Evaluation of soil liquefaction resistance with variable energy dynamic penetration test, PANDA®: state of the art

S. López Retamales
Ecole des Ponts ParisTech, Paris, France, sebastian.lopez-retamales@enpc.fr

J.C. Dupla[1], J. Canou[2]
Ecole des Ponts ParisTech, Paris, France, jean-claude.dupla@enpc.fr[1], jean.canou@enpc.fr[2]

M. Benz Navarrete
Sol-Solution, Riom, France

ABSTRACT: Soil liquefaction is one of the most destructive problems produced during earthquakes and its analysis presents difficulties associated with taking samples and performing tests under the surface. Against this, in situ analysis method based on penetration tests (CPT, SPT) have been developed. Each penetration equipment has its own characteristics and different application possibilities, so it is necessary to expand the analysis methods to other equipment. This paper presents a state of the art on the application of the PANDA® variable energy light dynamic penetrometer in the analysis of resistance to liquefaction. An exhaustive bibliographic review of the PANDA® penetrometer developed methods has been carried out. It is concluded that the methods presented are applicable for the preliminary analysis of liquefaction resistance.

Keywords: Liquefaction, dynamic penetrometer, PANDA®, SPT, CPT.

1. Introduction

The liquefaction of soils is one of the most destructive problems associated with the earthquakes and the consequences can be disastrous and of high cost, for this reason the evaluation of the liquefaction potential is necessary, however, due to the characteristics of the soil, the laboratory work is difficult and the use of methods that involve the use of field equipment may present limitations. Therefore, the availability of different methods of analysis of potential for liquefaction in the field allows a more precise analysis..

The liquefaction potencial evaluation is based on the calculation of a safety factor that implies the ratio between the seismic request induced in a soil mass (CSR) and the resistance opposed by it (CRR) [1]. For the calculation of cyclic resistance (CRR), it is possible to use laboratory or field tests, however, the methodologies based on in situ tests have proven to be more technically and economically accessible, relegating laboratory tests to projects where the magnitude and resources involved justify the employment.

In this article, we review the current methods for evaluation of liquefaction potential in the field through the use of penetration equipment, emphasizing methods developed based on a dynamic energy penetrometer of variable energy.

Dynamic energy penetrometers of variable energy allow the execution of field penetration tests quickly and with smaller equipment compared to static penetrometers such as the Cone Penetration Test (CPT).

The use of dynamic penetrometers in the analysis of liquefaction potential is not new, the stantard penetration test (SPT) being the equipment used in the simplified procedure of Seed and idriss (1971) [1], however, it has been recognized that it presents disadvantages from the point of view of repeatability, quality control and applicability in coarse granular soils [2]. Another of the factors that impact on the accuracy of the results obtained is the need to apply constant energy by means of the fall of the striking mass from a normalized height, as is the case with the dynamic penetrometers used in the liquefaction potencial evaluation such as: SPT, Becker penetrometer and DCP [3], [4]. A dynamic variable energy penetrometer eliminates the uncertainty associated with the variability of the blow by measuring the resistance of each blow and not using a standard value associated with the number of blows required to obtain a penetration.

The PANDA® dynamic variable energy penetrometer (NF 94-105) [5], developed in France, allows to obtain soil resistance to be penetrated by a conical tip and has been used in a wide range of geotechnical applications, including development of some methodologies for the analysis of liquefaction potential, which have been mainly used in the analysis of mining tailings dams [6]–[12].

This article presents the theory of liquefaction potential evaluation of soil and methods of field evaluation and methods based on the application of dynamic energy penetration tests with the PANDA 2® and PANDA 3® penetrometer.

2. Cyclic Stress Ratio (CSR) and Cyclic Resistance Ratio (CSR)

2.1. CSR

The cyclic stress ratio (CSR) represents the seismic demand generated in a soil deposit. The magnitude of the solicitation depends directly on the characteristics of the earthquake likely to occur at a site and on the geotechnical properties of the soil.

For the calculation of (CSR) a rigid soil column of height h and density γ is considered in which a cutting zone is produced at the base of the column. The shear force is mobilized by the action of an accelerated movement at a_{max} (Figure 1a).

