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GUIDELINES FOR FIELD ASSESSMENT OF LIQUEFACTION HAZARD IN URBAN AREAS THROUGH GEOPHYSICAL METHODS

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Abstract

Liquefaction phenomena as an earthquake-induced effect in loose saturated soils is considered very dangerous for structures founded over such materials. Peru's capital and its surrounding districts are located over alluvial deposits with a shallow water table, so it can be inferred that liquefaction potential is high. In addition, Peru's capital is currently in a process of fast urban expansion which makes lots of people to settle in zones that are prone to liquefaction posing a risk to housing and infrastructure projects. The common practice for liquefaction potential assessment recommends field testing such as SPT-based and CPT-based methods although a more cost-effective method is the use of geophysical methods which are not destructive and faster. The latter suits the need of a developing country which lacks of enough funding for soil exploration using SPT or CPT. In recent years there have developed several methodologies to include geophysical parameters in liquefaction potential assessment. However, the reliability of such methodologies in places far from where they were developed must be tested in order to properly apply them. Therefore, this paper reviews geophysical-based correlations for liquefaction assessment aiming to perform a comparative analysis among them and SPT-based methods. Furthermore, the paper aims to establish guidelines for liquefaction hazard assessment in three levels of analysis starting from the qualitative one which shows the importance of geomorphological and geologic conditions, then it is important to evaluate past behavior of soils during earthquakes to finally estimate the factor of safety. Findings indicate that geophysical-based methods for assessing liquefaction potential have similar results to those obtained following SPT-based methods except in the range of soils with moderately stiffness (i.e. shear wave velocity is between 185 m/sec to 250 m/sec) whose liquefaction potential is difficult to predict.

Keywords: Liquefaction Potential Assessment, Geophysical Methods, MASW, Moderately Stiff soils

1. Introduction

Liquefaction occurs when the soil's shear strength suddenly plummets due to an increase in porewater pressure induced by dynamic loading. This event is problematic in geotechnics since it can cause failure in structures founded on liquefiable soils.

Lima is placed in a highly seismic area and is also home of approximately 32% of Peru's total population. Both facts made Lima to be very vulnerable in case an earthquake hits. There is historical evidence that in some areas like San Vicente de Cañete -located 144 km in the southern part of Lima- occurred liquefaction during the earthquakes of 1948 (Silgado, 1978) [1] and 1974 (Huaco et al, 1975) [2]. However, seismic microzonation of such areas including estimation of liquefaction hazard zones are not available. This issue is due to several factors, one of them is the high costs of using direct methods for geotechnical exploration (such as boreholes, SPT, CPT, among others) to evaluate the liquefaction potential. Another factor is the lack of validation of indirect methods of liquefaction potential assessment like those based on measuring surface waves. These have been developed in an empirical fashion in other conditions. As a result, it is important to analyze the applicability of these methods to soils of different stress history and genetic conditions.

This paper proposes guidelines to assess liquefaction hazard in urban areas using geophysical methods to estimate the liquefaction potential aiming to reduce costs of geotechnical exploration. The applicability of these geophysical-based methods will be tested by comparing their results against those based on direct exploration like the widely-used SPT, and with historical seismic and geologic evidence of the area.

2. Research Methodology

This paper aims to propose guidelines to estimate liquefaction hazard in urban areas through geophysical methods. The Guideline will study such hazard in 3 levels starting from a qualitative one to then calculate the Factor of Safety. To evaluate the guideline's effectiveness, the district of San Vicente de Cañete will be used as Case Study. This is an urban area of fast population growth and whose geological conditions (alluvial deposits with high water table), and historical evidence (sandboils were reported to occur in past earthquakes) make it representative. The research methodology is shown below:

- ✓ Information related to geology, geomorphology and seismicity of San Vicente de Cañete will be gathered to preliminary assess the liquefaction hazard.
- ✓ Geotechnical Exploration of the area will be carried out using direct methods (pits and Dynamic Probing Light), and indirect (MASW)
- ✓ Liquefaction potential will be evaluated through the deterministic criteria using methods based on direct and indirect exploration
- ✓ A comparative analysis of the results will be performed
- ✓ Finally, the applicability of geophysical-based methods will be discussed and recommendations for further research will be proposed

