Master's Programme in Geoengineering

Effects of blasting vibrations in heap leaching piles located in the vicinity of an open pit mine

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Abstract

The current thesis project was done in collaboration with Terrafame, which is a large-open pit mining company located in Sotkamo, Finland. Terrafame extracts and process nickel and cobalt into battery chemicals, but also supplies zinc and copper to their customers for metal processing. Terrafame utilizes a bioleaching process to extract the minerals employing one primary heap leaching pile, and four secondary heap leaching piles, which annually receive approximately 18 M tons of crushed ore material.

Open pit is the most common mining method used for the extraction of minerals near the surface. If the blast is well designed most of the energy will be used to break the rock, nevertheless some of the leftover energy will be transformed into vibrations as ground motion or as air-blast.

Peak ground acceleration (PGA) is the most used strong motion parameter, although there is a lack of correlation with structural performance during past earthquakes. Thus, other parameters will be considered, including response spectrum, and Arias intensity (AI).

Slope stability is a must in the planning, operation, and post-closure stages of heap leaching piles. The most common causes for slope failure are the change in slope geometry, the variation of the pore water pressure, vibrations, and earthquakes.

Liquefaction occurs when the cyclic stresses bring the soil to a critical state where the material's strength drops dramatically, and its structure is not strong enough anymore to resist the shear stresses.

In this case, four secondary heap leaching piles are under study. Following a logical behavior, the closest leaching pile to the open pit possesses the highest values for acceleration, velocity, and amplitude compared with the other three piles.

However, there is an uncertainty given that the lowest values for acceleration, velocity and amplitude do not belong to the farthest leaching pile to the open pit. A more accurate statement of this behavior cannot be given at this point because there are no more leaching piles in farther distances to study or more points of measure.

For the preliminary liquefaction assessment, a simplified analysis was performed for the leaching piles where the lowest values of factor of safety (FoS) against liquefaction due to blasting vibrations are in the first three meters of the piles, but none of those represent an imminent risk for the integrity of the structures.

Arias intensity has proved to be efficient as a liquefaction triggering index. In this case, the values obtained of Arias intensity in the leaching piles due to blasting vibrations are very low, which means that there is not a risk for liquefaction.

Finally, it is recommended to make a wider geotechnical campaign to characterize the piles material, and also perform a definitive geotechnical risk assessment which includes a numerical model of the leaching piles with their geometry, and an accurate analysis of the variation of the pore pressure during a ground motion.

Keywords Blasting vibrations, heap leaching pile, peak ground acceleration, Arias intensity, liquefaction

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Samuel

Dedicatory

To God and my loved Mary, mother of the sorrows, for their comfort, protection, and for covering me with love day after day.

To my dad Néstor, my mom Ana, my brother Néstor, and my tita Cármen, for being the reason for my existence, for their pure love, for being my inspiration, my pillar, my comfort, and my support during my whole life, nothing of this would have been possible without you all.

To my uncles, aunt, and cousins.

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Abbreviations

a	Acceleration that is equal to the maximum horizontal acceleration a_{max}
AI	Arias intensity
AI95	Arias intensity when 95% of the total energy has been dissipated
BR	Bedrock
BR-E	Bedrock east
BR-W	Bedrock west
C_E	Energy ratio correction factor
C_N	Overburden correction factor
CRR	Cyclic resistance ratio
CRR ko	Cyclic resistance ratio corrected by anisotropy
CRR ko=1	Cyclic resistance ratio for 1 atm (isotropy)
CSL	Critical state line
CSR	Cyclic stress ratio
Φ	Friction angle in degrees
FoS	Factor of safety
g	Acceleration of gravity
ɣ	Total unit weight of the soil
ɣ'	Effective unit weight of the soil
HLP-1	Heap leaching pile 1
HLP-2	Heap leaching pile 2
HLP-3	Heap leaching pile 3
HLP-4	Heap leaching pile 4
HP	Heap leaching pile
km	kilometer
Ko	Coefficient of lateral earth pressure
L	Longitudinal plane
m	meter
MP	Measurepoint
N	Measured blown count for SPT
N 1	Measured blown count for SPT (Ohsaki, 1959)
N 2	Measured blown count for SPT (Japan Road Association, 1990)
N 3	Measured blown count for SPT (Dunham,1954)
N. AV	Measured blown count for SPT- average value
N_{60}	"N" blow count for an energy ratio of 60%
$(N_1)_{60}$	N_{60} corrected by overburden
OP	Open pit
P	Horizontal seismic force acting on the soil column
PGA	Peak ground acceleration
PGV	Peak ground velocity

P-Wave	Primary or longitudinal wave
r	Reduction factor
R-Wave	Rayleigh wave
σ	Total vertical stress at bottom of the soil column
σ’	Effective vertical stress at the bottom of the soil column
SPT	Standard penetration test
SSR	Seismic stress ratio
S-Wave	Secondary or shear wave
T	Transverse plane
t_d	Significant duration
Ts	Maximum shear stress
Tv	period of vibration
V	Vertical plane
W	Weight
z	Depth below ground level

1. Introduction

1.1. Background

Mining sector blasting operations generate ground vibrations that travel through the soil layers and bedrock, those vibrations have become a problem because of the effects they cause on the environment and surrounding area. A brief explanation by Taqieddin (1986) says that the seismic waves are generated during the blasting process as the result of the pressure pulses coming from the borehole's wall while it is expanding and contracting. Those pulses generate two basic waves: 1) Body Waves, which travel through the interior of the ground; and 2) Surface Waves, which are produced when the body waves impact a free surface.

According to Zhi-qiang et al., (2018), only from 20% to 30% of the total energy produced during blasting is focused into fragment and displace the rock mass, while the remaining percentage (70% to 80%) is dissipated in other forms such as noise, fly rock, vibrations, among others. Therefore, the stability of open-pit excavations, tailing dams, leaching piles, underground openings, belt conveyor systems, pipelines, and superficial structures, are some examples of facilities that need to be considered in any blasting operation since excessive levels of ground vibration may cause damage or even completely close down such operations (Taqieddin, 1986).

As this thesis will be focused on the effects of the blasting vibration over the secondary leaching piles, other types of structures will be disregarded. A heap leaching pile is a structure made of crushed rock material used as a mineral processing technology where various chemical solutions are leached from the top to extract valuable minerals.

Since the piles are mainly composed of cohesionless granular material and may become locally saturated from leach solution irrigation, the potential for liquefaction exists and is often considered during the design phase. As there is not wide information about heap leaching piles yet, reliable factors of safety (FoS), laboratory testing, and field observations are under constant judgment to provide new recommendations (Thiel & Smith, 2004).

As safety is indispensable in Terrafame mine, with this thesis, the company started to investigate the behavior of the vibrations coming from the open pit blasting as a possible source of instability for the leaching piles.

1.2. Motivation

Terrafame, on its constant mission to ensure the safety of its daily operation process is looking to characterize the current behaviour of their secondary heap leaching piles under the effects of blast vibrations.
This concern surged due to the risk of liquefaction in the secondary heap leaching piles (highly saturated structures) caused by their interaction with the blast vibrations coming from the open pit operation.
Liquefaction is a process that can take place in loose, cohesionless, saturated undrained soils when those are exposed to very rapid shear forces, (e.g., blasting and earthquakes) then, the material tries to densify, as a result, there is an increment of pore pressure, the effective stress decreases, leading to a dramatic drop of the strength in the soil.
This liquefaction can produce dramatic effects related to tremendous instabilities known as flow failures characterized by a sudden origin, their speed to develop, and the large distances that the liquefied material could reach.
Some other consequences due to the collapse of leaching piles could include the loss of equipment and machinery, discharge of the leaching solution in the surrounding area reaching in some cases bodies of water, damage to the infrastructure (including roads, ponds, pipes, among others), and above all, put risk on the personnel.

1.3. Research questions

- What is the behaviour of the heap leaching piles and bedrock while being affected by the induced vibrations from the blasting process in the open pit?
- What are the differences in the behaviour of the bedrock and the heap leaching piles during the vibration process caused by the blasting process in the open pit?
- What is the risk that the vibrations from the blasting process represent for the stability of the heap leaching piles?

1.4. Scope

This thesis aims to describe the behavior of the four secondary heap leaching piles and of the bedrock as a consequence of the blasting process in the open pit. At the same time, it is important to mention that this is a preliminary investigation due to the limited amount of information regarding the vibration data, as well as of the geotechnical properties of the heap leaching piles.

The response of the leaching piles will be assessed based on the measurements of vibrations performed in seven different locations in the mine. Additionally, a comparison of the behavior of the four secondary leaching piles will be carried out to determine their differences.

Having registered the vibration of the bedrock in two different locations, and in the open pit, it will be possible to see the differences in the response of the vibrations in the bedrock, and the leaching piles under the effects of the same blasting processes.

A preliminary liquefaction assessment of the leaching piles will be carried out using a simplified method. Besides, extra suggestions will be proposed as part of a future definitive geotechnical evaluation.

It is also relevant to say that this thesis will not cover the definitive geotechnical slope stability model or definitive geotechnical liquefaction risk assessment.

2. Literature review

2.1. Open pit mining

Open Pit is the most common mining method used for the extraction of minerals with a direct cone-shaped intervention in the surface producing an open pit (See Figure 1). This method is applicable when the mineral ore is near the surface, so, the first stage is about removing the soil and rock that is on the ore commonly through drill and blasting. The broken material resulting of the excavation of the pit is loaded and transported to the crushers, where the ore processing starts (Piro & Lipkina, 2020).
Open pit mines consist of a series of benches that are bisected by mine access and haulage roads that are angling down from the surface towards the bottom. Ensuring safety is critical to keep the pit walls stable until the closure of activities within the open pit, factors such as rock strength, faults, joints, and fractures are key to consider when the slope stability and slope angle determination are being carried out (Soni, 2021).

Figure 1. Open pit Terrafame (Chávez ,2023)

2.2. Effects of open pit blasting

Blasting is a process used when the conditions are not feasible for another excavation method. The blasting process requires a determined number of holes whose location and disposition are previously planned and calculated (space and burden) as part of the blasting design to release a certain amount of material volume. Once the holes have been drilled, they are charged with explosives and a detonator, and then covered with stemming

material, but here it is important to mention that not all holes are charged in the same way (geological and physical conditions also determine the charge weight in each hole).
Nicholson (2005) mentions that in a very short period of time after the detonation there is a release of very high temperature and high-pressure gases that expand rapidly carrying high forces overcoming the confining forces coming from the surrounding rock mass. This mentioned process is the one that crushes and fractures the surrounding rock until after some distance the destructive force coming from the blasting process is no longer greater than the rock strength, then the excess of energy not used in shattering the rock is transferred to the elastic zone and a disturbance (known as shock wave or ground vibration) propagates further than the blasting site.
A blasting carried out on the surface will always cause some environmental effects that usually discomfort and even could be dangerous for the human population in the surrounding area. The intensity or severity of those effects depends on a series of factors that could include the distance to the blasting site, safety measurements (use of cover mantles), powder factor, length of boreholes, and steaming length, among others.
Some environmental impacts derived from an open pit blasting that concern humans are fly-rock, ground vibration, air shock waves, effects on the water table, effects on water quality; while noise, dust, and nitrogen emissions concern more to individuals who are in close proximity to the explosion (Hänninen, 2023).

2.2.1. Blast induced ground motion

After a seismic event either natural as earthquakes or induced as blasting, energy is released and the ground shakes due to resultant seismic waves traveling through the earth's interior.
With blasting, a large amount of energy is released. If the blast is well designed most of the energy will be used to break the rock, nevertheless some of the leftover energy will be transformed into vibrations as ground motion or as air-blast. Although air-blast is the most noticeable effect of the blasting process (including noise effects), ground motion is the main vibration resulting from blasting (U.S Department of the Interior, 1987).
The Blasting Guidance Manual (1987) displays the variable factors and their influence over the ground motion during a blasting process (see Table 1).
The variable factors are divided into two branches, first, the ones within the control of mine operators, so, those could be managed in a way to minimize their effects, and second, the ones that are not within the control of the operators, hence those are not that "easy" or almost impossible to handle.

Table 1. Factors that influence ground motion (Blasting Guidance Manual, 1987)

Variables within the control of mine operators	Influence on ground motion		
	Signif.	Moderately signif.	Insignif.
1. Charge weight per delay	x		
2. Delay Interval	x		
3. Burden and spacing		x	
4. Stemming (amount)			x
5. Stemming (type)			x
6. Charge length and diameter			x
7. Angle of borehole			x
8. Direction of initiation		x	
9. Charge weight per blast			x
10. Charge depth			x
11. Bare vs. covered primacord			x
11. Charge confinement	x		
Variables not in control of mine operators			
1. General surface terrain			x
2. Type and depth of overburden	x		
3. Wind and weather conditions			x

The ground motion is a wave motion that spreads outwards from the blasting place through an elastic medium that could be either the ground or rock. This medium is composed of uncountable particles that acquire a random oscillatory motion when a disturbance process occurs (blasting), generating the waves in the way that the particles can transmit the energy successively to the next. In this process, there is also a loss of energy when the vibration waves spread outwards diminishing their intensity, while the particles start gradually to return to their rest position.

The blasting process generates three types of waves: P-waves (Primary or Longitudinal), S waves (Secondary or shear), and R- waves (Rayleigh), where each one of those possesses three dimensions with complex travel paths, those travel paths are influenced by many factors that include voids, fractures, distance, geology, topography, among others (Ainalis et al., 2016). Additionally, the particle motion generally is classified into surface (R waves) and body waves (R and S waves), being the ground motion the combination of all these waveforms as shown in Figure 2.

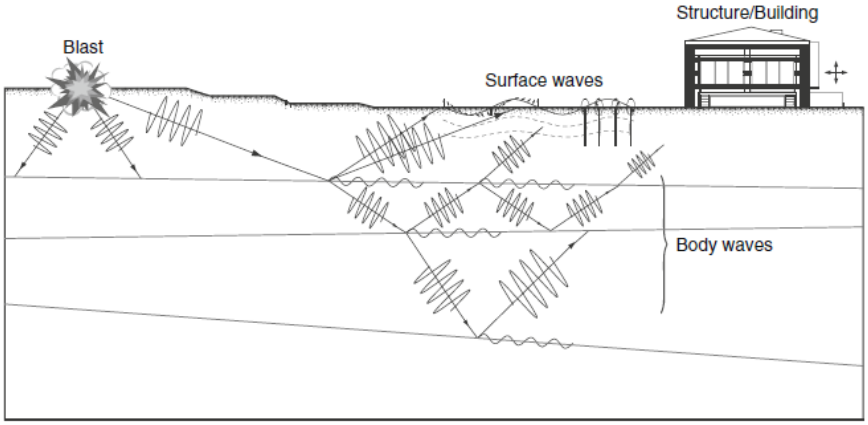


Figure 2. Surface and Body Waves (Ainalis, 2016)

The properties of the three types of waves described by Ainalis et al., (2016) are given in Table 2.

Table 2. Types of waves from blasting and their properties (Ainalis et al., 2016)

Wave	Type	Particle Motion	Relative Speed	Energy from Input [%]
P	Body	Longitudinal	Fastest	7
S	Body	Perpendicular	Between other two	26
R	Surface	Elliptic and retrograde	Slowest	67

2.3. Measurement of blast induced vibrations

Amplitude: It is the distance over which a particle moves when it is affected and set in motion by a seismic wave. It is not usually used as the sole parameter for ground motion.

Velocity: It is the speed reached by a particle during its oscillatory motion, being the parameter considered as the most relevant when talking about the measurement of structural damage possibility. It is important not to confuse peak ground velocity (PGV) with the propagation velocity which is the velocity at which the wave travels through the earth. An important fact to distinguish between those velocities is that PGV has very slow velocities in the range of mm/s, on the other hand, the propagation velocity is very fast, reaching even thousands of meters per second.

Acceleration: It is the rate of variation of the velocity of a vibrating particle when it is oscillating about its rest position. Acceleration alone is not a very used parameter in blasting vibration studies, unless some machinery could

be exposed to the vibrations because electromechanical devices sometimes could specify a maximum acceleration level.
(U.S Department of the Interior, 1987).

2.3.1. Planes to measure blast vibrations

The planes used to measure the random behaviour of the particles of the soil affected by the vibrations are longitudinal, vertical, and transverse (Figure 3). Where:

- Longitudinal: Measured in a direct line horizontally from the measurement point towards the blasting site.
- Vertical: Measured vertically perpendicular to the longitudinal and transverse planes.
- Transverse: Measured horizontally perpendicular to the longitudinal plane.

