were performed in three different zones of HLP such as the crest, the intermediate bench and the toe identified as Zone A, B and C in Figure 1. The resulting seismic response spectra were used for the calculation of seismic coefficients and SIPD and as comparison for the dynamic analysis. Figure 5 shows the seismic spectra obtained from the 1D SRA, 2D and 3D dynamic analysis for the Lima seismic record.
Figure 5: Response spectra’s of (a) Zone A (b) Zone B and (c) Zone C for the 1970 Lima seismic record from 1D SRA, 2D and 3D dynamic analyses.

Pseudo-static analyses
Using the resulting seismic response spectrum from the SRA and maximum allowable displacement of 30 cm, a horizontal seismic coefficient of 0.07 was calculated using the Bray and Travasarou (2009). The calculated FOS from the pseudo-static analysis result are shown in the Table 1. These results suggest that SIPD higher the 30 cm are likely to develop along the liner system for almost all cross-sections. It is important to remark that, although the static 3D limit equilibrium verified that the 3D effect of the valley had an important effect of its static stability, no such conclusion can be made for a pseudo-static analysis. The reason behind this is because the seismic coefficient is defined in 2D conditions and cannot be directly use in a 3D analysis. Hence, the calculation of SIPD in a 2D fashion was mandatory in order to verify these assumptions. Only a 3D dynamic analysis, such as the performed by Reyes et al. (2016) and included in this paper, could evaluate the seismic effect of the 3D geometry of the valley.

Seismic induced displacements by simplified methods
The calculation of SIPD for operation conditions, associated with a 100 years return period seismic event, was performed using the Makdisi and Seed (1978), Houston et al. (1987) and Bray and Travasarou (2007)
methods. For the calculation, it was necessary to know the seismic response in the foundation and heap itself. For that reason, for Makdisi and Seed (1978) and Houston et al. (1987) methods, 1D SRA using linear equivalent method were performed using DeepSoil (Hashash et al., 2016) using the synthetics motions mentioned earlier. Three soil columns per section were built for section 1-1’ and 2-2’, which were composed by leached ore of 30 to 100 m, foundation soil of 45 to 80 m and bedrock of 5 m thickness. For the specific case of the Houston et al. (1987) method, the computer program D-MOD (Matasovic, 1993) was used to calculate the SIPD. Finally, the Bray and Travasarou (2007) method used the resulting seismic response spectrum in free field conditions described in previous sections. The Table 3 presents the range and average values of horizontal displacements developed along the interface.

2D dynamic analysis

In order to verify the displacements developed along the interface, a 2D dynamic analysis as carried in the software FLAC (Itasca, 2011) for cross-sections 1-1’ and 2-2’. The analyses simulated the construction process of the HLP with a 5-stage sequence until the crest is reached, as can be seen in Figure 1. The models and zones satisfy the seismic wave transmission requirements given by Kuhlemeyer and Lysmer (1973) which recommend that maximum zone dimension should be less than one tenth to one eight of the maximum shear wave length associated with the highest frequency component of the input wave for any material.

Special care was taken when modeling both the static and dynamic properties of the HLP. For the leached ore static properties, its shear strength was modeled as a nonlinear envelope and its elastic modulus were based on the HS model hyperbolic relationship of modulus and confining pressure. Regarding its dynamic properties, the Mohr-Coulomb model was used and a hysteretic behavior was included. The modeled dynamic curves are presented in Figure 4. A similar approach was followed for the foundation soils, except no nonlinear envelope was used for its shear resistance. Finally, the soil-geomembrane interface was modeled as an interface element in FLAC with a shear strength resembling the nonlinear envelope defined in Figure 2. Similarly, it shear modulus was defined using a nonlinear envelope. Regarding its dynamic properties, the Mohr-Coulomb model for the interface managed to closely represent the dynamic properties discussed earlier in the paper for soil-geomembrane contacts. No additional hysteretic damping was added. Figure 6 shows the horizontal displacements for cross-sections 1-1’ and 2-2’ resulting from the 2D dynamic analysis and Table 3 presents the results in terms of displacements along interface.
3D dynamic analysis

After 2D dynamic analyses, only one cross-section showed SIPD over 30 cm, which was considered as the maximum allowable displacement for this project. Consequently, in order to account for the 3D effects already studied in the 3D limit equilibrium static slope stability analysis of this HLP (describes in a previous section), a 3D dynamic analysis was performed using the seismic records aforementioned. The dynamic properties and analysis staged-construction set up was the same as for the 2D analysis. A detailed description of the 3D dynamic model is presented by Reyes et al. (2016). Figure 7 shows, in plan view showing only the soil-geomembrane interface, the total SIPD along the liner system from the 1970 Lima seismic record, the shape of the failure surface developed in it and the an approximation sliding direction (arrow).