3. Theoretical Framework

The most widely-used approach to evaluate the liquefaction potential is the deterministic criteria which estimates the factor of safety by comparing seismic-induced stresses (CSR) and the cyclic resistance of soils (CRR).

$$FS = \frac{CRR}{CSR} \tag{1}$$

CSR can be calculated by applying simplified methods proposed by Seed and Idriss (1971) [3], while CRR requires in-situ or laboratory tests. Currently, it is accepted that factors like fabric, historical seismic strain and overconsolidation ratio have an important effect on the cyclic resistance of soils. These factors cannot be accurately modelled in laboratory, so it is more suitable to use field tests (Kramer, 1996) [4]. The SPT test is widely used to empirically calculate a soil's CRR based on correlations proposed by Idriss and Boulanger (2008) [5]. Shear wave velocity can also be used since it has a theoretical direct relation with soil's resistance to liquefaction because, like the shear modulus, they both increase with density (Towhata, 2008) [6]. In this regard, there are three approaches to establish a correlation between shear-wave velocity and liquefaction resistance (Chen et al, 2005) [7]:

- ✓ Methods that combine field tests to measure shear wave velocity, and laboratory tests to evaluate the soil's resistance to liquefaction like those proposed by Tokimtasu and Uchida (1990), Alba et al (1984), and Hatanaka et al (1997).
- ✓ Methods based only on field tests like Andrus and Stokoe (2000), and Stokoe et al (1988).
- ✓ Methods that use correlations between the SPT test and the shear wave velocity (Seed et al, 1983)

The second approach is used as a cost-effective method to evaluate liquefaction hazard. Laboratory tests to evaluate liquefaction resistance have many complexities since it is difficult to obtain undisturbed samples.

3.1 SPT-based method

Seed and Idriss (1971) used the rigid block analogy to propose an equation to calculate the resistance of soils to cyclic stresses during earthquakes. This equation (2) depends upon the maximum soil's acceleration $(a_{\max})$, total and effective vertical stress $(\sigma_0 \, y \, \sigma_0{}')$ and a factor of stress reduction (r_d) that decreases along wih depth.

$$CSR = \frac{\tau_d}{\sigma_0{}'} = 0.65 * \frac{a_{max}}{g} * \frac{\sigma_0}{\sigma_0{}'} * r_d \qquad (2)$$

Cyclic resistance of soils can be calculated using the number of blows of the SPT test. Idriss and Boulanger (2008) stated equation 3 for clean sands:

$$CRR_{M=7.5,\,\sigma'_{vc}=1} = \exp(\frac{(N1)_{60cs}}{14.1} + (\frac{(N1)_{60cs}}{126})^2 - (\frac{(N1)_{60cs}}{23.6})^3 + (\frac{(N1)_{60cs}}{25.4})^4 - 2.8) \qquad (3)$$

3.2 Methods based on shear wave velocity

In this paper, it will be used the methodology proposed by Andrus and Stokoe (2000) [8] for the estimation of the cyclic resistance of soils.

$$CRR = 0.022 * \left(\frac{K_c * V_{S1}}{100}\right)^2 + 2.8 * \left(\frac{1}{V_{S1C} - K_c * V_{S1}} - \frac{1}{V_{S1C}}\right) * MSF \qquad (4)$$

Where:

- ✓ $MSF = (\frac{Mw}{7.5})^{-2.56}$ where Mw is the magnitude of the design earthquake

- ✓ $Vs1 = Vs * (\frac{Pa}{\sigma'v})^{0.25}$ where Vs is the corrected shear wave velocity in m/sec, Pa is a referential pressure which is generally 100 Kpa and $\sigma'v$ is the effective confinement stress
- ✓ Kc is a factor that quantifies the effects of cementation and sedimentation of deposits. It is considered 1 given the lack of methods to reliably estimate it

The amount of fines FC (%) intervenes in the computation model through the variable V_{S1c} that is applied according to:

- ✓ $(V_{S1})_{cs} = 220$ for a content of fine (% FC) less than 5%
- ✓ $220 < (V_{S1})_{cs} \leq 200$ for a content of fine (% FC) above 5% and less than 35%
- ✓ $(V_{S1})_{cs} = 200$ for a content of fine (% FC) greater than 35%

4. Liquefaction Potential Assessment

It will be considered 3 levels of evaluation (see figure 1). The first is qualitative and is based on geomorphological conditions of soil formation and historical data. Youd and Perkins (1978) summarizes the degree of susceptibility to liquefaction according to the type of geomorphology of an area.