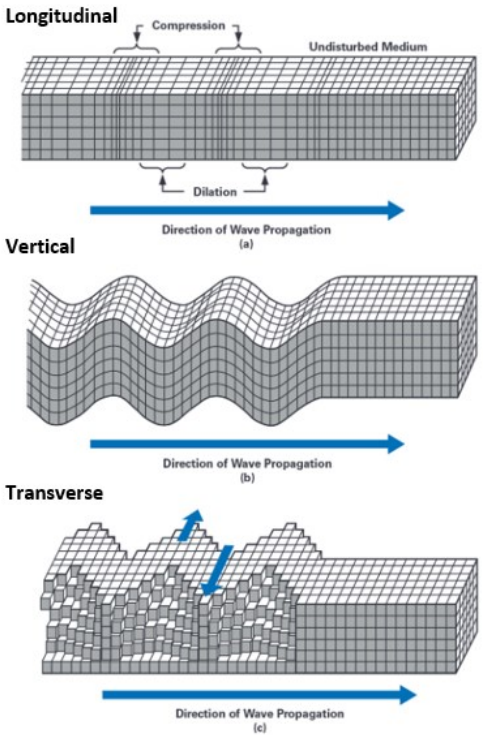


Figure 3. Component planes of vibrations (Based on Bolt, 1993)

2.4. Strong motion parameters

The ability of a single parameter to measure potential damage depends on which features of motion is capable to reflect. So, most of the times, it is recommended to correlate various parameters to obtain a clearer and wider vision of the situation.
Campbell (1985) points out that peak ground acceleration (PGA) is the most used strong motion reference parameter, although there are some criticisms on this usage due to a lack of correlation with structural performance during past earthquakes. This has led to the use and investigation of other parameters including peak ground velocity (PGV), response spectra, and Arias intensity (AI) that have been considered in this thesis, but also others such as Fourier spectra, power spectra density, and spectrum density could be employed. As a note, in the case of a liquefaction analysis, the maximum horizontal acceleration needs to be considered as part of the horizontal seismic force acting on the soil.
Since the total duration of a ground motion is not the interval of time between the start and the end of the accelerogram. In this thesis will be utilized the criteria stated by Husid (1996), which states that the significant duration of a strong motion is the interval of time during which the 5% to 95% of the accumulated energy is transmitted through the ground.

2.4.1. Arias intensity (AI)

Arias intensity represents the strength of the ground motion per second and has proven to be efficient in several applications, including the prediction of earthquakes-induced slope failure, damage in structures, and liquefaction triggering (Bahrampouri et al., 2021).
Kayen & Mitchell (1997) state that Arias intensity is highly reliable as an approach to assess the potential for liquefaction in soils during earthquakes, this is based in the seismographs of the horizontal components of ground motion, besides, it is an appropriate measure to correlate with energy-based field penetration test methodologies such as standard penetration test (SPT).
The advantages of using Arias intensity as a measure for ground motion severity in a liquefaction assessment are that the Arias intensity value is obtained from the integration of the entire seismograph waveform, so, it incorporates both the amplitude and duration elements of a ground motion.
Kayen & Mitchell (1997) also provided more information about real liquefaction cases in the USA and Japan related to Arias intensity values in the surface (I_h) (See Table 3).
Arias intensity is a value measured when 95% of the energy has been dissipated, to distinguish it from the concept of Arias intensity (AI), the

quantitative magnitude will be called Arias intensity 95 (AI95) from now on.

Table 3. Cases of liquefaction in the United States and Japan with Arias intensity (Based on Kayen & Mitchell, 1997)

Earthquake	Year	Site	Seismograph	I_h [m/s]*	Liquefaction
Hills Elmore Ranch	1987	Wildlife Liquefaction Array	Wildlife Surface	0.44	No
Loma Prieta	1989	Treasure Island	Treasure Island	0.54	Yes
Loma Prieta	1989	Bay Farm Island Lagoon	Alameda NAS	0.8	Yes
Loma Prieta	1989	Coyote Creek	Milptias	0.69	No
Miyagi Ken Oki	1973	Arahama	Sendai Kokutetsu	0.14	No
Miyagi Ken Oki	1978	Nakamura 4	Sendai Kokutetsu	0.42	Yes
Chiba-ken Chubu	1980	Owi Island No1	Owi Island No1	0.106	No
*Ih: Surface Arias intensity					

2.4.2. Site response spectrum

A response spectrum is a practical method to characterize the movements of a terrain and its effects over the structures. The response spectrum gives a convenient way to summarize the maximum response of the linear systems of one freedom degree to a particular component of the terrain movement (Chopra, 2014).
In this thesis, the site response spectrums were computed with the software SeismoMatch v2024 using accelerograms as input. SeismoMatch calculations are based on "An Improved method for nonstationary Spectral Matching" developed by Al Atik & Abrahamson (2010).

2.4.3. Site design spectrum

A design spectrum is a representation of the characteristics of the last movements registered at the site, typically incorporates the ground motion behavior in a diversity of earthquake scenarios affecting the site and does not represent any specific earthquake scenario.

It was thought to design new structures and assess the seismic safety of the existing structures during a ground movement that could be the product of an earthquake or a blasting (Chopra, 2014).

2.5. Heap leaching piles

Arizona Mining Guidance Manual (2005) describes a leaching pile as a mining facility constructed with crushed ore material deposited in a heap (Figure 4), where a leaching solution is irrigated from the top surface, to percolate through the granular material to extract the desired minerals.

Figure 4. Heap leaching pile - Terrafame mine (Chávez, 2023)

Figure 5 is a schematic of a solution distribution system used to spread the lixiviant to the pile generally from the top part, besides, it is also exhibited the solution collection system that passed through the heap and contains the valuable metals in it.

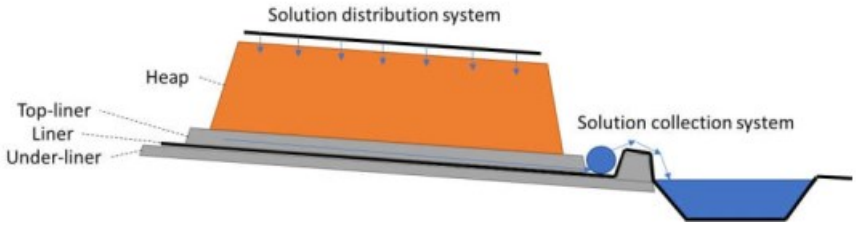


Figure 5. Simplified heap leach structure (Arpalahti, 2021)

Arpalahti (2021) mentions that the solution inside the heaps has two mechanisms to flow: By gravity (downward flow), and by capillary forces,

that will later spread laterally in the heaps. Figure 6 explains the flow process of the solutions in a heap leaching pile.

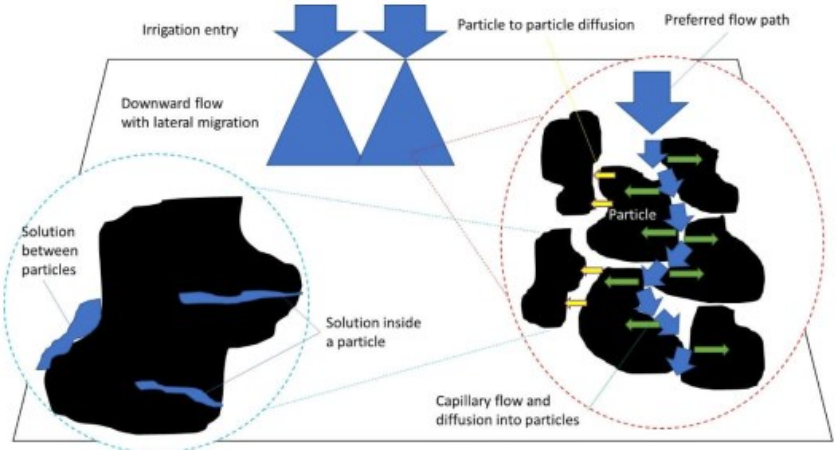


Figure 6. Illustration of solution flowing in a leach heap (Arpalahti, 2021)

2.5.1. Geotechnical risks of heap leaching piles

Table 4 proposed by Thiel & Smith (2004) presents a summary of the key geotechnical concerns related to heap leaching piles that are advised to have in mind when planning, designing, and working daily with leaching piles.

Table 4. Key geotechnical concerns of heap leaching piles (Thiel & Smith, 2004)

Performance Area	Key Concerns
Slope Stability	Global and deep-seated failures due to extreme heights and base pressures
	Sliding blocks stability along geomembrane interfaces
	Effects of active leaching with elevated degrees of saturation
	Long-term chemical and biological degradation of ore
	First-lift stability affected by lift thickness (5 m to 50 m) and stacking direction
Liquefaction	Earthquake-induced failures
	Possible static liquefaction flow-slides
	Tropical installations can have large surplus water balances
	Designs include interim catch benches and temporary caps
	Phreatic levels range
Liner	Coarse rock "overliner" systems
Durability & Leakage	Extreme pressures caused by weight of the heap and the equipment
	Durability against chemical attack
	Valley fill systems create very high solution levels

2.5.2. Slope stability

Slope stability check is a must when the use of a natural slope or the construction of a soil structure is planned. It is essential to be aware of the soil properties and the external factors acting over the slope to be able to ensure the stability and safety of the project.
A failure in an inclined surface takes place when the ground's own strength is not enough to keep the stability, which means basically that in the slip surface the ratio between the soil strength and the overturning forces is unfavourable.
In the geotechnical design, the stability in natural conditions needs to be sufficient before starting a construction, then it is necessary to be sure that the changes that will be produced, the construction won't cause any failure. Many factors should be considered during the geotechnical design such as various loading situations (including the machinery loads), chances in weather conditions like heavy rain, changes in groundwater level, freezing and thawing, and vibrations, among others.
The most common causes of failure are the change in the geometry of the slope, the variation of the water level (pore pressure variation), vibrations, and earthquakes (Korkiala-Tanttu et al., 2023).

- A change in the geometry (surface) of the slope is mainly about excavations, fills, or wave erosion. It is important to mention that the most common fatal accidents occur in relatively shallow depths (2 m to 5 m), but also the addition of unexpected new loads causes failures.
- Variation in the water level is directly related to the rising of pore water pressure (The increase of pore water pressure produces the reduction of the shear strength in the structure of the soil). Some structures such as channels, and dams could retain huge amounts of water causing a heavy saturation of the surrounding areas, additionally weather conditions (e.g., heavy rain, frost-melting processes, long dry seasons), and construction actions can also derive an important change in the water table.
- Vibrations and Earthquakes can also lead a failure, especially when they are combined with a saturated soil mass because liquefaction could be triggered.

2.5.3. Liquefaction

Liquefaction involves soil deformations in loose, cohesionless, saturated undrained soils, caused by monotonic, transient, or repeated disturbance. When a rapid loading acts over a saturated undrained soil, it tries to densify

causing an increment in the excess of pore pressure and a decrease in the effective stress, leading to a reduction in the ground shear strength.
Liquefaction occurs when the cyclic stresses bring a critical state to the soil, where the material's strength drops dramatically, and its structure is not strong enough anymore to resist the shear stresses. This liquefaction can produce dramatic effects related to tremendous instabilities known as flow failures characterized by a sudden origin, their speed to develop, and the large distances that the liquefied material could reach (Rocscience Inc., 2021).
According to Thiel & Smith (2004), seismically induced liquefaction is limited to 20 m depth because further than that the confining pressures are great enough to reduce the risk or susceptibility for a failure.
Besides, liquefaction can be triggered also under static conditions, especially in the mining industry. Additionally, as an important fact, for liquefaction, it is required that the material possess a minimum saturation degree equal to 85%, hence.
Table 5 based on Thiel & Smith (2004) extends a series of indicators which increase considerably the flow slide susceptibility in heap leaching piles.

Table 5. Indicators for flow slide susceptibility in heap leaching piles (Based on Thiel & Smith, 2004)

Parameter of characteristic	**Susceptibility value (approximate)**
Height	If the height is >= 100m
Foundation slope	If the slope is >= 15 degrees
Inter-bench slopes	If the slope is near to the angle of repose
Saturated permeability of ore	If the permeability of ore is<= 1x10-2 cm/s
Saturation (at any point in the heap)	If the saturation of the material is >= 85%
Other factors:	There is no toe support in the pile
	Finner material near the base
	High level of water
	Impermeable layer at base

2.6. Risk assessment of the heap leaching piles

The stability of a heap leaching pile always must be considered because a failure in this type of structure could produce great damage in the surrounding area. The assessment of the stability in leaching piles is carried out mainly based on the evaluation of shear strength of the piles under static and seismic load conditions.

Stability evaluations should be driven through the analysis during the construction stages, the maximum planned phreatic surfaces (that could result in a very sensitive destabilizing factor within the structure of the leaching pile), and the ultimate pile height.
If the heap leaching piles are constructed and planned to be in a seismic zone, the design needs to be based on the maximum probable earthquake (MPE) which is defined as the maximum earthquake that is likely to occur during a 100-year interval (80% probability of not being exceeded in 100 years) and shall not be less than the maximum historical event.
Where human life is potentially threatened, the maximum credible earthquake (MCE) should be used. MCE is the maximum earthquake that appears capable of occurring under the presently known tectonic framework.
Recommended design criteria for dynamic stability includes a factor of safety (FoS) for pseudo-static analyses and allowable deformations for the deformation analyses. For pseudo-static analyses, the factor of safety (FoS) should be equal to or greater than 1.1 using conservatively assumed material strengths. Where a site-specific testing is performed for material properties, a factor of safety (FoS) of 1.0 can be used. A detailed deformation analysis should be performed if these FoS criteria are not met and should also be performed in cases where: There is a high potential for pore water pressure buildup and associated significant strength loss, or a failure could result in loss of life or major environmental impact (Arizona Department of Environmental Quality, 2005).
Table 6 and Table 7 are based on the Best Available Demonstrated Control (BADCT) from the Arizona Mining Manual (2005) recommendations for static and dynamic stability criteria.

Table 6. Dynamic stability design criteria (Based on Arizona Mining Manual, 2005)

Dynamic stability design criteria	
Facility	**Design criteria**
Heap Leach Pile	• For final construction stages: - Computed pseudo-static Factor of safety (FoS): 1.1[(2)] without testing. - Computed pseudo-static Factor of safety (FoS): 1.0[(2)] with testing. • For intermediate construction stages: - Computed pseudo-static Factor of safety (FoS): 1.0[(2)] with or without testing.

	and/or (4)
	• Liners and covers: - Deformations of 30 cm, without geomembranes. (5) - Deformations of 15 cm, with geomembranes. (5) • Covers that are maintained: - Deformations of 30 cm. (5)
(2) Applicable only when material types involved (e.g., clayey soils or large, coarse rock fragments) do not exhibit high potential for pore water pressure buildup and associated significant strength loss under loading. (4) For conditions with a high potential for pore water pressure buildup and associated significant strength loss deformation analyses must be completed. Also, if loss of life or major environmental impacts are potentially imminent under failure. conditions deformation analyses should be performed. (5) Larger deformations may be acceptable if engineering evaluations are provided to demonstrate that they will not jeopardize containment integrity.	

Table 7. Static stability design criteria (Based on Arizona Mining Manual, 2005)

Static stability design criteria	
Facility	**Minimum required factor of safety**
Heap Leach Pile	Factor of safety (FoS); 1.5 Without testing
	Factor of safety (FoS): 1.3 With testing
Testing refers to site specific testing of material shear strengths and/or liner interface strengths and quality control testing (e.g., moisture, density, grain size) during construction. The testing program should establish drained shear strength parameters for long-term (static) stability analyses and, where appropriate, undrained shear strength parameters for rapid loading conditions (e.g., earthquake or rapid drawdown).	

When determining the factor of safety (FoS) against liquefaction it is important to also consider the next factors:

- Soil type: As the heap leaching piles are made of a cohesionless gravelly sand moraine (srHkMr), it is imperative to be aware that cohesionless soils such as fine to medium sands, and low plasticity sands are the type of soil most susceptible to liquefaction during an earthquake.
- Ground water table: As there is constant irrigation of solution from the top surface of the leaching piles, an important level of liquid is expected within the piles. Hence, it is important to remember that for liquefaction the soil must be underwater, therefore, if an increment in the solution level is expected, anticipated actions must be taken adequately.