The original work of Seed and Idriss [1], recognized the non-uniform nature of the seismic movement and applied a factor of 0.65 that converts the typical seismic record into a series of uniform equivalent cycles, in the same way and recognizing the nature granular soil mass and energy dissipation associated with granular motion (Figure 1b), a reduction factor r_d (Figure 1c) is included.. Finally the CSR is obtained through eq.1.

$$CSR = \frac{\tau_{av}}{\sigma'_{v0}} = 0{,}65\left(\frac{a_{max}}{g}\right)\left(\frac{\sigma_{v0}}{\sigma'_{v0}}\right) r_d \tag{1}$$

With:
σ_{v0} = Total vertical stress..
σ'_{v0} = Effective vertical stress.
a_{max} = Maximum horizontal acceleration.
r_d = Reduction factor.

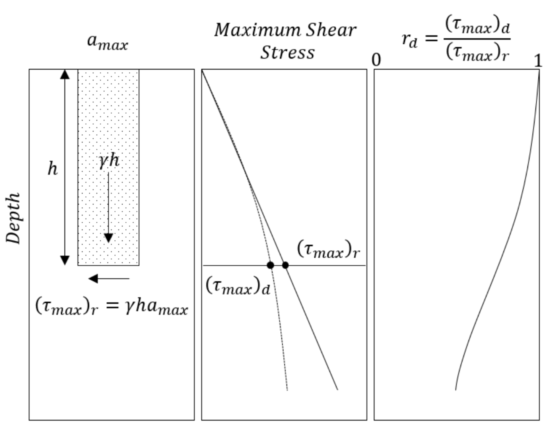

Figure 1. Rigid body scheme (a), maximum shear force (b) and reduction factor r_d (c).

The reduction factor r_d can be calculated through eq. 2. for depths less than 34 meters and eq. 3 for depths greater than 34 meters [13].

$$\ln(r_d) = \alpha(z) + \beta(z)M_w \tag{2}$$

With:

$$\alpha(z) = -1{,}012 - 1{,}126\sin\left(\frac{z}{11{,}73} + 5{,}133\right)$$

$$\beta(z) = 0{,}106 - 1{,}118\sin\left(\frac{z}{11{,}28} + 5{,}142\right)$$

With:
z = Depth (m).
M_w= Seismic magnitude.

$$r_d = 0{,}12e^{0{,}22M_w} \tag{3}$$

Other equations have been proposed in the bibliography ([2], [13]–[15]).

2.2. CRR

The cyclic stress ratio CRR represents the resistance of the soil against the occurrence of liquefaction, so it represents the limit value of soil behavior change.

The CRR is obtained from field or laboratory tests. The most used laboratory tests are cyclic triaxial equipment and cyclic simple shear test equipment. Other laboratory equipment used are: resonant column test, torsional cutting test vibrating table test, centrifugal dynamics [16]. The most commonly used in situ tests are: static and dynamic penetrometers (SPT, CPT, BPT) in addition to the analysis of wave propagation (V_s). In the case of penetrometers, the most used are the SPT and the CPT which has allowed the creation of a robust database of real cases, which is constantly updated with data of new earthquakes.

The principle of the evaluation of CRR from penetration tests consists in obtaining an index or resistance value by applying different correction factors associated with the soil, the possible earthquake and the equipment and its operating conditions.

2.3. Correction factors

2.3.1. Magnitude scaling factor

The development of the simplified method of evaluation of the liquefaction potential was based on the construction of a database with data from sites where earthquakes of magnitudes close to 7.5 occurred [17]. Therefore, it is necessary to extend the method to earthquakes of another magnitude by using a magnitude scale factor (MSF). The application of the MSF allows the consideration of any earthquake and convert it to a magnitude equivalent to 7.5 or vice versa (eq. 4 and eq. 5), being possible to apply it both to the calculation of CRR and CSR.

$$CRR_{7,5;1} = \frac{CRR_{M_W}}{MSF} \tag{4}$$

$$CSR_{7,5;1\,atm} = \frac{CSR_{M;\sigma'_V}}{MSF} \tag{5}$$

The MSF can be obtained through equations. For sandy soils, the use of eq. 6 [15], and for clay soils the eq. 7 [18].

$$MSF = 6{,}9e^{\left(\frac{-M_W}{4}\right)} - 0{,}058 \qquad (\leq 1{,}8) \tag{6}$$

$$MSF = 1{,}12e^{\left(\frac{-M_W}{4}\right)} - 0{,}828 \qquad (\leq 1{,}13) \tag{7}$$

Otras ecuaciones para obtencion del MSF pueden ser obtenidas en la bibliografias ([3], [18]–[20]).