The second level focus on the importance of applying the Electrical Resistivity Tomography method to determine the existence of the phreatic level because it is known that for liquefaction to occur the soil must be saturated. In addition, cheap direct exploration methods like trial pits must be used as geophysical methods itself can lead to wrong interpretations. For example, shear wave velocity of non-liquefiable soft clayey soils can be similar to those of loose sandy soils that can liquefy (Andrus and Stokoe, 1996) [8]. An additional benefit of the direct exploration is that it can be known the soil particle size distribution, as well as its indexes of consistency. This is important since a liquefiable soil should be predominantly granular. The liquefaction can also occur in fine soils, but only in silts of low plasticity which meet the following criteria (Wang, 1979):

- ✓ Fine fraction less than 0.005mm < 15%
- ✓ Liquid Limit (LL) < 35%
- ✓ Natural Moisture Content > 0.9LL
- ✓ Liquidity index < 0.75

Then, the third level will require the estimation of the maximum probable earthquake in the area in order to apply the methodology of Seed and Idriss (1971) and Andrus and Stokoe (2000) to determine the values of the CSR and the CRR respectively. The probability of liquefaction (PL) shall be determined on the basis of the FS (safety factor) and according to what is established by Juang et al (2001) [10]: (5)

$$P_L = \frac{1}{1 + \left(\frac{FS}{0.73}\right)^{3.4}} \quad (5)$$

For "moderately stiff" soils (Vs between 185 m/sec to 250 m/sec), liquefaction shall be subject to a more detailed analysis because in such interval the limit of applicability of the methodology is reached. Kayen et al (2013) [11] indicated that the shear wave velocity limits suggested by Andrus and Stokoe (2000) (215 m/sec as maximum for liquefiable soils) at high levels of CSR (greater than 0.35) are somewhat conservative since, in the past ten years, new cases of study were found in which existed liquefiable soils whose potential is not properly calculated by the empirical models of Andrus and Stokoe (2000). It is important to indicate that the range of shear wave velocity proposed is similar to that recommended by the Uniform Building Code (UBC), but its upper limit is set based on the findings of Kayen et al (2013).

Finally, “moderately stiff” soils should be studied more in depth in cases where high values of stress induced by earthquakes are expected (CSR greater than 0.35). It is suggested to use the constitutive model UBCSand to perform numerical analysis.

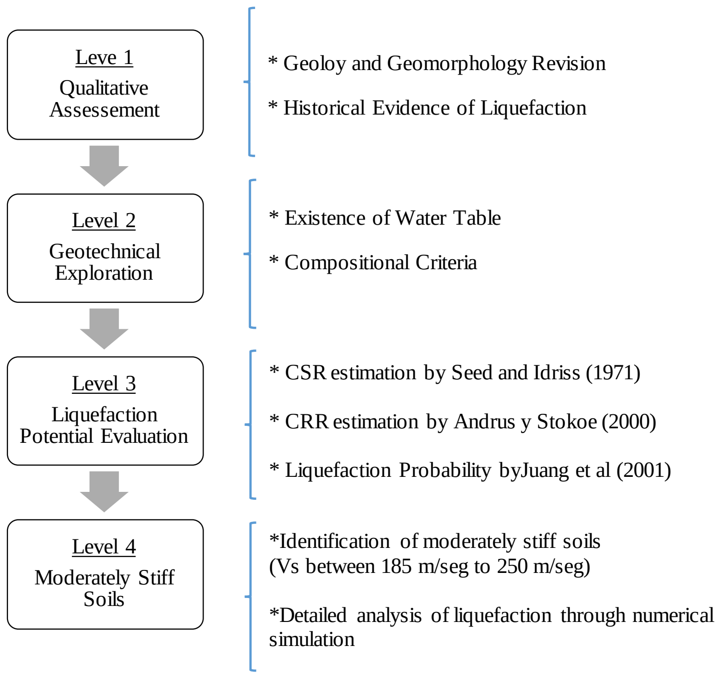


Figure 1 Proposal for liquefaction hazard assessment

4.1 Assessment of Liquefaction Hazard in San Vicente de Cañete

4.1.1 Geology and Geomorphology

Most of the urban area of San Vicente de Cañete is founded on alluvial deposits from the Holocene forming a terrace as can be seen in the following satellite image (figure 2):