- Engineering judgement: It is important to be clear that the Cyclic Resistance Ratio (CRR) and the Seismic Stress Ratio (CSR) analysis is only an approximation to the real situation, so, depending on the requirements and importance of the project, it is imperative to look for more accurate solutions to bring more efficient, trustable, and close answers to the real situation. For example, utilizing a software and an adequate geotechnical model which will be able to calculate the variation of the pore water pressure and the volume change of the cohesionless material during the time.

3. Methodology

3.1. Equipment and data collection

The devices used to register the ground motion in the heap leaching piles were an Infra mini data logger (Table 8) and a V12 sensor triaxial geophone (Table 9), both manufactured by Sigicom.

Table 8. Infra mini data logger features

Infra mini data logger		
Manufacturer	Sigicom	 (Sigicom, 2024)
Features	• Used to monitor construction activities, rock blasting, train, traffic, road traffic, and vibrations in buildings, among others. (5-300 Hz) • Complete digital data logger • High-capacity data storage (memory card) • Flexible communication capabilities (Cellular communication to send information)	

Table 9. Infra V12 triaxial geophone features

Infra V12 triaxial geophone		
Manufacturer	Sigicom	 (Sigicom, 2024)
Features	• Used to monitor construction activities, blasting, train traffic, road traffic, vibration in buildings, among others. • Counts with vibration sensing elements sensitive in three directions X, Y, and Z. • Filtering, signal processing, and detection are done digitally in the geophone.	

FORCIT CONSULTING was the company in charge of measuring the induced vibrations at Terrafame Mine. On 2023, FORCIT used seven points of measure in a period from June 22nd to November 7th where four points

were installed in the heap leaching piles (one for each leaching pile), two points in the bedrock (east and west), and from the 1st of September 2023 until November 7th, one measuring point was placed in the open pit. Table 10 summarises the location of the equipment in the field, as well as Figure 7.

Table 10. Location and measurement period for data collection devices

Location	Label	Year	Period	
Open pit	Open pit	2023	September 1st	November 7th
Bedrock-East	BR-E	2023	June 22nd	August 31st
Bedrock-West	BR-W	2023	June 22nd	November 7th
Heap leaching Pile 1	HLP-1	2023	June 22nd	November 7th
Heap leaching Pile 2	HLP-2	2023	June 22nd	November 7th
Heap leaching Pile 3	HLP-3	2023	June 22nd	November 7th
Heap leaching Pile 4	HLP-4	2023	June 22nd	November 7th

Figure 7. Location of points of measure for Leaching Piles, and Open Pit (Google Earth, 2024)

3.2. Geotechnical characterization of the leaching piles

After an examination of the crushed ore material samples, Pöyry Finland Oy (2020) determined that the color of those vary from light green to dark green (green samples are mostly moist and loose mainly located on the surface), and grey samples are mostly dry although they can be moist too. The constitutive granular material of the leaching piles corresponds to a gravelly sand moraine (srHkMr), where the distribution of the particle size

is: 40% of gravel (<2 mm) and approximately 28% of fine materials (<0.063 mm), as it is presented in Figure 8.

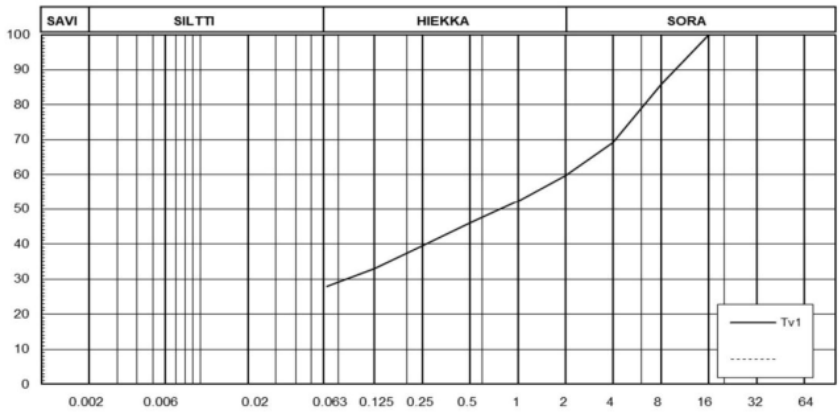


Figure 8. Gradation curve for the leaching pile constitutive granular material (Pöyry Finland Oy, 2020)

3.3. Characterization of the heap leaching piles using the Standard Penetration Test (SPT)

Equations 1, 2, and 3 proposed by Ohsaki (1959), Japan Road Association (1990), and Dunham (1954) respectively were used to find out the number of blows related to the Standard Penetration Test (SPT) based on the friction angle (Φ) of the cohesionless material. The number of blows corresponding to the SPT will be called "N", and it is important to be aware that is also an indicator of the shear resistance of a soil, as well as the static-dynamic in situ test results.

$$N = N1 = \frac{(\Phi - 15)^2}{20}$$

Eq. 1

$$N = N2 = \frac{(\Phi - 15)^2}{15}$$

Eq. 2

$$N = N3 = \frac{(\Phi - 20)^2}{12}$$

Eq. 3

The amount of energy delivered during the blowing process in the SPT is between 55% to 60% maximum (theoretically) using a hammer of 63.5

kilograms with a drop of 76.2 cm. Seed et al., (1984) recommended adopting N_{60} value as a standard that represents the blow count for an energy ratio of 60%, and can be calculated as follows (Equation 4):

$$N_{60} = N * C_E$$

Eq. 4

Where:
N= Measured blown count for SPT
C_E= Energy ratio correction factor

Additional correction factors could also be used for N_{60} value, but on this occasion, only the energy ratio correction will be used because the SPT method was not directly carried out in situ.
Seed et al., (1984) proposed empirical values for C_E presented in Table 11.

Table 11. CE proposed empirical values (Seed et al., 1984)

Hammer type	**C_E**
Doughnut hammer	0.5-1.0
Safety hammer	0.7-1.2
Automatic hammer	0.8-1.3

To find out the N_{60} value a C_E equal to 0.8 will be used due to the type of hammer and standing on the safest side of the calculation taking the lowest value due to the uncertainty of the process.
Idriss & Boulanger (2008) state that SPT penetration resistances increase with the depth of the test, which means that the values obtained from different places and depths cannot be directly compared with each other unless they were measured under comparable conditions of vertical effective stresses.
Then, an overburden correction factor C_N is used to correct the penetration resistance values N_{60}, and to simulate their equivalent value as if they would have been obtained under a vertical effective stress equal to 1 atm. Equation 5 shows how to calculate $(N_1)_{60}$.

$$(N_1)_{60} = C_N * N_{60}$$

Eq. 5

Cubrinovski & Ishihara (1999) mentions a function proposed by Meyerhof (1957) used by the Japanese Design code of Highway Bridge Foundations to estimate the liquefaction resistance of sandy deposits (Equation 6) to obtain C_N. To use Equation 6, the values for σ'_V must be in kPa.

$$C_N = \frac{1.7}{0.7 + \frac{\sigma'_V}{98}}$$

Eq. 6

3.4. Liquefaction preliminary assessment

This liquefaction preliminary assessment is based on the literature presented by Prasad (2011) and Idriss & Boulanger (2008), and its objective is to carry out a "simplified method" to ascertain if the material from the leaching piles possesses the potential to liquefy during a seismic event, referring more precisely to the ground motion caused by the blasting process at the pit.
For this analysis it will be considered a soil column with a unitary width and length, besides, it will be assumed that the soil column will move horizontally as a rigid body in response to the maximum horizontal acceleration (a_{max}) obtained from the registers belonging to the blasting processes.
It is relevant to mention that this method works well for depths of about 20 m- 25 m, since where the uncertainty to obtain the parameters increases, and special conditions for a further (greater depths) and more detailed analysis needs to be performed.

Equation 7 refers to the second Newton's law, while Equation 8 is the equivalence of the mass based on the weight and gravity force.

$$P(force) = mass * acceleration$$

Eq. 7

If:

$$mass = \frac{W}{g} = \frac{\gamma * z}{g}$$

Eq. 8

Substituting Equation 8 in Equation 7, the horizontal seismic force is obtained in Equation 9:

$$Ts = P = \frac{\gamma * z}{g} * a = \sigma * \frac{a}{g}$$

Eq. 9

To continue with the process, it is necessary to divide the Equation 9, for σ' on both sides of the equation to get Equation 10 for the maximum shear stress.

$$\frac{Ts}{\sigma'} = \frac{\sigma}{\sigma'} * \frac{a}{g}$$

Eq. 10

Where:

P= Horizontal seismic force acting on the soil column
T_s= Maximum shear stress
ɣ= Total unit weight of the soil (kN/m^3)
z= Depth below ground level
a= Acceleration that is equal to the maximum horizontal acceleration a_{max} (m/s^2)
σ= Total vertical stress at the bottom of the soil column
σ'= Effective vertical stress at the bottom of the soil column

Equation 11 considers that the column of soil does not behave as a rigid body during the ground motion, a depth reduction factor (r) was added on the right side of Equation 10 to take into account that the soil is deformable.

$$\frac{Ts}{\sigma'} = r * (\frac{\sigma}{\sigma'} * \frac{a}{g})$$

Eq. 11

Seed et al., (1975) proposed a simplified method to convert the typical earthquake record to an equivalent series of uniform stress cycles assuming Equation 12.

$$Taverage = Tcyclic = 0.65 * Ts$$

Eq. 12

3.4.1. Cyclic stress ratio (CSR) and seismic stress ratio (SSR)

To assess the liquefaction process, the following needs to be calculated, the dimensionless parameters, Cyclic Stress Ratio (CSR) or Seismic Stress Ratio (SSR) defined as presented in Equation 13, while Equation 14 provides the value of the Reduction Factor (r).

$$CSR = SSR = \frac{Tcyclic}{\sigma'}$$

Eq. 13

$$Reduction\ Factor\ (r\ ) = 1 - 0.012 * z$$

Eq. 14

In Equation 14 the value of z is the depth in meters below the ground surface where the liquefaction analysis is being performed.

Equation 15 is the final equation for CSR.

$$CSR = SSR = 0.65 * r * (\frac{\sigma}{\sigma'} * \frac{a}{g})$$

Eq. 15

3.4.2. Cyclic resistance ratio (CRR)

The cyclic resistance ratio (CRR) represents the resistance of the soil against liquefaction. The most common way or method used to determine liquefaction resistance is through the number of standard penetration test blows (N), corrected N_{60}.
Additionally, Seed et al., (1985) propose approximated potential damages ranges based on the N_{60} presented in Table 12.

Table 12. Liquefaction potential damage (Seed et al., 1985)

Range N.	Range of SPT-N60	Potential Damage
1	0-20	High
2	20-30	Medium
3	>30	No significant damage

On the other hand, the Cyclic Resistance Ratio (CRR), could be obtained from Figure 9 which reflects the result of many tests on different cohesionless materials. It needs to be said that although Figure 9 was not made based on Terrafame's site conditions, it is a good reference for an approximated calculation.

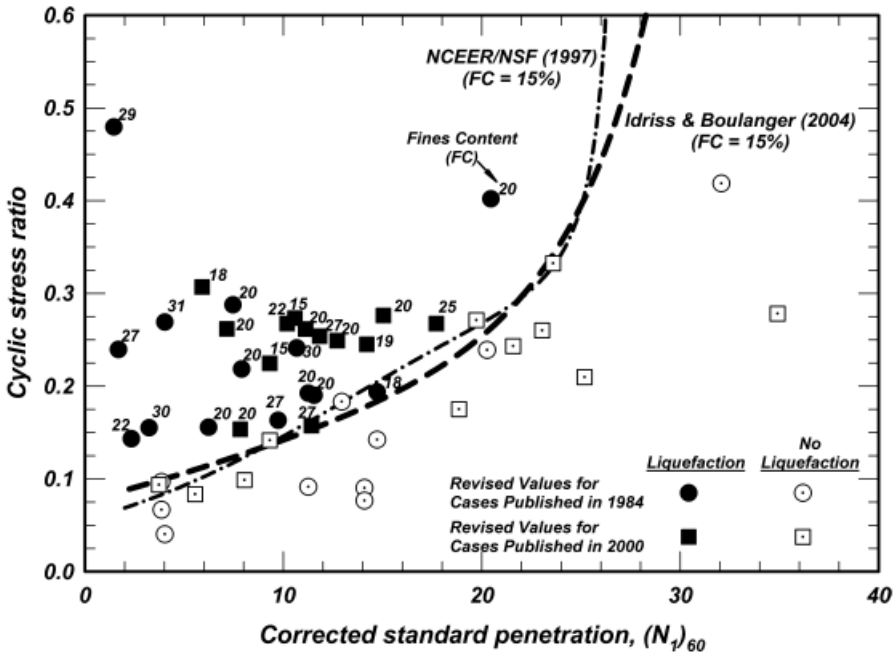


Figure 9. (N1)60 (SPT) and Cyclic Resistance Ratio case histories of cohesionless soils (Idriss & Boulanger, 2008)

3.4.3. Factor of safety (FoS) against liquefaction

Equation 16 gives the value for the factor of safety (FoS) that reflects the resistance of the soil to liquefaction and correlates directly the cyclic resistance ratio (CRR) with the cyclic stress ratio (CSR).

$$FoS = \frac{CRR}{CSR}$$

Eq. 16

Therefore, if the cyclic resistance ratio (CRR) is greater than the cyclic stress ratio (CSR), the liquefaction could not occur in the site, but it is important to notice that if the FoS is slightly greater than 1, liquefaction may still occur in a lower layer during an earthquake, and due to the upwards flow of water, it could trigger liquefaction of the upper layers.

4. Results and discussion

4.1. Movement characterization of the heap leaching piles

One concern was determining if the blast process was causing reflection movements in the heap leaching piles. This means that after a blasting, the percolation solution within the leaching piles is moving from one side to another ("bouncing effect") as a consequence of the vibrations, generating a forced movement of the piles.
Hence, if the number of movements registered in the leaching piles is higher than those recorded in the bedrock or in the pit, an extra factor is producing an unexpected behaviour in the piles.
But, after analysing and comparing several accelerograms from different blasting events in the pit, the number of movements recorded in the leaching piles and in the bedrock is directly related to the number of blasts given in the open pit. So, there is no reflection movements affecting the leaching piles.
Figure 10 presents an example where one blast in the pit produces one movement in the leaching pile 1, as well as one movement in the bedrock. On the other hand, Figure 11 shows with another example that if two blasts are carried out in the open pit, two motions are also given in the heap leaching pile 1 and in the bedrock.

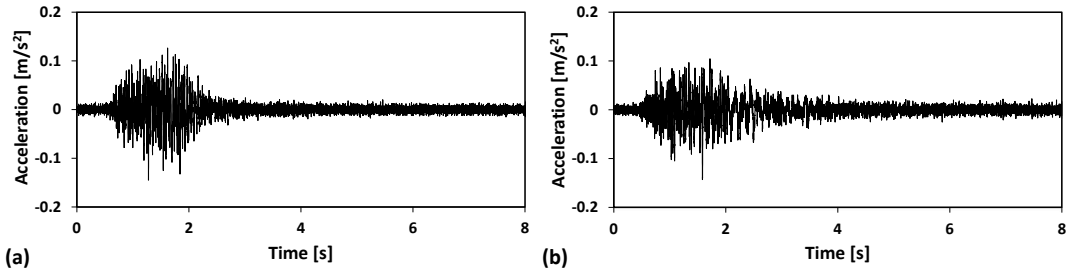


Figure 10. (a) Accelerogram bedrock east; (b) accelerogram heap leaching pile 1 - One movement recorded.

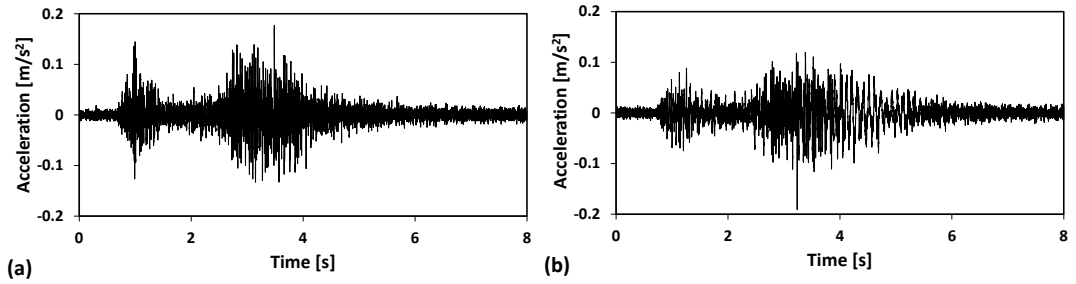


Figure 11. (a) Accelerogram bedrock west; (b) accelerogram heap leaching pile 1 - Two movements recorded.