Discussion

Table 3 presents the results of SIPD by the simplified methods, 2D and 3D dynamic analyses. It is important to note that while only 2 sections were presented in this paper, results of SIPD for the other cross-sections resulted in negligible displacements. The Makdisi and Seed (1978) results are presented in Table 3 in a range of displacements due to the nature of the original chart used for this method; the values presented deviate significantly form the other methods and were not taken into account during design.
The Houston et al. (1987) results are presented as average values of the soil columns used during calculation. On the other hand, a range of displacements is predicted by the Bray and Travasarou (2007) technique, which is a characteristic of its formulation. Finally, the FLAC 2D and 3D results are presented both a range and average values of displacements: the ranges show the maximum and minimum displacements developed along the interface and within the failure surface while the average values are representative of the whole failure.

![Image: Interface shear displacements for the 1970 Lima seismic record](image)

Figure 7: Interface shear displacements for the 1970 Lima seismic record

Table 3: SIPD values for 100 year return-period

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<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td>Range</td>
<td>Average</td>
<td>Range</td>
<td>Range</td>
</tr>
<tr>
<td>1-1’</td>
<td>Lima</td>
<td></td>
<td>0.4-5.0</td>
<td>20.7</td>
<td>6.0-24.0</td>
<td>1.2-19.2</td>
</tr>
<tr>
<td></td>
<td>Atico</td>
<td></td>
<td></td>
<td>10.1</td>
<td></td>
<td>11.1</td>
</tr>
<tr>
<td>2-2’</td>
<td>Lima</td>
<td></td>
<td>3.0-25.0</td>
<td>47.3</td>
<td>14.3-57.2</td>
<td>11.0-59.0</td>
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<tr>
<td></td>
<td>Atico</td>
<td></td>
<td></td>
<td>28.9</td>
<td></td>
<td>43.0</td>
</tr>
<tr>
<td>3D model</td>
<td>Lima</td>
<td></td>
<td>Range</td>
<td>5.0-30.0</td>
<td></td>
<td>~20.0</td>
</tr>
<tr>
<td></td>
<td>Atico</td>
<td></td>
<td>Average</td>
<td>5.0-30.0</td>
<td></td>
<td>~20.0</td>
</tr>
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</table>

The FLAC 2D model results in an average displacement of 10.7 cm, ranging from 1.2 and 19.2 cm for the section 1-1’, while for section 2-2’ results in an average displacement of 44.7 cm, ranging from 11.0 and 63.1 cm. The Bray and Travasarou (2007) analysis resulted in ranges of SIPD for each cross-section that were consistent with the average values from the FLAC 2D analyses. Regarding the Houston
et al. (1987) method, it yields relatively similar giving displacements to the Bray and Travasarou (2007) and FLAC analysis. However, it is clear that the Bray and Travasarou (2007) results are more consistent and reliable in comparison with the other simplified methods.

The results of the 3D analysis show that the average displacements in the interface within the failure surface are lower than the minimum required and in general lower than the maximums estimated by the other approaches. This can be interpreted as an indicative of the 3D effect of the valley, which is similar to the one evaluated in the 3D static limit equilibrium slope stability analysis of Reyes et al. (2015). Also, the analysis 3D shows that some sectors of the interface exhibit displacements very close to 30 cm, particularly when the interface is confined by only one or two lifts of leached ore at the toe of the heap. Also, this sectors match the ones where the ore exhibits also relatively high horizontal displacements, hence, influencing relative displacements in the interface. The low damping ratio of the ore may induce these displacements as well. Figure 6 shows SIPD in the leached ore, where it can be seen that the largest displacements in both the 2D and 3D occur near the toe of the heap in low-confinement areas. Both the localized interface and leached ore high displacement are consistent with the findings of Reyes and Pérez (2015).