Figure 2 Satellite Image of the area of study

According to Youd and Perkins (1978), the probability of Holocene sediments deposited on an alluvial terrace to liquefy is moderate to low. However, this qualitative indicator is not sufficient to determine the hazard of liquefaction of soils in the area.

4.1.2 Historical Seismicity and Soil Behavior During Past Earthquakes

According to historical records, soil liquefaction has occurred in two opportunities in the district of San Vicente de Cañete. The first occurred on 28 May 1948. A strong earthquake caused several landslides in swampy terrain. At the foot of the Hill Candela cracks were formed, observing in the place small landslides due to the saturation of the terrain (Silgado, 1978). The maximum intensity of this earthquake was VII MM and its magnitude was of Ms = 7.0. Anotherr earthquake took place in October 1974. According to Huaco et al (1975) and Giesecke et al (1980) [12], liquefaction occurred in the Cañete valley, where the water table is very superficial. The most important local phenomenon was in the Cooperative La Quebrada, covering an area of 30,000 m2. In the northern zone sandboils was seen. The maximum intensity of this earthquake was VIII MM and its magnitude was Ms = 7.5. This evidences that the area is exposed to earthquakes of medium to high magnitude and that has already happened the phenomenon of liquefaction.

4.1.3 Soil Exploration and Test Results

Trial pits were excavated and the Dynamic Probing Light was used since SPT equipment was not available. In addition, standard laboratory tests (density, limits of consistency and others) were carried out. The geophysical exploration encompassed 12 lines of MASW. The results allowed to define the nature of the soil in the area from which 3 types of soil were predominant: well graded gravel (GW), Silty Sand (SM) and Organic Clay (CL) whose geographical distribution and the location of the field tests are shown in figure 3. The points P1 to P7 indicate the area where trial pits and DPL were carried out, while points L1 to L12 indicate the location of the lines of the MASW test. It should be noted that water table was only found in 3 points: P2 at 0.80m, in P4 at 3.20m and P6 at 2.20m.

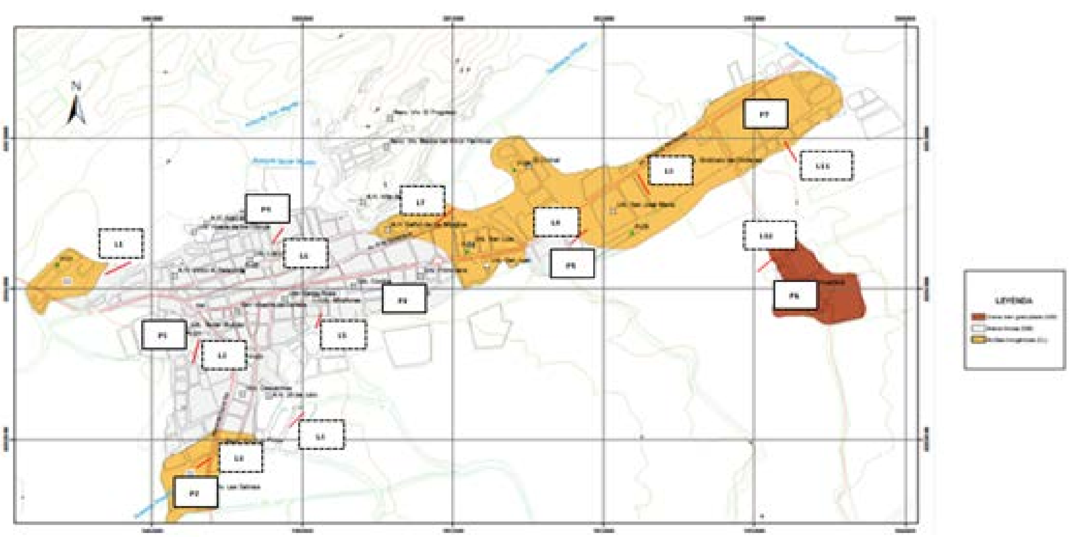


Figure 3 Distribution of soils in the area of San Vicente de Cañete.