4.2. Maximum measured values for acceleration, velocity, and amplitude

This section presents the maximum (peak) values registered for acceleration, velocity, and amplitude in the heap leaching piles.

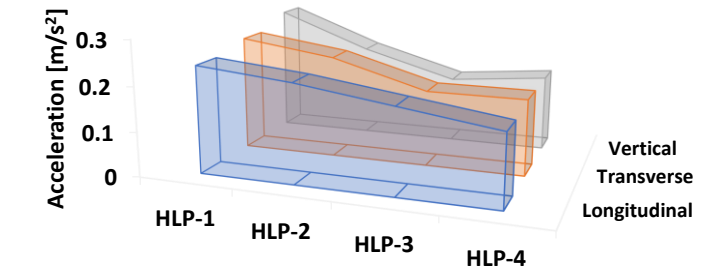
Figure 12 shows that generally the horizontal planes (longitudinal and transverse) present greater values (for acceleration, velocity, and amplitude) if it is compared with the vertical plane.

At this point it is relevant to remind that the order of the leaching piles from the closest to the farthest to the open pit is pile 1 (HLP-1), pile 2 (HLP-2), pile 3 (HLP-3), and pile 4 (HLP-4).

For piles 1 and 2 the distance from the blasting source influences directly because the closer the leaching pile is to the open pit, the higher its values for acceleration, velocity, and amplitude. Later, this tendency is lost when leaching pile 3 presents lower values compared with leaching pile 4, despite pile 3 not being the farthest. The reason for this is unknown yet in this study stage due to the lack of data to analyze and compare.

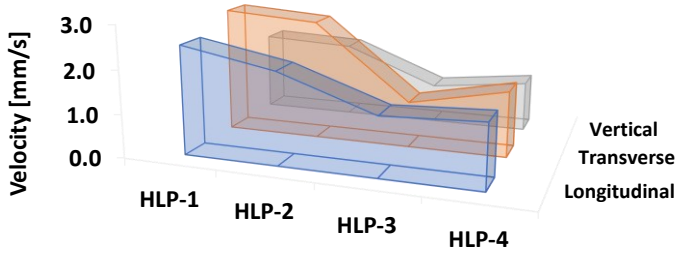
In this case, the parameter peak ground velocity (PGV) will not be studied more in detail, or used for further calculations because the literature is pointing it more as a damage assessment parameter over structures. Hence, PGV is more often used to correlate strong motions characteristics with their effects over structural and non-structural elements such as masonry walls, columns, beams, among others.

On the other hand, the peak acceleration values in the horizontal plane presented in the Figure 12 (a) for the four heap leaching piles 0.26 m/s² (HLP-1), 0.23 m/s² (HLP-2), 0.19 m/s² (HLP-3), and 0.17 m/s² (HLP-4), are fundamental to determine the cyclic shear stress acting over the leaching piles for performing the liquefaction assessment.



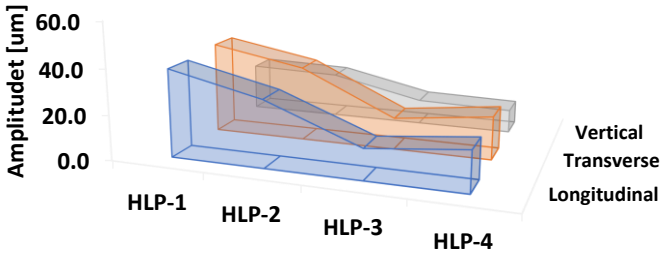
	HLP-1	HLP-2	HLP-3	HLP-4
Longitudinal	0.24	0.22	0.19	0.16
Transverse	0.26	0.23	0.17	0.17
Vertical	0.29	0.21	0.15	0.17

(a) Longitudinal Transverse Vertical



	HLP-1	HLP-2	HLP-3	HLP-4
Longitudinal	2.5	2.1	1.35	1.45
Transverse	2.9	2.75	1.05	1.5
Vertical	1.85	1.75	0.9	1.15

(b) Longitudinal Transverse Vertical



	HLP-1	HLP-2	HLP-3	HLP-4
Longitudinal	38.90	29.60	13.60	17.9
Transverse	40.80	33.20	14.10	19
Vertical	21.30	20.20	11.20	10.3

(c) Longitudinal Transverse Vertical

Figure 12. Measured values in the longitudinal, transverse, and vertical planes for (a) Maximum acceleration in heap leaching piles; (b) Maximum velocity in heap leaching piles; (c) Maximum amplitude in heap leaching piles

4.3. Site design spectrums

A site design spectrum was made for each one the four heap leaching piles, the bedrock east & west, and for the open pit. For this last, ten site response spectrums were used for each one of the mentioned locations.
A statistical analysis based on the mean-plus-one-standard-deviation of the pseudo-acceleration values was applied to find out the peak values that drive the curve for the site design spectrum. The prefix "pseudo" is used to avoid possible confusion between the acceleration of the spectrum and the true ground acceleration values.
In this case, the mean-plus-one-standard-deviation analysis is equivalent to compute the non-exceedance 84th percentile of the values of pseudo–acceleration given by the site response spectrums. Figure 13 exemplifies the way in which a site design spectrum is obtained from a 84th percentile of non-exceedance (solid line) statistical analysis of the site response spectrums (dashed coloured lines).

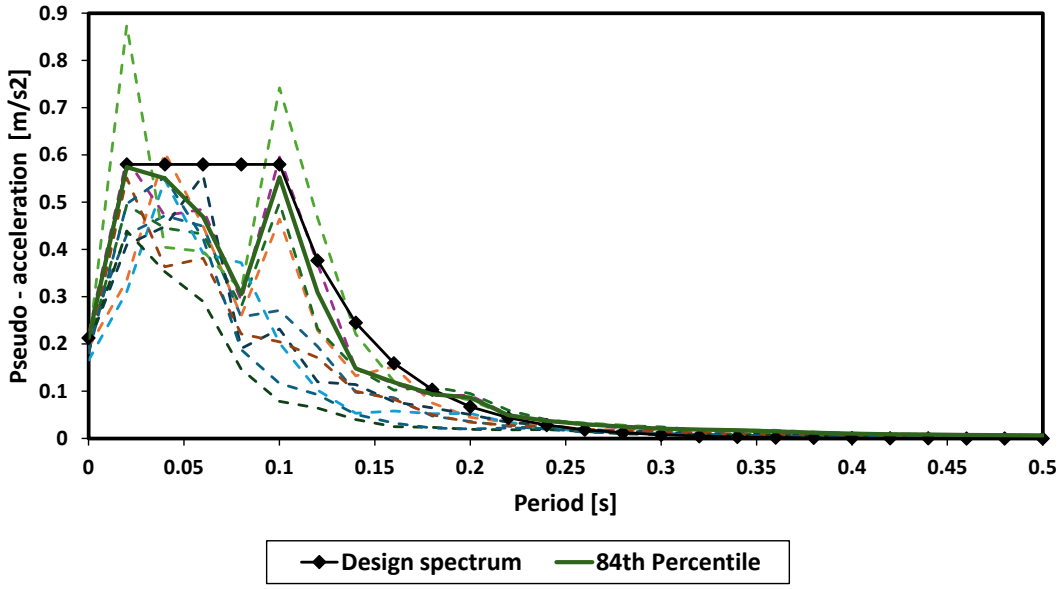


Figure 13. Exemplification of the obtention of a site design spectrum

Concerning leaching piles, from Figure 14 it is clear that pile 1 has the highest values of pseudo–acceleration, being also the closest one to the open pit. Then, following the same trend, the second leaching pile possesses the second highest value for the pseudo–acceleration, and it is the second closest to the open pit.
Pile 3 and pile 4 have almost the same behavior referring to the design spectrum, although it was expected that pile 4 possesses the lowest values as it is the farthest point to the open pit. This supports the trend of the peak

values for acceleration, velocity, and amplitude, where pile 3 had the lowest values, as well as in this statistical case.
In the leaching piles, structures with periods of vibration from around 0.02 to 0.15 seconds are the most affected by the effects of blast vibrations. When talking about structures, these can also be temporary structures such as the ones used to support the conveyor belts that are constantly working to raise the leaching piles.
Unfortunately, again, a more accurate statement about piles 3 and 4 cannot be given at this point because there are no more leaching piles in further distances to study and to obtain more accurate comments to contribute to the analysis.
The site design spectrums belonging to the bedrock east and west were also plotted in Figure 14, and the first noticeable feature is their different shape in the curve, where a pronounced peak suggest a greater affectation on structures with a lower period of vibration if compared with the leaching piles.
Hence, the period of affection in bedrock is substantially lower than in leaching piles, this indicates that using the spectra from the leaching piles may overestimate the design amplifications for rock.
Different shapes in the design spectrum curves also describe the stiffness of the soil, thus, as heap leaching piles have a flat top part in their behavior are related to a soft – medium sand. On the other side, the peaks for bedrock, reflect a stiff/rock material, as it is better exposed and supported with Figure 15.

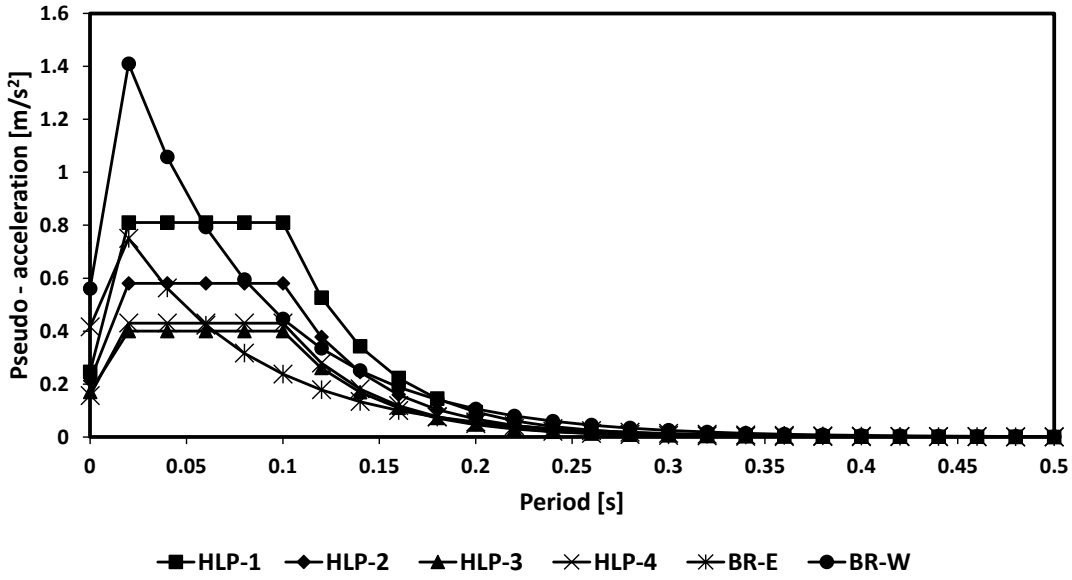


Figure 14. Site design spectrum for heap leaching piles and bedrock points

This last analysis and results also fit in Figure 15 proposed by Seed et al., (1996) where the shape of the site design spectrum is based on different soil conditions (stiffness), and again, the period of affection in rock is substantially lower than compared with less stiff soils.

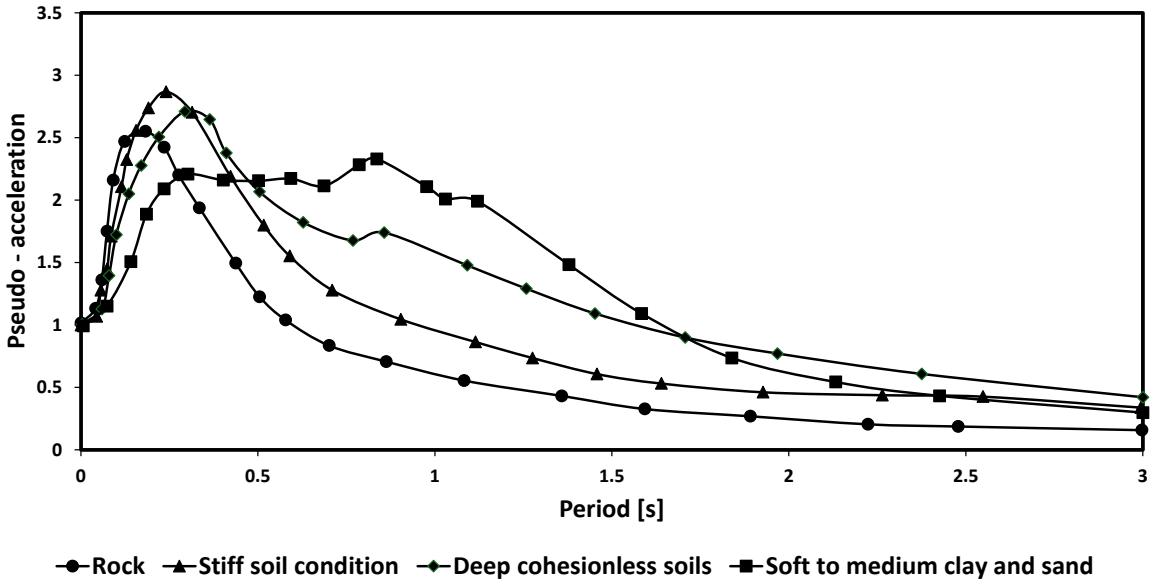


Figure 15. Average acceleration spectra for different soil conditions (Based on Seed & Idriss, 1969)

Bedrock west is located 200 m closer to the open pit and presents higher values for pseudo acceleration than bedrock east (Figure 14). So, more site conditions are affecting the response of bedrock east and west regarding the noticeable difference in their answer.

Finally, Figure 16 contains the site design spectrums corresponding to heap leaching piles, bedrock, and open pit. As it is clear, the scale in the vertical axis works only for the open pit because its values are too high (logical behavior because of its proximity to the blasting sources) compared with bedrock and leaching piles. Anyway, Figure 16 is more comparative information of the results to highlight the magnitude of the values of pseudo-acceleration for the open pit and its differences compared with heap leaching piles and bedrock.

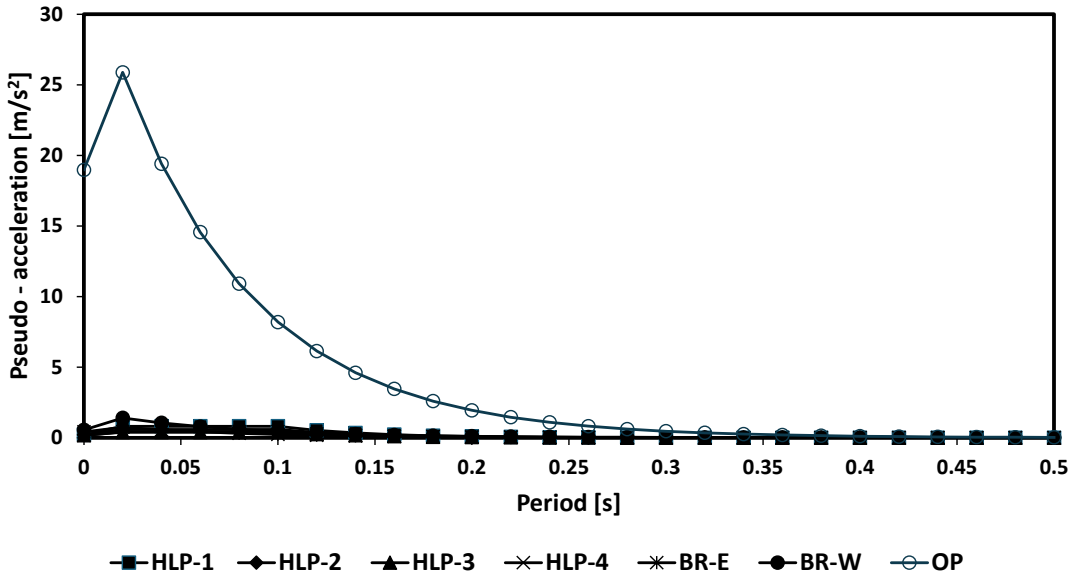


Figure 16. Site design spectrum for heap leaching piles, bedrock points, and open pit

4.4. Arias intensity (AI) and significant duration (t_d) for strong motions

Regarding the significant duration (t_d) the places related to rock (bedrock east, bedrock west, and open pit) are the fastest in dissipating seismic energy, which means that their significant duration is lower than in leaching piles.

Also, the rock-related measure points receive a higher amount of energy from the blasting vibrations (higher values of Arias intensity 95) if compared with the leaching piles.