Finally, the comparison of the 1D, 2D and 3D seismic response of the leached ore in the heap surface, as shown in Figure 5, show an evident difference between the methods that showcase the influence of the valley geometry. First of all, Figure 5c shows the response of Zone C, which is an area at the toe of the heap; there the seismic response for short periods is different for each method, with an important amplification for the 2D and 3D cases. This amplification occurs around the natural period of the leached ore in that area. Figure 5b shows a similar trend for Zone B, with a relatively similar response for the 1D, 2D and 3D methods for large period (>0.5s); however, for short periods (<0.5s) the influence slope of the heap influences a difference between the 1D and 2D responses. Also, the 3D response exhibits a larger difference, probably due to the influence of valley. On the other hand, Figure 5a (Zone A) shows a more clear difference between the response yielded by the 1D, 2D and 3D analysis. While the response between the 1D and 2D analysis for short periods is somewhat close, the 3D response resulted in a significant amplification. For long periods (>0.5s) the results of the 1D, 2D and 3D are different particularly around the periods of 0.75 to 1.5s, probably influenced by the leached ore response as well as the valley’s.

Conclusions

A case study was presented of a seismic analysis of a valley-fill heap leach pad, in which the calculation of SIPD was the key to guarantee its stability. 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 was used to scale the leached ore large size particle to fit standard-size laboratory equipment and to test crushed leached ore on resonant columns, torsional shear and cyclic triaxial devices. Large scale direct shear tests were performed on low permeability soil-textured geomembrane interface with normal pressures ranging from 100 to 2000 kPa. The results of these tests allowed modelling the dynamic properties of materials such as leached ore and foundation soil. Non-linear shear strength envelopes were used to characterize the shear strength of both leached ore and interface.

The seismic analysis focused only in translational failures; hence, the integrity of the geomembrane of the liner system was of concern. Initially, the static stability was overcome using a 3D limit equilibrium slope stability analysis (Reyes et al., 2015) since the 2D analysis showed an apparent instability. Then, pseudo-static analyses were carried out in several cross-sections showing, again, an apparent instability. As a consequence, the analysis focused on determining seismic induced permanent displacements on the soil-geomembrane interface. One-dimensional seismic response analysis and calculation of seismic induced permanent displacements by simplified methods and two and three-dimensional dynamic analysis were carried out. The amount of geotechnical information available, the use of nonlinear shear strength envelopes and advanced constitutive models were crucial to complete all of these evaluations and to allow their comparison. The Makdisi and Seed (1978), Houston et al. (1987) and Bray and Travasarou (2007) method were used as well as two and three-dimensional analysis in FLAC and FLAC 3D, respectively.

The simplified calculation and two-dimensional analysis showed that only one cross-section yielded displacements higher than the maximum allowed. In particular the Bray and Travasarou (2007) method yielded similar results to the ones calculated by FLAC 2D. The Houston et al. (1987) were also similar but with a larger variability, as can be seen in Table 3. On the other hand, the Makdisi and Seed (1978) results were not consistent with the other methods. Since only one cross-section showed an apparent instability, a 3D dynamic analysis was performed in FLAC 3D which showed a significant decrease in the displacements within the estimated failure surface. It can be concluded that, for the studied case, the valley decreases the seismic induced displacements in the interface. A similar effect is encountered for the static conditions, as evaluated by Reyes et al. (2015). On the other hand, in general, the two and three-dimensional analysis results on an increased seismic response of the heap leach pad, probably influenced by the heap slope and valley geometry. Figure 5 shows how the response varies in different sectors of the heap and for the different approaches. Then, for the evaluated heap, it can be concluded that the valley tends to increase the seismic response of the heap in surface. However, due to time limitations when performing the 3D analysis, no comparisons could be completed in other sectors of the heap to properly evaluate the response of the heap near the side valley slopes.
The authors recommend the use of upper bound results of the Bray and Travasarou (2007) formulation to calculate seismic induced permanent displacements of heap leach pads. Also, it is important that the mining industry shifts the design criterion of seismic design of heaps towards defining a maximum level of allowable displacements for the liner system’s geomembrane. Additionally, more research is needed to assess the seismic response of leached ore, which directly influence the seismic response of the heap and particularly its closure seismic design.

References