According to the figure shown the greater part of the area under study is located on silty sand (lead color) and organic clays (light brown).

4.1.4 Compositional Criteria

The application of the compositional criteria will be used in the 3 zones that have a high water table, whose physical properties are shown in Table 1:

Table 1 Summary of physical properties of the soils of the selected zones

Exploration point	Depth (m)	Water Table Level (m)	Gravel (%)	Sand (%)	Fines (%)	Liquid Limit (%)	Plastic Limit (%)	Plastic Index (%)	SUCS classification	Type of Soil
P2	0.8	0.8	-	47.4	52.6	44	28	16	CL	Silty Clay
P4	3.2	3.2	-	55	45	24	16	8	SM	Silty Sand
P6	2.2	2.2	70	27	3	26	19	7	GW	Well-graded Gravel

As can be observed, soils corresponding to P2 are discarded due to the high amount of fines (52.6%). The soils of P4 are in accordance with the compositional criteria as they are sandy and their contents of fine has low plasticity (LL less than 35). In the case of soils of P6, liquefaction is unlikely because there are well-graded gravels that favor drainage. However, it will be evaluated liquefaction potential since it has an important matrix of sand.

4.1.5 Earthquake Hazard Assessment

Case study is located in the peruvian coast where the mechanisms of subduction have generated large-magnitude earthquakes. With the purpose of assessing the potential of liquefaction of soils, it is necessary to estimate the magnitude of a probable earthquake. This was done by applying the probabilistic method and taking as a reference the seismogenic sources, and laws of attenuation suggested by Gamarra and Aguilar (2009) [13]. In addition, it was used as a reference the earthquake hazard study made for the design of tailings dams of the CONDESTABLE [14] mining project located in the city of "Mala" at 50 km from the district of San Vicente de Cañete. The result of the study provides a maximum acceleration of 0.19g and an earthquake of 7.05MW for a period of return of 100 years, 0.27g (7.35 Mw) for 200 years, and 0.40g (7.91 MW) for 475 years of period of return.

4.1.6 SPT-based results

It will be applied the methodology of seed and Boulanger (2008) for soils with fine content greater than 35% in the 2 points where it was likely the occurrence of liquefaction. There were no data of SPT, but from the DPL test so that it will be used expressions of correlation proposed by Lingwanda et al (2014) [15]:

$$N_{60} = 1.01 * N_{10} + 0.44 \qquad (6)$$

What is virtually equivalent to the same number of blows. From here the depth curve Vs safety factor can be plotted whose results are summarized in Table 2 and 3. The minimum factor of safety is 1.1 according to what is recommended by the Southern California Earthquake Center (SCEC).

Table 2 Potential and probability of liquefaction in the point P4

Depth (m)	N field	Results					
		TR = 100 years		TR = 200 years		TR = 475 years	
		FS	PL	FS	PL	FS	PL
0.3	1	-	0.00%	-	0.00%	-	0.00%
0.6	2	-	0.00%	-	0.00%	-	0.00%
0.9	2	-	0.00%	-	0.00%	-	0.00%
1.2	3	-	0.00%	-	0.00%	-	0.00%
1.5	5	-	0.00%	-	0.00%	-	0.00%
1.8	5	-	0.00%	-	0.00%	-	0.00%
2.1	5	-	0.00%	-	0.00%	-	0.00%
2.4	7	-	0.00%	-	0.00%	-	0.00%
2.7	8	-	0.00%	-	0.00%	-	0.00%
3	8	-	0.00%	-	0.00%	-	0.00%
3.3	8	0.90	32.92%	0.53	74.88%	0.30	95.36%
3.6	9	0.91	32.17%	0.59	67.57%	0.32	94.14%
3.9	11	1.09	20.33%	0.67	57.65%	0.38	90.59%
4.2	11	1.09	20.33%	0.63	62.07%	0.35	92.21%
4.5	12	1.25	13.84%	0.75	47.70%	0.43	85.95%
4.8	14	1.31	12.11%	0.85	37.34%	0.47	81.47%
5	14	1.31	12.11%	0.90	32.53%	0.49	79.90%