The measure points located in the bedrock and mainly in the open pit are the closest locations to the blasting source, so, the energy received by those is greater than the energy received by the leaching piles.

On the other hand, for heap leaching piles there is an important reduction in the vibration's intensity due to the interface of soil-rock as it is stated by Jayasinghe (2018).

Regarding the heap leaching piles, the farther the leaching pile is from the open pit, the greater their significant duration (t_d). Thus, the motion in leaching piles three and four lasts longer than in leaching piles one and two.

Finally, there is not a clear tendency concerning the amount of energy dissipated as Arias intensity 95 between the leaching piles.

4.5. Correlations for ground motion parameters

As peak ground acceleration (PGA) is one of the easiest parameters to obtain from the ground motion measurement, it is imperative to find out if there is a correlation of (PGA) with other ground motion parameters such as Arias intensity 95 (AI95) or significant duration (t_d).
Figure 17 correlates the values for PGA and AI95 where equation (a) replicates the behaviour for the leaching piles 1, 2, and 4; on the other side, equation (b) replicates the behaviour for leaching pile 3. To be able to use those equations, the variable "y" represents (AI95) in [m/s] and the variable "x" represents (PGA) in [m/s^2].
As an approach for solving the uncertainty in piles 3 and 4, this analysis shows that the behaviour of leaching pile 3 slightly differs from the other three, while pile 4 keeps the trend and similar behaviour to piles 1 and 2.

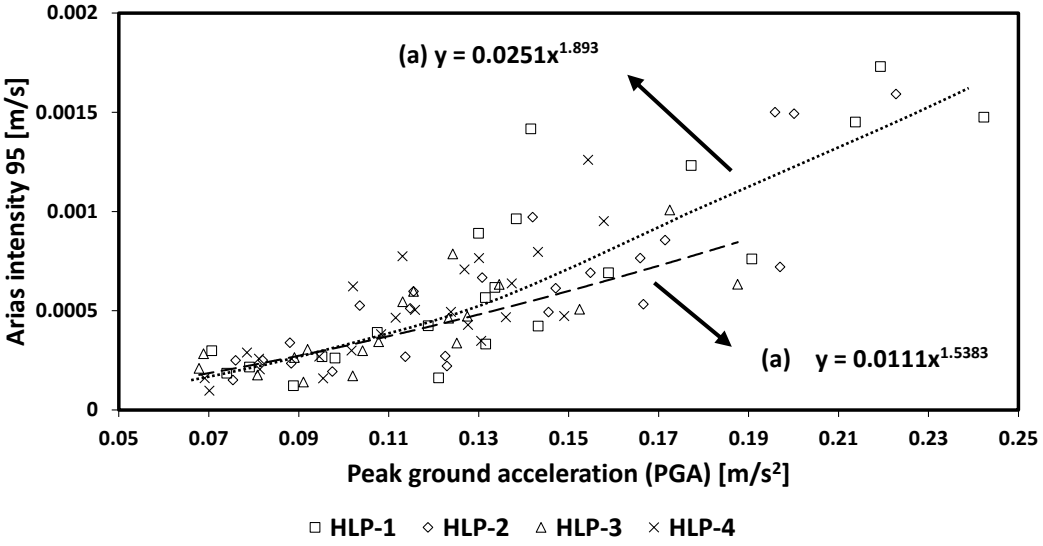


Figure 17. Correlation between Arias intensity 95 and peak ground acceleration

Those equations work only for Terrafame mine and its site conditions, nevertheless, in further studies, those can be used in another location to see how well the calculated values fit into the real measurements.
Although the trend of the equations works well for this range of values, it is recommended to carefully handle the results if higher values of PGA are used as input in the proposed equations.
Furthermore, it can be stated that the significant duration (t_d) is not a parameter that has any relation to PGA or AI95 magnitudes. Sometimes there are some assumptions that are wrongly conceived regarding the damaging features of strong motions, and that are often repeated and spread such as that the destructive capacity of a ground motion increases with its duration, nevertheless, the duration by itself does not indicate the

damage potential at all. The fact is that if there are two accelerograms with similar peak ground acceleration (PGA) and frequency, but one of them possesses a longer duration, it is more likely to be more destructive. Hence, if the same amount of energy is transmitted in a very short period, it results in a considerably greater damage.

4.5.1. Preliminary liquefaction assessment in heap leaching piles.

The simplified method for a preliminary liquefaction risk assessment in the heap leaching piles requires computing the cyclic resistance ratio (CRR) correlated with the shear resistance of the cohesionless material based on $(N_1)_{60}$, and the cyclic stress ratio (CSR) calculated using the maximum horizontal acceleration on site.

For each leaching pile it was considered two different sections given that the height of each pile varies considerably in the north part (lower) and the south part (higher). If only one profile for each pile is considered, there would be an overestimation of the factor of safety (FoS) against liquefaction in the lowest area, additionally, a saturated condition (worst case) of the material was considered from the top surface of the leaching piles.

The location of the drillings is available in Annex 1, the values for friction angles are presented in Annex 2, while N_{60} values are in Annex 3, finally $(N_1)_{60}$ and the factors of safety (FoS) against liquefaction due to blasting vibrations are displayed in Annex 4.

As a general comment, there is no imminent risk of failure due to liquefaction in the heap leaching piles caused by the blast vibrations. The values obtained in the preliminary liquefaction assessment for the piles are in a range between 2.6 to 5.8 where the lowest values for each leaching pile belong to the first two or three meters (surface) of the pile.

It is important to highlight that this is a simplified method to approach reference results, so, there are important limitations recreating the real conditions in the heap leaching piles, so, the obtained FoS should be interpreted with caution and criteria. A summary of the minimum and maximum values obtained for each leaching pile is described in Table 13.

Table 13. Summary of factors of safety (FoS) against liquefaction due to blasting vibrations in heap leaching piles

Heap leaching pile	Section	FoS		Comment
		min	max	
1	South	2.6	3.5	The lowest value on the surface (first meter)
	North	3.1	5.4	The lowest value is at 17 m depth. FoS in the surface is 3.5
2	South	3.0	3.9	The lowest value on the surface (2 first meters)
	North	3.0	4.3	The lowest value on the surface (2 first meters)
3	South	3.9	5.3	The lowest value on the surface (first meter)
	North	3.6	5.3	The lowest value on the surface (first meter)
4	South	4.4	5.8	The lowest value on the surface (first meter)
	North	4.4	5.6	The lowest value on the surface (first meter)

Finally,
Table 14 shows the maximum values of Arias intensity 95 for each one of the four leaching piles.

Table 14. Maximum values of Arias intensity 95 for heap leaching piles

Facility	Arias intensity 95 [m/s]
HLP-1	0.00170
HLP-2	0.00159
HLP-3	0.00100
HLP-4	0.00130

For this specific case, although the Arias intensity criteria shows that there is no risk for liquefaction in the heap leaching piles because the highest values of Arias intensity in the piles are not even considered in the lowest part of the plot made by Kayen (1997) used for liquefaction assessment based on $(N_1)_{60}$ and Arias intensity (Figure 18).
In this case, this fact could give a "false" safety sensation while assessing the heap leaching piles because the FoS based on Arias intensity is equal to the ratio of Arias intensity required to cause initial liquefaction (At least 0.25 m/s according to Figure 18) to the Arias intensity estimated from the ground motion (0.0017 m/s in this case).
So, the Arias intensity method provides a better perspective on liquefaction danger if the registered values for Arias intensity are at least 0.1 m/s.

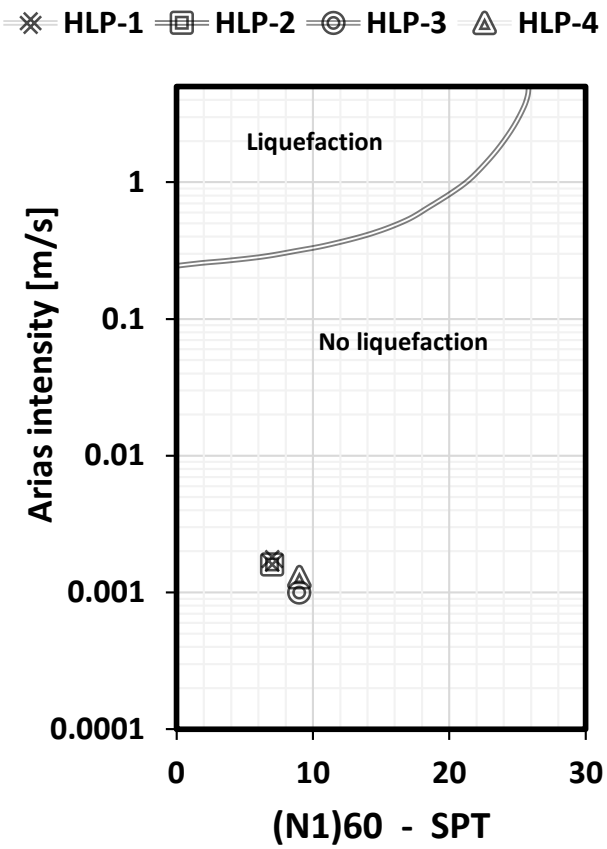


Figure 18. Plot for liquefaction assessment- Arias intensity 95 vs. (N1)60 – Arias intensity values for heap leaching piles (Based on Kayen, 1997)

5. Conclusions

- The recorded movements in the heap leaching piles are directly related to the number of blasts in the open pit. Likewise, the number of motions detected over the rock-related facilities (bedrock east, bedrock west, and open pit) is also detected in the recordings for the heap leaching piles. Therefore, the irrigated saturation solution does not cause a reflection in the movement of the heap leaching piles.

- Heap leaching piles 1 and 2 exhibit an expected behavior where acceleration, velocity, and amplitude values increase as the distance to the blasting source decreases. Therefore, the closer the piles are to the pit, the higher these values. Nevertheless, pile 3 deviates from this trend. Despite not being the farthest one from the pit, it has the lowest acceleration, velocity, and amplitude values.

- Arias intensity and significant duration of the ground motion were used to characterize the energy dissipation process for the mining facilities. The bedrock east, bedrock west, and open pit (rock-related) release more energy as Arias intensity, reaching up to 1.2 m/s during the ground motion, and can dissipate it faster compared with the heap leaching piles (composed granular material).
  It is important to highlight that the rock-related points are closer to the blasting source compared with the leaching piles. Thus, distance also plays a significant role in the amount of energy that these facilities receive.

- A simplified method was used for a preliminary liquefaction risk assessment of the four secondary leaching piles due to blasting vibrations. Where the obtained factors of safety (FoS) are in a range from 2.6 to 4.0 in most cases, being the lowest values repetitively located on the surface of the piles.
  This last analysis indicates that there is no imminent risk of liquefaction in the leaching piles. But it is important to consider that this simplified procedure has important limitations replicating the real conditions in the heap leaching piles. Therefore, it provides referential results which should be interpreted with caution and critical judgement.

- Arias intensity criteria indicates that there is no risk of liquefaction in the heap leaching piles caused by blast vibrations. However, here

the FoS values could create a false sense of security while assessing the leaching piles.
In this case, the FoS is equal to the ratio between the Arias intensity required to initiate liquefaction (at least 0.25 m/s) and the Arias intensity estimated from the ground motion (0.0017 m/s in this case). Therefore, the Arias intensity method provides a better perspective on liquefaction risk if the recorded Arias intensity values are at least 0.1 m/s.

- A broader geotechnical campaign is needed to gather more information about the granular constitutive material of the leaching piles.
  The tests should include, but not be limited to, geophysical measurements, a broader gradation characterization, assessment of material saturation at various depths and locations, installation of piezometers at strategic points, dynamic triaxial test, direct shear test, and permeability measurements at different depths, among others.

- A definitive geotechnical risk assessment is highly recommended to address the limitations of the preliminary liquefaction assessment.
  This should include a complete geotechnical material characterization (as previously mentioned), the development of a geotechnical model incorporating the slope's geometry, analysis of long-term volume changes in the leaching piles, and the evaluation of the variation in the effective stresses due to the increase of the pore pressure during seismic events.

Considerations and future scope

Due to the lack of information, this thesis had some limitations in providing more accurate answers related to the behavior of the secondary heap leaching piles. Therefore, extra considerations are exposed in this section as part of a future scope research related to material testing and a numerical model of the heap leching piles.
As feedback, it is suggested to implement more seismic measure points (Table 15), and to install them vertically aligned at different depths (from top part of the leaching piles until bedrock). This approach will enable to perform a more accurate study of the vibration's amplification in the bedrock, bedrock-soil interface, and heap leaching piles.
Finally, it is strongly recommended to have a detailed record of the blasting calendar and blasting features to correlate them with the effects of blasting vibrations.

Table 15. Measurement of seismic properties as future scope

Type	Test	Measurement	Description/ Requirement
Seismic properties	Seismograph monitoring	Vibration levels	Measure peak ground velocity, peak ground acceleration, and frequency values during ground motions.
	Ground vibration monitoring (Geophones)	Ground vibrations at the source	Measure vibration levels directly at the blasting site.

Table 16 contains tests suggested for a better geotechnical characterization of the heap material. Additionally, it is important to obtain basic parameters such as air void content and degree of saturation of the samples.

Table 16. Measurement of geotechnical properties as future scope

Type	Test	Measurement	Description/ Requirement
Geotechnical properties	Sieve Analysis & Hydrometer	Particle size Distribution	Determine changes in particle size distribution as a long-term effect of the solution over the crushed ore material
	Constant Head and Falling Head Permeability Tests	Permeability	Measure the permeability of heap material to assess changes in flow characteristics.
	Direct shear/Cyclic direct shear/ Cyclic triaxial test	Material Strength	Assess the strength of the heap material under regular and cyclic loading.
	Bulk Density Test	Density	Measure the density of the heap material along the depth.

Since the beginning of this project, it was evident that the scope was limited due to the amount of information and geotechnical characterization of the material. However, using the available data, a preliminary liquefaction assessment was conducted based on a simplified procedure.
This simplified approach has technical limitations if compared with modern numerical models. These limitations can be overcome by using a comprehensive geotechnical model which would provide a more adequate and accurate reference for a definitive risk stability assessment of the heap leaching piles.

The following bullet points provide some considerations for the numerical modelling of the heap leaching piles.

- Use finite element or discrete element modeling software.
- Simulate the propagation of blasting vibrations through the heap leaching pile.
- Model the stability and structural integrity of the heap under dynamic loading.
- Include interaction between the heap material and underlying structures.
- Calibrate models using experimental data and validate against observed outcomes.
- Perform sensitivity analysis to understand the influence of various parameters.
- Simulate the long-term effects of repeated blasting on heap stability and leaching efficiency.

Furthermore, it is important to think about extreme cases after the end-of-operation process of the heap leaching piles. Hence, some considerations are displayed below:

- Incorporate desiccation cracks to account for failure due to preferential progressive flow.
- As the leaching piles will be subjected to freeze-thaw cycles after the treatment is stopped. The FoS with respect to freezing related failure need to be considered.