Table 3 Potential and probability of liquefaction in the point P6

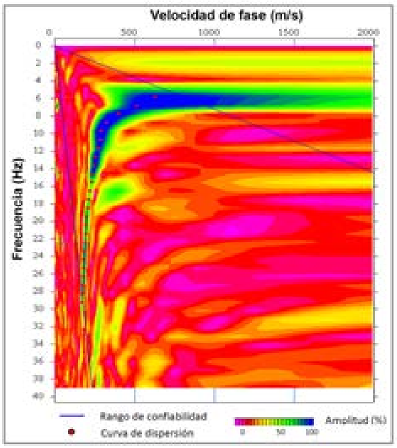
Depth (m)	N field	Results					
		TR = 100 years		TR = 200 years		TR = 475 years	
		FS	PL	FS	PL	FS	PL
0.3	2	-	0.00%	-	0.00%	-	0.00%
0.6	3	-	0.00%	-	0.00%	-	0.00%
0.9	3	-	0.00%	-	0.00%	-	0.00%
1.2	4	-	0.00%	-	0.00%	-	0.00%
1.5	6	-	0.00%	-	0.00%	-	0.00%
1.8	7	-	0.00%	-	0.00%	-	0.00%
2.1	7	-	0.00%	-	0.00%	-	0.00%
2.4	16	1.73	5.08%	1.24	14.33%	0.68	56.32%
2.7	16	1.73	5.08%	1.22	14.78%	0.67	57.65%
3	18	1.91	3.67%	1.26	13.42%	0.69	55.30%
3.3	18	1.75	4.87%	1.15	17.58%	0.64	61.14%
3.6	18	1.67	5.70%	1.05	22.65%	0.58	68.75%
3.9	21	1.77	4.70%	1.18	16.27%	0.67	57.65%
4.2	22	1.77	4.70%	1.23	14.60%	0.68	56.62%
4.5	22	2.15	2.46%	1.30	12.20%	0.73	49.80%
4.8	24	2.14	2.51%	1.42	9.50%	0.81	41.30%
5	27	2.43	1.65%	2.50	1.50%	1.86	3.99%

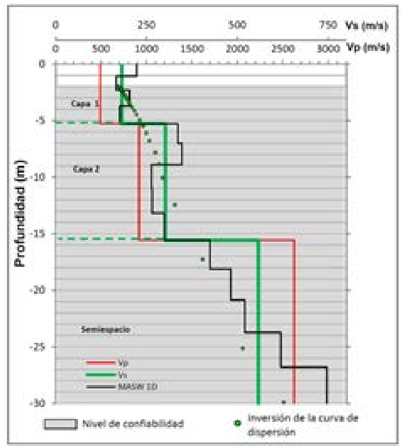
The point P4 is in danger of liquefaction for the earthquakes of 200 and 475 years of return period (moderate to high magnitude). In case of the earthquake of 100 years of return period, liquefaction could occur, but only in a small layer (0.3m) and the factor of safety is almost the minimum required so it is possible to assume that damages could be minimal.

The point P6 has no danger of liquefaction for earthquakes of 100 and 200 years of period of return. For the earthquake of 475 years of return period, liquefaction is very likely (more than 50% of probability in average). Seismic induced stress (CRR) for this magnitude were calculated in the order of 0.25.

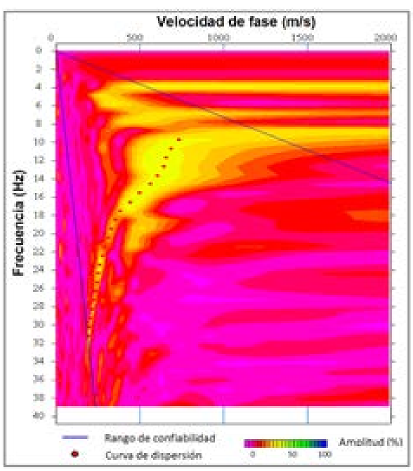
4.1.7 Vs-based results

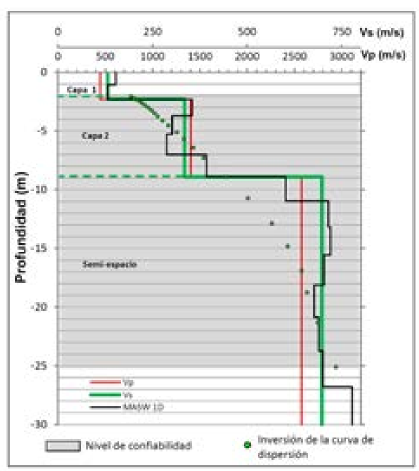
The points of geotechnical exploration P4 and P6 were made in areas close to the lines of MASW L6 and L10 respectively as can be seen in figure 3. The seismic profiles obtained (Figures 4 to 7 and tables 4 and 5) will be used to obtain the value of CRR according to the expression provided by Andrus and Stokoe (equation 4).





Figures 4 and 5 Curve of dispersion and velocity profile of the line L6





Figures 6 and 7 Curve of dispersion and velocity profile of the LINE L10

Table 4 Summary of Line L6

Seismic Layer	Thickness (m)	Vs (m/seg)	Vp (m/seg)	Density (gr/cm3)
1	5	182	493	1.4
2	10	303	918	1.6
Semi-space	-	559	2627	2.1

Table 5 Summary of Line L10

Seismic Layer	Thickness (m)	Vs (m/seg)	Vp (m/seg)	Density (gr/cm3)
1	2	131	443	1.4
2	7	298	1105	1.7
Semi-space	-	698	2573	2.1

The shear wave velocity and maximum densities were used to determine the value of the potential of liquefaction. The results are summarized in table 6.

Table 6 Potential and probability of liquefaction

MASW line	Exploration point	PGA (g)	Water Table Level (m)	Layer (m)	Results					
					TR = 100 years		TR = 200 years		TR = 475 years	
					FS	PL	FS	PL	FS	PL
L6	P4	0.19G	3.2	[3.2 - 5]	0.93	30.62%	0.59	67.23%	0.33	93.49%
L10	P6	0.19G	2.2	[2.2 - 5]	2.07	2.80%	1.71	5.29%	0.74	48.43%

5. Discussion of Results

The values of the safety factor obtained allow to infer that the methodology proposed by Andrus and Stokoe (2000) provides similar results to the method based on the SPT for the area under study. Both, SPT-based and Vs-based methods, predict the potential of liquefaction very similarly. For the point P4, in all the earthquake cases the safety factor is less than 1.1 and the liquefaction probability is similar for layers under the water table down to 5m of depth. In the point P6, liquefaction only occurs in the case of the earthquake of 475 years of period of return which was predicted using both methods.

5.1 Areas with Moderately Stiff Soils

In this case of study, there were no layers of soils within the range assumed for moderately stiff soils (Vs between 185 m/seg to 250 m/seg). However, as it was seen in other studies, such soils cause issues in predicting the occurrence of liquefaction and therefore they should be studied in a more detailed way.

6. Conclusions and Recommendations

- ✓ The comparative analysis between the method of evaluation of potential based on the SPT test and the proposed by Andrus and Stokoe (2000) allowed to establish that the latter can be used reliably for studies of liquefaction hazard of soils in the district of San Vicente de Cañete since the values of the safety factor and the likelihood of liquefaction are very similar.
- ✓ MASW lines must be complemented by studies of direct exploration as trial pits to determine the existence of the water table level (a prerequisite for the occurrence of liquefaction) and obtain samples in order to apply the compositional criteria.
- ✓ It is recommended to perform numerical analysis using the finite element method in some commercial software for study in greater detail the undrained behavior of saturated moderate stiff soils under dynamic loads. The model establishing UBCSand of Byrne et al (2004) is a good choice because it has been developed to study the phenomenon of liquefaction and presents correlations to determine its parameters from field tests which reduces the cost of performing the numerical simulation.

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