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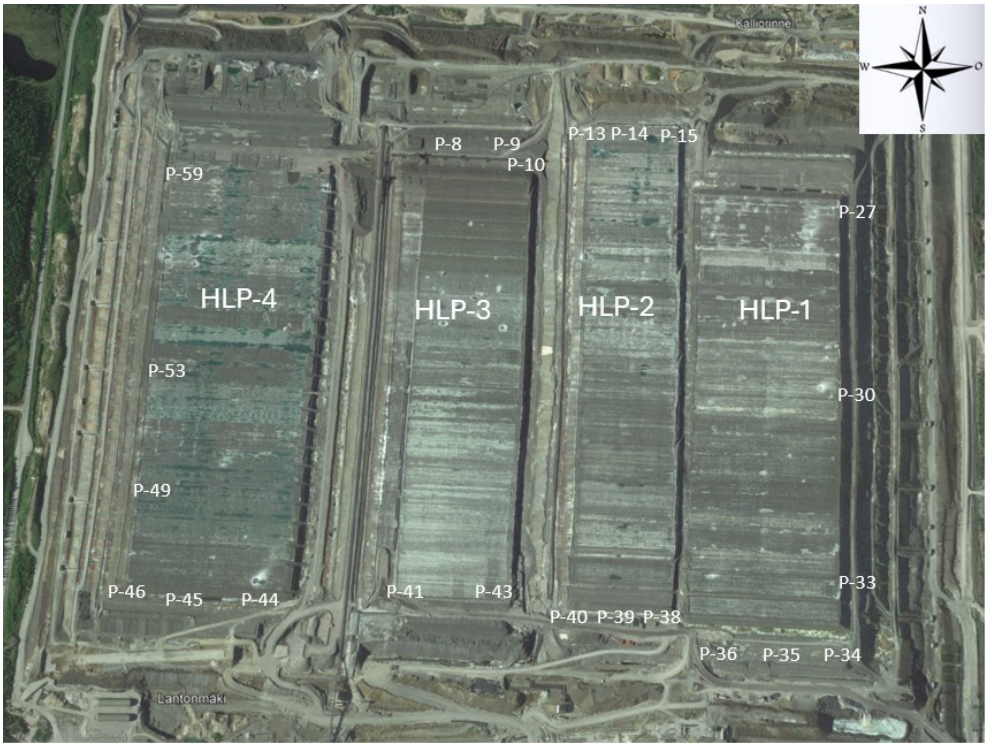
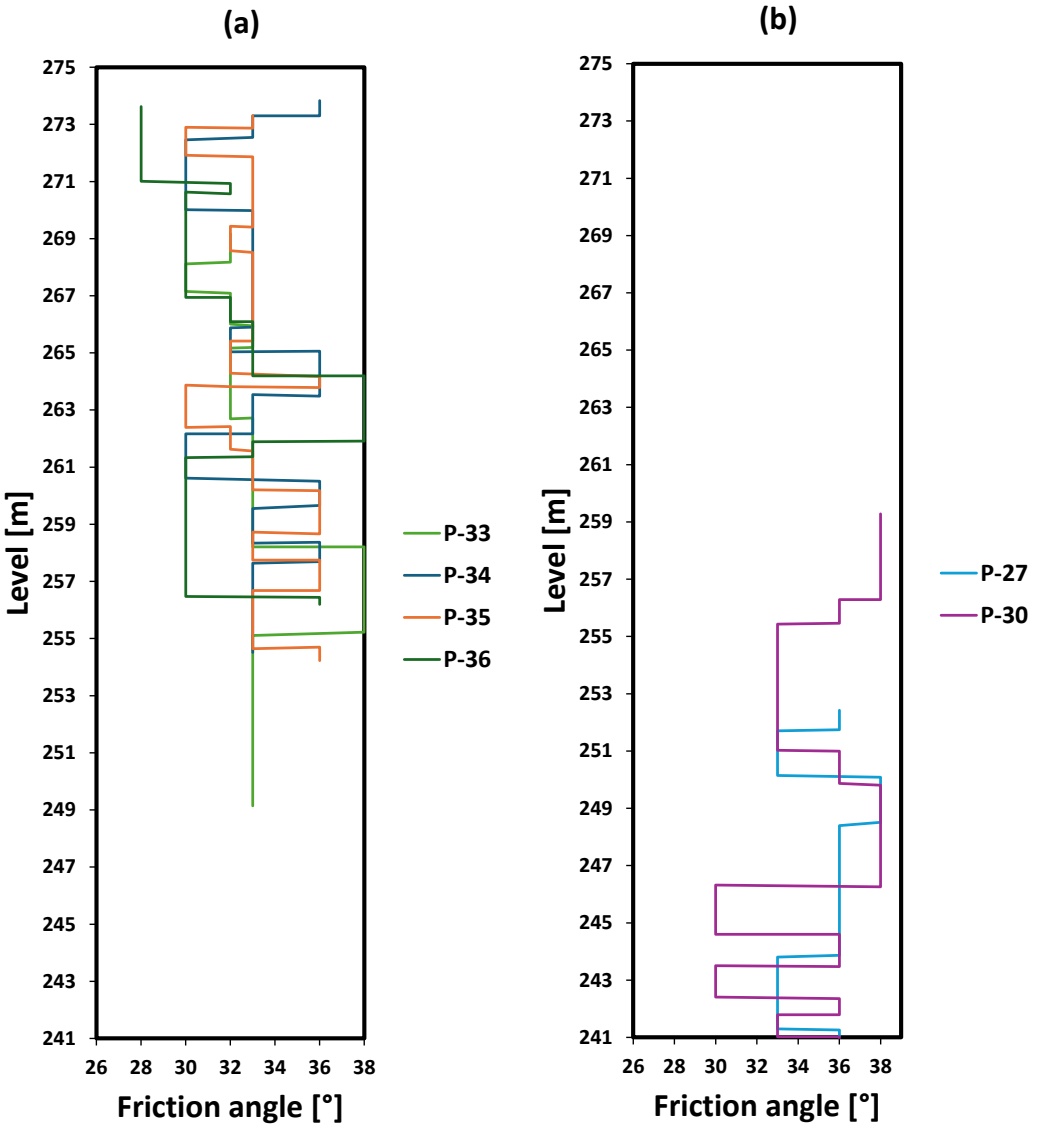
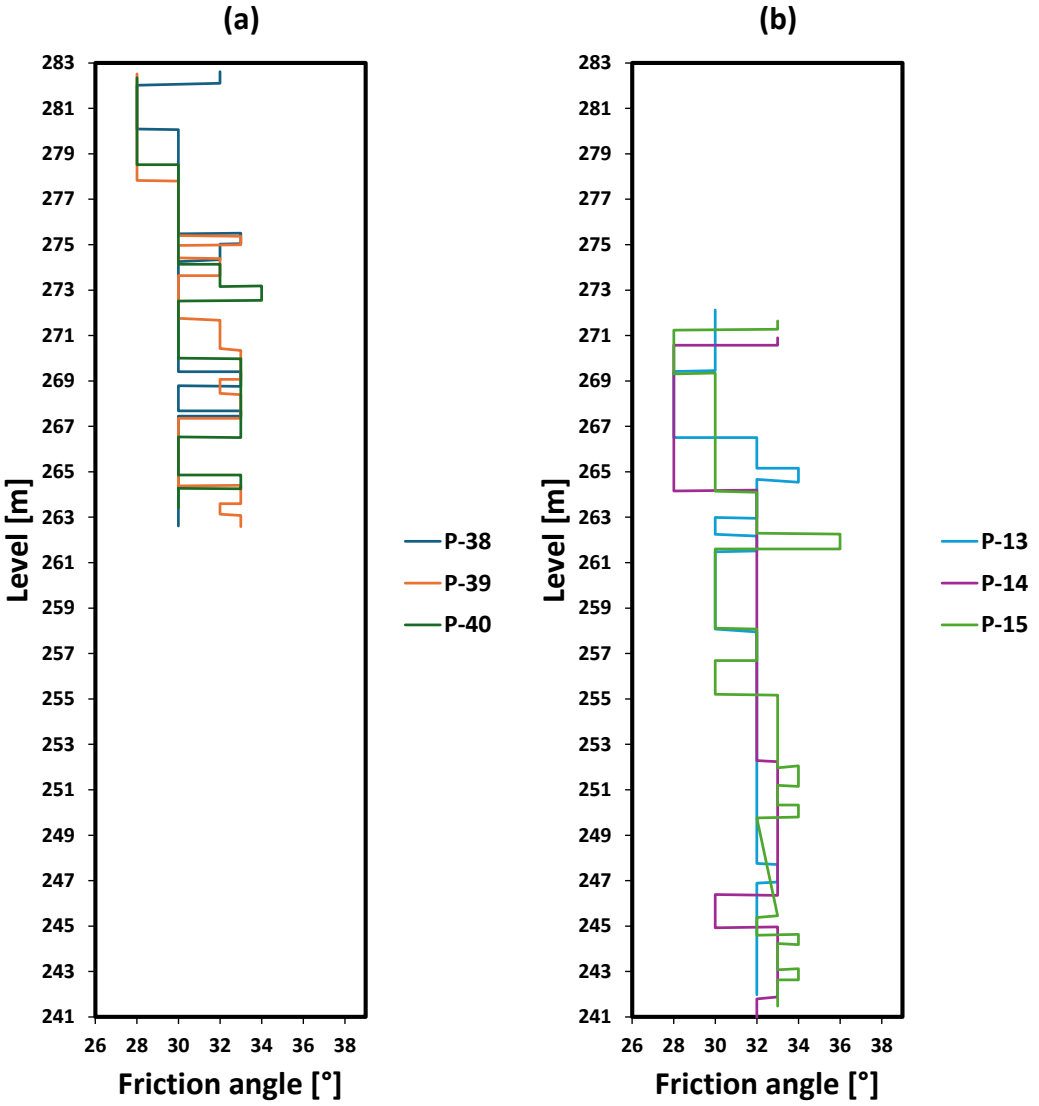
Annexes



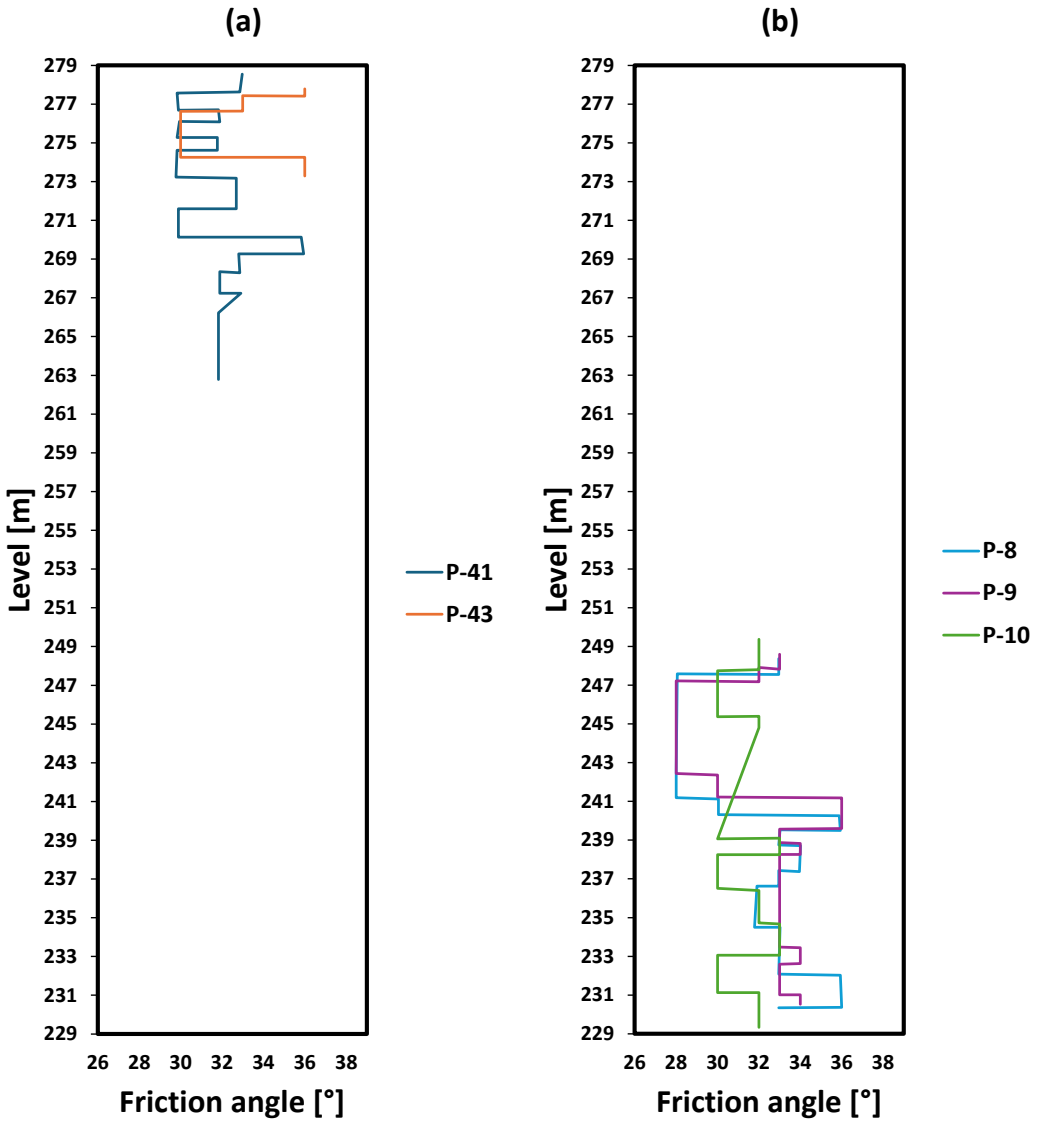
Location of drillings in heap leaching piles 1, 2, 3, and 4



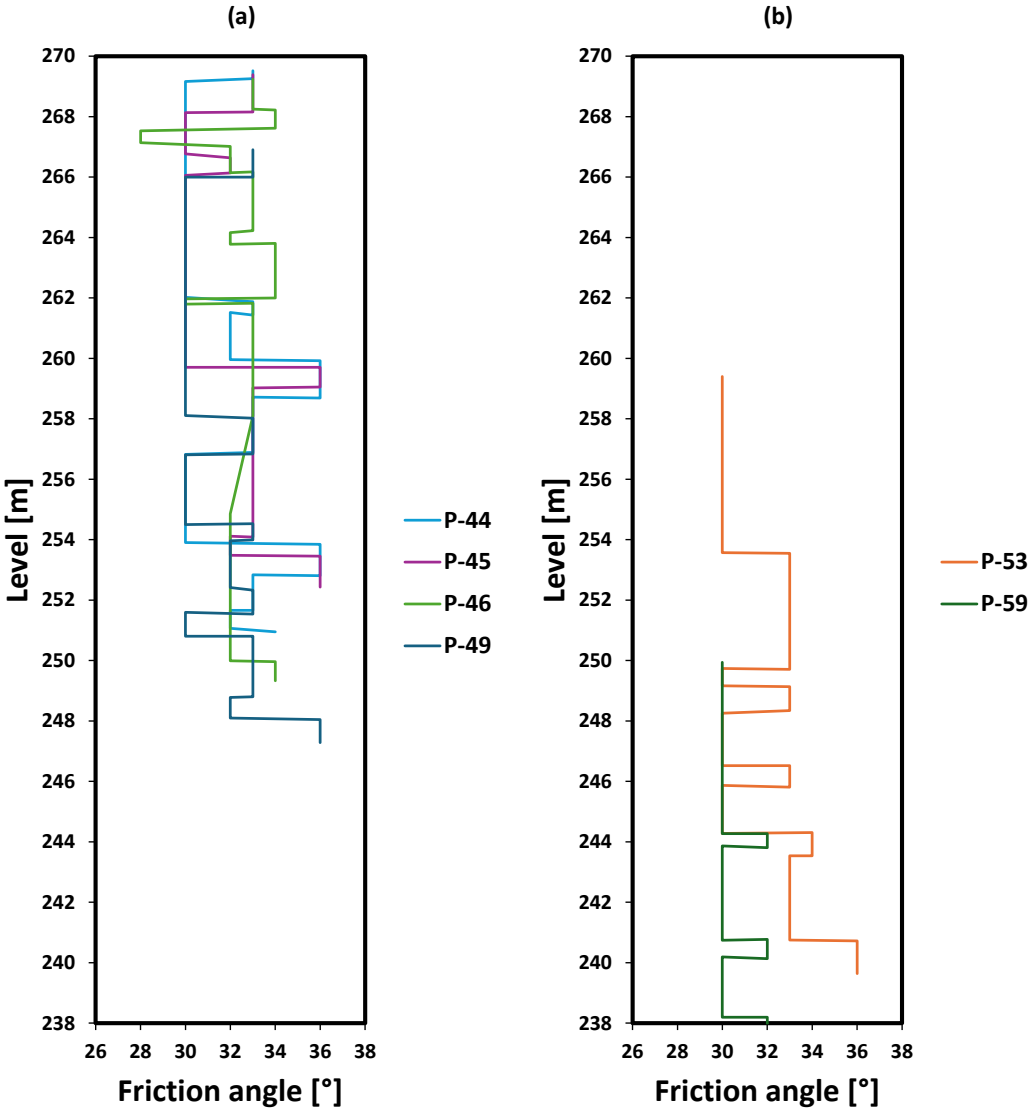
Friction angles for heap leaching pile 1. (a) South section and (b) north section



Friction angles for heap leaching pile 2. (a) South section and (b) north section



Friction angles for heap leaching pile 3. (a) South section and (b) north section



Friction angles for heap leaching pile 4. (a) South section and (b) north section

Friction angles (Φ) for heap leaching pile 1 – south section

HLP:		1							
Section:		South							
Situation:		Current							
Drilling:		33-34-35-36							
GW (m):		273.00							
Level (m)		**Depth (m)**	**Φ**	**N 1**	**N 2**	**N 3**	**N. AV**	**C_E**	**N_{60}**
273.00	271.00	0.00	28	8	11	5	8	0.8	7
271.00	267.00	2.00	30	11	15	8	12	0.8	9
267.00	264.00	6.00	32	14	19	12	15	0.8	12
264.00	256.00	9.00	30	11	15	8	12	0.8	9
256.00	252.00	17.00	33	16	22	14	17	0.8	14
252.00	252.00	21.00	33	16	22	14	17	0.8	14

Friction angles (Φ) for heap leaching pile 1 – north section

HLP		1							
Section:		North							
Situation:		Current							
Drilling:		27-30							
GW (m):		259.00							
Level (m)		**Depth (m)**	**Φ**	**N 1**	**N 2**	**N 3**	**N. AV**	**C_E**	**N_{60}**
259.00	250.00	0.00	33	16	22	14	17	0.8	14
250.00	246.00	9.00	33	16	22	14	17	0.8	14
246.00	245.00	13.00	30	11	15	8	12	0.8	9
245.00	243.00	14.00	36	22	29	21	24	0.8	19
243.00	242.00	16.00	30	11	15	8	12	0.8	9
242.00	241.00	17.00	36	22	29	21	24	0.8	19
241.00	240.00	18.00	33	16	22	14	17	0.8	14
240.00	240.00	19.00	36	22	29	21	24	0.8	19

Friction angles (Φ) for heap leaching pile 2 – south section

HLP		2							
Section:		South							
Situation:		Current							
Drilling:		38-39-40							
GW (m):		282.00							
Level (m)		**Depth (m)**	**Φ**	**N 1**	**N 2**	**N 3**	**N. AV**	**C_E**	**N_{60}**
282.00	280.00	0.00	28	8	11	5	8	0.8	7
280.00	278.00	2.00	28	8	11	5	8	0.8	7
278.00	263.00	4.00	30	11	15	8	12	0.8	9
263.00	263.00	19.00	30	11	15	8	12	0.8	9

Friction angles (Φ) for heap leaching pile 2 – north section

HLP		2							
Section:		North							
Situation:		Current							
Drilling:		13-14-15							
GW (m):		271.00							
Level (m)		**Depth (m)**	**Φ**	**N 1**	**N 2**	**N 3**	**N. AV**	**C_E**	**N_{60}**
271.00	269.00	0.00	28	8	11	5	8	0.8	7
269.00	264.00	2.00	28	8	11	5	8	0.8	7
264.00	263.00	7.00	32	14	19	12	15	0.8	12
263.00	258.00	8.00	30	11	15	8	12	0.8	9
258.00	257.00	13.00	32	14	19	12	15	0.8	12
257.00	255.00	14.00	30	11	15	8	12	0.8	9
255.00	247.00	16.00	32	14	19	12	15	0.8	12
247.00	245.00	24.00	30	11	15	8	12	0.8	9
245.00	0.00	26.00	30	11	15	8	12	0.8	9

Friction angles (Φ) for heap leaching pile 3 – south section

HLP:		3							
Section:		South							
Situation:		Current							
Drilling:		41-43							
GW (m):		278.00							
Level (m)		**Depth (m)**	**Φ**	**N 1**	**N 2**	**N 3**	**N. AV**	**C_E**	**N_{60}**
278.00	273.00	0.00	30	11	15	8	12	0.8	9
273.00	272.00	5.00	33	16	22	14	17	0.8	14
272.00	270.00	6.00	30	11	15	8	12	0.8	9
270.00	269.00	8.00	36	22	29	21	24	0.8	19
269.00	262.00	9.00	32	14	19	12	15	0.8	12
262.00	262.00	16.00	32	14	19	12	15	0.8	12

Friction angles (Φ) for heap leaching pile 3 – north section

HLP:		3							
Section:		North							
Situation:		Current							
Drilling:		8-9-10							
GW (m):		249.00							
Level (m)		**Depth (m)**	**Φ**	**N 1**	**N 2**	**N 3**	**N. AV**	**C_E**	**N_{60}**
249.00	247.00	0.00	28	8	11	5	8	0.8	7
247.00	241.00	2.00	28	8	11	5	8	0.8	7

241.00	236.00	8.00	30	11	15	8	12	0.8	9
236.00	234.00	13.00	32	14	19	12	15	0.8	12
234.00	233.00	15.00	33	16	22	14	17	0.8	14
233.00	231.00	16.00	30	11	15	8	12	0.8	9
231.00	229.00	18.00	32	14	19	12	15	0.8	12
229.00	229.00	20.00	32	14	19	12	15	0.8	12

Friction angles (Φ) for heap leaching pile 4 – south section

HLP:		4						
Section:		South						
Situation:		Current						
Drilling:		44-45-46-49						
GW (m):		269.00						
Level (m)		**Depth (m)**	**Φ**	**N 1**	**N 2**	**N 3**	**N. AV**	**C_E**	**N_{60}**
269.00	258.00	11.00	30	11	15	8	12	0.8	9
258.00	257.00	22.00	32	14	19	12	15	0.8	12
257.00	254.00	23.00	30	11	15	8	12	0.8	9
254.00	252.00	26.00	32	14	19	12	15	0.8	12
252.00	251.00	28.00	30	11	15	8	12	0.8	9
251.00	248.00	29.00	32	14	19	12	15	0.8	12
248.00	248.00	32.00	32	14	19	12	15	0.8	12

Friction angles (Φ) for heap leaching pile 4 – north section

HLP:		4						
Section:		North						
Situation:		Current						
Drilling:		53-59						
GW (m):		259.00						
Level (m)		**Depth (m)**	**Φ**	**N 1**	**N 2**	**N 3**	**N. AV**	**C_E**	**N_{60}**
259.00	254.00	5.00	30	11	15	8	12	0.8	9
254.00	252.00	10.00	32	14	19	12	15	0.8	12
252.00	251.00	12.00	30	11	15	8	12	0.8	9
251.00	250.00	13.00	33	16	22	14	17	0.8	14
250.00	238.00	14.00	30	11	15	8	12	0.8	9
238.00	238.00	26.00	30	11	15	8	12	0.8	9

Preliminary liquefaction assessment due to blasting vibrations in heap leaching pile 1 – south section

Level (m)	Depth (m)	ɣ (kN/m3)	ɣ'(kN/m3)	Φ	Ko	σ (kN/m2)	σ' (kN/m2)	PGA (m/s²)	r	CSR	N_{60}	C_N	$(N_1)_{60}$	CRR ko=1	CRR ko	FoS
273.0	0.0															
272.0	1.0	21	11.2	28	0.53	21.00	11.20	0.26	0.99	0.03	7	1.00	7	0.12	0.08	2.6
271.0	2.0	21	11.2	30	0.50	42.00	22.40	0.26	0.98	0.03	9	1.00	9	0.14	0.09	2.9
270.0	3.0	21	11.2	30	0.50	63.00	33.60	0.26	0.96	0.03	9	1.00	9	0.14	0.09	2.9
269.0	4.0	21	11.2	30	0.50	84.00	44.80	0.26	0.95	0.03	9	1.00	9	0.14	0.09	3.0
268.0	5.0	21	11.2	30	0.50	105.00	56.00	0.26	0.94	0.03	9	1.00	9	0.14	0.09	3.0
267.0	6.0	21	11.2	32	0.47	126.00	67.20	0.26	0.93	0.03	12	1.00	12	0.16	0.10	3.5
266.0	7.0	21	11.2	32	0.47	147.00	78.40	0.26	0.92	0.03	12	1.00	12	0.16	0.10	3.5
265.0	8.0	21	11.2	32	0.47	168.00	89.60	0.26	0.90	0.03	12	1.00	12	0.16	0.10	3.5
264.0	9.0	21	11.2	30	0.50	189.00	100.80	0.26	0.89	0.03	9	1.00	9	0.14	0.09	3.2
263.0	10.0	21	11.2	30	0.50	210.00	112.00	0.26	0.88	0.03	9	1.00	9	0.14	0.09	3.2
262.0	11.0	21	11.2	30	0.50	231.00	123.20	0.26	0.87	0.03	9	1.00	9	0.14	0.09	3.3
261.0	12.0	21	11.2	30	0.50	252.00	134.40	0.26	0.86	0.03	9	1.00	9	0.14	0.09	3.3
260.0	13.0	21	11.2	30	0.50	273.00	145.60	0.26	0.84	0.03	9	1.00	9	0.14	0.09	3.4
259.0	14.0	21	11.2	30	0.50	294.00	156.80	0.26	0.83	0.03	9	0.74	7	0.12	0.08	3.0
258.0	15.0	21	11.2	30	0.50	315.00	168.00	0.26	0.82	0.03	9	0.70	6	0.12	0.08	3.0
257.0	16.0	21	11.2	30	0.50	336.00	179.20	0.26	0.81	0.03	9	0.67	6	0.12	0.08	3.1
256.0	17.0	21	11.2	33	0.46	357.00	190.40	0.26	0.80	0.03	14	0.64	9	0.14	0.09	3.4
255.0	18.0	21	11.2	33	0.46	378.00	201.60	0.26	0.78	0.03	14	0.62	9	0.14	0.09	3.5
254.0	19.0	21	11.2	33	0.46	399.00	212.80	0.26	0.77	0.02	14	0.59	8	0.13	0.08	3.2
253.0	20.0	21	11.2	33	0.46	420.00	224.00	0.26	0.76	0.02	14	0.57	8	0.13	0.08	3.2
252.0	21.0	21	11.2	33	0.46	441.00	235.20	0.26	0.75	0.02	14	0.55	8	0.13	0.08	3.3

Preliminary liquefaction assessment due to blasting vibrations in heap leaching pile 1 – north section

Level (m)	Depth (m)	γ (kN/m3)	γ'(kN/m3)	Φ	Ko	σ (kN/m2)	σ' (kN/m2)	PGA (m/s²)	r	CSR	N_{60}	C_N	$(N_1)_{60}$	CRR ko=1	CRR ko	FoS
259.0	0.0															
258.0	1.0	21	11.2	33	0.46	21.00	11.20	0.26	0.99	0.03	14	1.00	14	0.18	0.11	3.5
257.0	2.0	21	11.2	33	0.46	42.00	22.40	0.26	0.98	0.03	14	1.00	14	0.18	0.11	3.5
256.0	3.0	21	11.2	33	0.46	63.00	33.60	0.26	0.96	0.03	14	1.00	14	0.18	0.11	3.6
255.0	4.0	21	11.2	33	0.46	84.00	44.80	0.26	0.95	0.03	14	1.00	14	0.18	0.11	3.6
254.0	5.0	21	11.2	33	0.46	105.00	56.00	0.26	0.94	0.03	14	1.00	14	0.18	0.11	3.7
253.0	6.0	21	11.2	33	0.46	126.00	67.20	0.26	0.93	0.03	14	1.00	14	0.18	0.11	3.7
252.0	7.0	21	11.2	33	0.46	147.00	78.40	0.26	0.92	0.03	14	1.00	14	0.18	0.11	3.8
251.0	8.0	21	11.2	33	0.46	168.00	89.60	0.26	0.90	0.03	14	1.00	14	0.18	0.11	3.8
250.0	9.0	21	11.2	33	0.46	189.00	100.80	0.26	0.89	0.03	14	1.00	14	0.18	0.11	3.9
249.0	10.0	21	11.2	33	0.46	210.00	112.00	0.26	0.88	0.03	14	1.00	14	0.18	0.11	3.9
248.0	11.0	21	11.2	33	0.46	231.00	123.20	0.26	0.87	0.03	14	1.00	14	0.18	0.11	4.0
247.0	12.0	21	11.2	33	0.46	252.00	134.40	0.26	0.86	0.03	14	1.00	14	0.18	0.11	4.0
246.0	13.0	21	11.2	30	0.50	273.00	145.60	0.26	0.84	0.03	9	1.00	9	0.14	0.09	3.4
245.0	14.0	21	11.2	36	0.41	294.00	156.80	0.26	0.83	0.03	19	1.00	19	0.24	0.14	5.4
244.0	15.0	21	11.2	36	0.41	315.00	168.00	0.26	0.82	0.03	19	0.70	14	0.18	0.11	4.0
243.0	16.0	21	11.2	30	0.50	336.00	179.20	0.26	0.81	0.03	9	0.67	6	0.12	0.08	3.1
242.0	17.0	21	11.2	36	0.41	357.00	190.40	0.26	0.80	0.03	19	0.64	12	0.16	0.10	3.8
241.0	18.0	21	11.2	33	0.46	378.00	201.60	0.26	0.78	0.03	14	0.62	9	0.14	0.09	3.5
240.0	19.0	21	11.2	36	0.41	399.00	212.80	0.26	0.77	0.02	19	0.59	11	0.16	0.09	3.8

Preliminary liquefaction assessment due to blasting vibrations in heap leaching pile 2 – south section

Level (m)	Depth (m)	γ (kN/m3)	γ'(kN/m3)	Φ	Ko	σ (kN/m2)	σ' (kN/m2)	PGA (m/s²)	r	CSR	N_{60}	C_N	$(N_1)_{60}$	CRR ko=1	CRR ko	FoS
282.0	0.0															
281.0	1.0	21	11.2	28	0.53	21.00	11.20	0.23	0.99	0.03	7	1.00	7	0.12	0.08	3.0
280.0	2.0	21	11.2	28	0.53	42.00	22.40	0.23	0.98	0.03	7	1.00	7	0.12	0.08	3.0
279.0	3.0	21	11.2	28	0.53	63.00	33.60	0.23	0.96	0.03	7	1.00	7	0.12	0.08	3.1
278.0	4.0	21	11.2	30	0.50	84.00	44.80	0.23	0.95	0.03	9	1.00	9	0.14	0.09	3.4
277.0	5.0	21	11.2	30	0.50	105.00	56.00	0.23	0.94	0.03	9	1.00	9	0.14	0.09	3.4
276.0	6.0	21	11.2	30	0.50	126.00	67.20	0.23	0.93	0.03	9	1.00	9	0.14	0.09	3.5
275.0	7.0	21	11.2	30	0.50	147.00	78.40	0.23	0.92	0.03	9	1.00	9	0.14	0.09	3.5
274.0	8.0	21	11.2	30	0.50	168.00	89.60	0.23	0.90	0.03	9	1.00	9	0.14	0.09	3.5
273.0	9.0	21	11.2	30	0.50	189.00	100.80	0.23	0.89	0.03	9	1.00	9	0.14	0.09	3.6
272.0	10.0	21	11.2	30	0.50	210.00	112.00	0.23	0.88	0.03	9	1.00	9	0.14	0.09	3.6
271.0	11.0	21	11.2	30	0.50	231.00	123.20	0.23	0.87	0.02	9	1.00	9	0.14	0.09	3.7
270.0	12.0	21	11.2	30	0.50	252.00	134.40	0.23	0.86	0.02	9	1.00	9	0.14	0.09	3.7
269.0	13.0	21	11.2	30	0.50	273.00	145.60	0.23	0.84	0.02	9	1.00	9	0.14	0.09	3.8
268.0	14.0	21	11.2	30	0.50	294.00	156.80	0.23	0.83	0.02	9	1.00	9	0.14	0.09	3.9
267.0	15.0	21	11.2	30	0.50	315.00	168.00	0.23	0.82	0.02	9	0.70	6	0.12	0.08	3.4
266.0	16.0	21	11.2	30	0.50	336.00	179.20	0.23	0.81	0.02	9	0.67	6	0.12	0.08	3.5
265.0	17.0	21	11.2	30	0.50	357.00	190.40	0.23	0.80	0.02	9	0.64	6	0.12	0.08	3.5
264.0	18.0	21	11.2	30	0.50	378.00	201.60	0.23	0.78	0.02	9	0.62	6	0.12	0.08	3.6
263.0	19.0	21	11.2	30	0.50	399.00	212.80	0.23	0.77	0.02	9	0.59	5	0.11	0.07	3.3
262.0	20.0	21	11.2	30	0.50	420.00	224.00	0.23	0.76	0.02	9	0.57	5	0.11	0.07	3.4

Preliminary liquefaction assessment due to blasting vibrations in heap leaching pile 2 – north section

Level (m)	Depth (m)	γ (kN/m3)	γ'(kN/m3)	Φ	Ko	σ (kN/m2)	σ' (kN/m2)	PGA (m/s²)	r	CSR	N_{60}	C_N	$(N_1)_{60}$	CRR ko=1	CRR ko	FoS
271.0	0.0															
270.0	1.0	21	11.2	28	0.53	21.00	11.20	0.23	0.99	0.03	7	1.00	7	0.12	0.08	3.0
269.0	2.0	21	11.2	28	0.53	42.00	22.40	0.23	0.98	0.03	7	1.00	7	0.12	0.08	3.0
268.0	3.0	21	11.2	28	0.53	63.00	33.60	0.23	0.96	0.03	7	1.00	7	0.12	0.08	3.1
267.0	4.0	21	11.2	28	0.53	84.00	44.80	0.23	0.95	0.03	7	1.00	7	0.12	0.08	3.1
266.0	5.0	21	11.2	28	0.53	105.00	56.00	0.23	0.94	0.03	7	1.00	7	0.12	0.08	3.1
265.0	6.0	21	11.2	28	0.53	126.00	67.20	0.23	0.93	0.03	7	1.00	7	0.12	0.08	3.2
264.0	7.0	21	11.2	32	0.47	147.00	78.40	0.23	0.92	0.03	12	1.00	12	0.16	0.10	4.0
263.0	8.0	21	11.2	30	0.50	168.00	89.60	0.23	0.90	0.03	9	1.00	9	0.14	0.09	3.5
262.0	9.0	21	11.2	30	0.50	189.00	100.80	0.23	0.89	0.03	9	1.00	9	0.14	0.09	3.6
261.0	10.0	21	11.2	30	0.50	210.00	112.00	0.23	0.88	0.03	9	1.00	9	0.14	0.09	3.6
260.0	11.0	21	11.2	30	0.50	231.00	123.20	0.23	0.87	0.02	9	1.00	9	0.14	0.09	3.7
259.0	12.0	21	11.2	30	0.50	252.00	134.40	0.23	0.86	0.02	9	1.00	9	0.14	0.09	3.7
258.0	13.0	21	11.2	32	0.47	273.00	145.60	0.23	0.84	0.02	12	1.00	12	0.16	0.10	4.3
257.0	14.0	21	11.2	30	0.50	294.00	156.80	0.23	0.83	0.02	9	1.00	9	0.14	0.09	3.9
256.0	15.0	21	11.2	30	0.50	315.00	168.00	0.23	0.82	0.02	9	0.70	6	0.12	0.08	3.4
255.0	16.0	21	11.2	32	0.47	336.00	179.20	0.23	0.81	0.02	12	0.67	8	0.13	0.08	3.5
254.0	17.0	21	11.2	32	0.47	357.00	190.40	0.23	0.80	0.02	12	0.64	8	0.13	0.08	3.6
253.0	18.0	21	11.2	32	0.47	378.00	201.60	0.23	0.78	0.02	12	0.62	7	0.12	0.08	3.5
252.0	19.0	21	11.2	32	0.47	399.00	212.80	0.23	0.77	0.02	12	0.59	7	0.12	0.08	3.6
251.0	20.0	21	11.2	32	0.47	420.00	224.00	0.23	0.76	0.02	12	0.57	7	0.12	0.08	3.6
250.0	21.0	21	11.2	32	0.47	441.00	235.20	0.23	0.75	0.02	12	0.55	7	0.12	0.08	3.7
249.0	22.0	21	11.2	32	0.47	462.00	246.40	0.23	0.74	0.02	12	0.53	6	0.12	0.08	3.7
248.0	23.0	21	11.2	32	0.47	483.00	257.60	0.23	0.72	0.02	12	0.51	6	0.12	0.08	3.8

247.0	24.0	21	11.2	30	0.50	504.00	268.80	0.23	0.71	0.02	9	0.49	4	0.10	0.07	3.3
246.0	25.0	21	11.2	30	0.50	525.00	280.00	0.23	0.70	0.02	9	0.48	4	0.10	0.07	3.3
245.0	26.0	21	11.2	30	0.50	546.00	291.20	0.23	0.69	0.02	9	0.46	4	0.10	0.07	3.4

Preliminary liquefaction assessment due to blasting vibrations in heap leaching pile 3 – south section

Level (m)	**Depth (m)**	**ɣ (kN/m3)**	**ɣ'(kN/m3)**	**Φ**	**Ko**	**σ (kN/m2)**	**σ' (kN/m2)**	**PGA (m/s²)**	**r**	**CSR**	**N_{60}**	**C_N**	**$(N_1)_{60}$**	**CRR ko=1**	**CRR ko**	**FoS**
278.0	0.0															
277.0	1.0	21	11.2	30	0.50	21.00	11.20	0.19	0.99	0.02	9	1.00	9	0.14	0.09	3.9
276.0	2.0	21	11.2	30	0.50	42.00	22.40	0.19	0.98	0.02	9	1.00	9	0.14	0.09	4.0
275.0	3.0	21	11.2	30	0.50	63.00	33.60	0.19	0.96	0.02	9	1.00	9	0.14	0.09	4.0
274.0	4.0	21	11.2	30	0.50	84.00	44.80	0.19	0.95	0.02	9	1.00	9	0.14	0.09	4.1
273.0	5.0	21	11.2	33	0.46	105.00	56.00	0.19	0.94	0.02	14	1.00	14	0.18	0.11	5.0
272.0	6.0	21	11.2	30	0.50	126.00	67.20	0.19	0.93	0.02	9	1.00	9	0.14	0.09	4.2
271.0	7.0	21	11.2	30	0.50	147.00	78.40	0.19	0.92	0.02	9	1.00	9	0.14	0.09	4.2
270.0	8.0	21	11.2	36	0.41	168.00	89.60	0.19	0.90	0.02	19	1.00	19	0.24	0.14	6.8
269.0	9.0	21	11.2	32	0.47	189.00	100.80	0.19	0.89	0.02	12	1.00	12	0.16	0.10	4.9
268.0	10.0	21	11.2	32	0.47	210.00	112.00	0.19	0.88	0.02	12	1.00	12	0.16	0.10	5.0
267.0	11.0	21	11.2	32	0.47	231.00	123.20	0.19	0.87	0.02	12	1.00	12	0.16	0.10	5.1
266.0	12.0	21	11.2	32	0.47	252.00	134.40	0.19	0.86	0.02	12	1.00	12	0.16	0.10	5.1
265.0	13.0	21	11.2	32	0.47	273.00	145.60	0.19	0.84	0.02	12	1.00	12	0.16	0.10	5.2
264.0	14.0	21	11.2	32	0.47	294.00	156.80	0.19	0.83	0.02	12	1.00	12	0.16	0.10	5.3
263.0	15.0	21	11.2	32	0.47	315.00	168.00	0.19	0.82	0.02	12	0.70	9	0.14	0.09	4.6
262.0	16.0	21	11.2	32	0.47	336.00	179.20	0.19	0.81	0.02	12	0.67	8	0.13	0.08	4.2

Preliminary liquefaction assessment due to blasting vibrations in heap leaching pile 3 – north section

Level (m)	Depth (m)	γ (kN/m3)	γ'(kN/m3)	Φ	Ko	σ (kN/m2)	σ' (kN/m2)	PGA (m/s²)	r	CSR	N_{60}	C_N	$(N_1)_{60}$	CRR ko=1	CRR ko	FoS
249.0	0.0															
248.0	1.0	21	11.2	28	0.53	21.00	11.20	0.19	0.99	0.02	7	1.00	7	0.12	0.08	3.6
247.0	2.0	21	11.2	28	0.53	42.00	22.40	0.19	0.98	0.02	7	1.00	7	0.12	0.08	3.7
246.0	3.0	21	11.2	28	0.53	63.00	33.60	0.19	0.96	0.02	7	1.00	7	0.12	0.08	3.7
245.0	4.0	21	11.2	28	0.53	84.00	44.80	0.19	0.95	0.02	7	1.00	7	0.12	0.08	3.7
244.0	5.0	21	11.2	28	0.53	105.00	56.00	0.19	0.94	0.02	7	1.00	7	0.12	0.08	3.8
243.0	6.0	21	11.2	28	0.53	126.00	67.20	0.19	0.93	0.02	7	1.00	7	0.12	0.08	3.8
242.0	7.0	21	11.2	28	0.53	147.00	78.40	0.19	0.92	0.02	7	1.00	7	0.12	0.08	3.9
241.0	8.0	21	11.2	30	0.50	168.00	89.60	0.19	0.90	0.02	9	1.00	9	0.14	0.09	4.3
240.0	9.0	21	11.2	30	0.50	189.00	100.80	0.19	0.89	0.02	9	1.00	9	0.14	0.09	4.4
239.0	10.0	21	11.2	30	0.50	210.00	112.00	0.19	0.88	0.02	9	1.00	9	0.14	0.09	4.4
238.0	11.0	21	11.2	30	0.50	231.00	123.20	0.19	0.87	0.02	9	1.00	9	0.14	0.09	4.5
237.0	12.0	21	11.2	30	0.50	252.00	134.40	0.19	0.86	0.02	9	1.00	9	0.14	0.09	4.5
236.0	13.0	21	11.2	32	0.47	273.00	145.60	0.19	0.84	0.02	12	1.00	12	0.16	0.10	5.2
235.0	14.0	21	11.2	32	0.47	294.00	156.80	0.19	0.83	0.02	12	1.00	12	0.16	0.10	5.3
234.0	15.0	21	11.2	33	0.46	315.00	168.00	0.19	0.82	0.02	14	0.70	10	0.15	0.10	4.9
233.0	16.0	21	11.2	30	0.50	336.00	179.20	0.19	0.81	0.02	9	0.67	6	0.12	0.08	4.2
232.0	17.0	21	11.2	30	0.50	357.00	190.40	0.19	0.80	0.02	9	0.64	6	0.12	0.08	4.3
231.0	18.0	21	11.2	32	0.47	378.00	201.60	0.19	0.78	0.02	12	0.62	8	0.13	0.08	4.4
230.0	19.0	21	11.2	32	0.47	399.00	212.80	0.19	0.77	0.02	12	0.59	7	0.12	0.08	4.3
229.0	20.0	21	11.2	32	0.47	420.00	224.00	0.19	0.76	0.02	12	0.57	7	0.12	0.08	4.4

Preliminary liquefaction assessment due to blasting vibrations in heap leaching pile 4 – south section

Level (m)	Depth (m)	γ (kN/m3)	γ'(kN/m3)	Φ	Ko	σ (kN/m2)	σ' (kN/m2)	PGA (m/s²)	r	CSR	N_{60}	C_N	$(N_1)_{60}$	CRR ko=1	CRR ko	FoS
269.0	0.0															
268.0	1.0	21	11.2	30	0.50	21.00	11.20	0.17	0.99	0.02	9	1.00	9	0.14	0.09	4.4
267.0	2.0	21	11.2	30	0.50	42.00	22.40	0.17	0.98	0.02	9	1.00	9	0.14	0.09	4.4
266.0	3.0	21	11.2	30	0.50	63.00	33.60	0.17	0.96	0.02	9	1.00	9	0.14	0.09	4.5
265.0	4.0	21	11.2	30	0.50	84.00	44.80	0.17	0.95	0.02	9	1.00	9	0.14	0.09	4.6
264.0	5.0	21	11.2	30	0.50	105.00	56.00	0.17	0.94	0.02	9	1.00	9	0.14	0.09	4.6
263.0	6.0	21	11.2	30	0.50	126.00	67.20	0.17	0.93	0.02	9	1.00	9	0.14	0.09	4.7
262.0	7.0	21	11.2	30	0.50	147.00	78.40	0.17	0.92	0.02	9	1.00	9	0.14	0.09	4.7
261.0	8.0	21	11.2	30	0.50	168.00	89.60	0.17	0.90	0.02	9	1.00	9	0.14	0.09	4.8
260.0	9.0	21	11.2	30	0.50	189.00	100.80	0.17	0.89	0.02	9	1.00	9	0.14	0.09	4.9
259.0	10.0	21	11.2	30	0.50	210.00	112.00	0.17	0.88	0.02	9	1.00	9	0.14	0.09	4.9
258.0	11.0	21	11.2	32	0.47	231.00	123.20	0.17	0.87	0.02	12	1.00	12	0.16	0.10	5.6
257.0	12.0	21	11.2	30	0.50	252.00	134.40	0.17	0.86	0.02	9	1.00	9	0.14	0.09	5.1
256.0	13.0	21	11.2	30	0.50	273.00	145.60	0.17	0.84	0.02	9	1.00	9	0.14	0.09	5.1
255.0	14.0	21	11.2	30	0.50	294.00	156.80	0.17	0.83	0.02	9	1.00	9	0.14	0.09	5.2
254.0	15.0	21	11.2	32	0.47	315.00	168.00	0.17	0.82	0.02	12	0.70	9	0.14	0.09	5.1
253.0	16.0	21	11.2	32	0.47	336.00	179.20	0.17	0.81	0.02	12	0.67	8	0.13	0.08	4.7
252.0	17.0	21	11.2	30	0.50	357.00	190.40	0.17	0.80	0.02	9	0.64	6	0.12	0.08	4.8
251.0	18.0	21	11.2	32	0.47	378.00	201.60	0.17	0.78	0.02	12	0.62	8	0.13	0.08	4.9
250.0	19.0	21	11.2	32	0.47	399.00	212.80	0.17	0.77	0.02	12	0.59	7	0.12	0.08	4.9
249.0	20.0	21	11.2	32	0.47	420.00	224.00	0.17	0.76	0.02	12	0.57	7	0.12	0.08	4.9

Preliminary liquefaction assessment due to blasting vibrations in heap leaching pile 4 – north section

Level (m)	Depth (m)	γ (kN/m3)	γ'(kN/m3)	Φ	Ko	σ (kN/m2)	σ' (kN/m2)	PGA (m/s²)	r	CSR	N_{60}	C_N	$(N_1)_{60}$	CRR ko=1	CRR ko	FoS
259.0	0.0															
258.0	1.0	21	11.2	30	0.50	21.00	11.20	0.17	0.99	0.02	9	1.00	9	0.14	0.09	4.4
257.0	2.0	21	11.2	30	0.50	42.00	22.40	0.17	0.98	0.02	9	1.00	9	0.14	0.09	4.4
256.0	3.0	21	11.2	30	0.50	63.00	33.60	0.17	0.96	0.02	9	1.00	9	0.14	0.09	4.5
255.0	4.0	21	11.2	30	0.50	84.00	44.80	0.17	0.95	0.02	9	1.00	9	0.14	0.09	4.6
254.0	5.0	21	11.2	32	0.47	105.00	56.00	0.17	0.94	0.02	12	1.00	12	0.16	0.10	5.2
253.0	6.0	21	11.2	32	0.47	126.00	67.20	0.17	0.93	0.02	12	1.00	12	0.16	0.10	5.3
252.0	7.0	21	11.2	30	0.50	147.00	78.40	0.17	0.92	0.02	9	1.00	9	0.14	0.09	4.7
251.0	8.0	21	11.2	33	0.46	168.00	89.60	0.17	0.90	0.02	14	1.00	14	0.18	0.11	5.8
250.0	9.0	21	11.2	30	0.50	189.00	100.80	0.17	0.89	0.02	9	1.00	9	0.14	0.09	4.9
249.0	10.0	21	11.2	30	0.50	210.00	112.00	0.17	0.88	0.02	9	1.00	9	0.14	0.09	4.9
248.0	11.0	21	11.2	30	0.50	231.00	123.20	0.17	0.87	0.02	9	1.00	9	0.14	0.09	5.0
247.0	12.0	21	11.2	30	0.50	252.00	134.40	0.17	0.86	0.02	9	1.00	9	0.14	0.09	5.1
246.0	13.0	21	11.2	30	0.50	273.00	145.60	0.17	0.84	0.02	9	1.00	9	0.14	0.09	5.1
245.0	14.0	21	11.2	30	0.50	294.00	156.80	0.17	0.83	0.02	9	1.00	9	0.14	0.09	5.2
244.0	15.0	21	11.2	30	0.50	315.00	168.00	0.17	0.82	0.02	9	0.70	6	0.12	0.08	4.6
243.0	16.0	21	11.2	30	0.50	336.00	179.20	0.17	0.81	0.02	9	0.67	6	0.12	0.08	4.7
242.0	17.0	21	11.2	30	0.50	357.00	190.40	0.17	0.80	0.02	9	0.64	6	0.12	0.08	4.8
241.0	18.0	21	11.2	30	0.50	378.00	201.60	0.17	0.78	0.02	9	0.62	6	0.12	0.08	4.8
240.0	19.0	21	11.2	30	0.50	399.00	212.80	0.17	0.77	0.02	9	0.59	5	0.11	0.07	4.5
239.0	20.0	21	11.2	30	0.50	420.00	224.00	0.17	0.76	0.02	9	0.57	5	0.11	0.07	4.6