2.3.2. Overburden correction factor

Reviewing the basic assumptions of the simplified procedures of Seed and Idriss [1], it is indicated that the results under 15 meters of depth may not be precise, likewise, based on cyclic triaxial test it has been shown that the CRR increases with increasing pressure confinement [17], which increases as soil depth increases. To incorporate this overload effect, a correction factor for overload k_σ has been recommended for overload pressures greater than 100 kPa (eq. 8) [21].

$$k_\sigma = 1 - C_a \ln\left(\frac{\sigma'_{v0}}{P_a}\right) \qquad \leq 1{,}1 \tag{8}$$

Where the C_a is a coefficient can be obtained based on the penetration resistance of CPT and SPT tests and the relative density of the soil (eq. 9, eq. 10 and eq. 11).

$$C_a = \frac{1}{18{,}9 - 17{,}3 D_R} \qquad \leq 0.3 \tag{9}$$

$$C_a = \frac{1}{18{,}9 - 2{,}55\sqrt{(N_1)_{60}}} \qquad \leq 0.3 \tag{10}$$

$$C_a = \frac{1}{37{,}3 - 8{,}27 (q_{c1N})^{0{,}264}} \qquad \leq 0.3 \tag{11}$$

2.3.3. Sloping ground correction factor k_a

The original formulation of the evaluation method of Seed and Idriss [1], considers a level surface soil. In order to consider the effects generated by the inclication of the soil (static shear stresses), a correction factor for soil slope k_a is included [21]. This factor is based on the use of the parameter α which is the result of the normalization of static shear stress τ_{st} that acts in a plane with respect to the vertical effective stress σ'_{v0} (eq. 12)

$$\alpha = \frac{\tau_{st}}{\sigma'_{v0}} \tag{12}$$

The α value is used to obtain the k_a [21], however, the need for further research has been indicated and the application by geotechnical specialists is recommended.

2.4. Cyclic Resistance Ratio (CRR)

The cyclic stress ratio CRR represents the resistance of the soil against the occurrence of liquefaction, so it represents the limit value of soil behavior change.

The CRR is obtained from field or laboratory tests. The most used laboratory tests are cyclic triaxial equipment and cyclic simple shear test equipment. Other laboratory equipment used are: resonant column test, torsional cutting test vibrating table test, centrifugal dynamics [16]. The most commonly used in situ tests are: static and dynamic penetrometers (SPT, CPT, BPT) in addition to the analysis of wave propagation (V_s). In the case of penetrometers, the most used are the SPT and the CPT which has allowed the creation of a robust database of real cases, which is constantly updated with data of new earthquakes.

The principle of the evaluation of CRR from penetration tests consists in obtaining an index or resistance value by applying different correction factors associated with the soil, the possible earthquake and the equipment and its operating conditions.

2.5. Standard Penetration Test (SPT)

The standard penetration test (SPT) was one of the first equipment used in the application of the simplified procedure for the evaluation of the potential for liquefaction [1], [22]. Although the SPT has disadvantages associated with the repetitiveness and quality control of the results, the database that has been constantly updated has allowed obtaining accurate results in the analysis of the liquefaction potential.

The SPT test consists in obtaining the number of blows required to achieve a penetration of 30 centimeters in a soil layer (N_m). The number of blows obtained must be corrected to reduce the effects of factors associated with the equipment and procedure. Once the corrections have been applied ($(N_1)_{60cs}$), the CRR calculation is with eq 15 [19], [23], [24].

$$CRR_{7,5;1} = e^{\left(\frac{(N_1)_{60cs}}{14.1} + \left(\frac{(N_1)_{60cs}}{126}\right)^2 - \left(\frac{(N_1)_{60cs}}{23{,}6}\right)^3 + \left(\frac{(N_1)_{60cs}}{25{,}4}\right)^4 - 2{,}8\right)} \tag{15}$$

The equation for obtaining the CRR value is based on an earthquake of 7.5 degrees. For other quantities, it must be corrected to an equivalent seismic magnitude value by applying the Magnitude Scaling Factor (MSF). It is also necessary to apply te overburden correction factor (k_σ) and sloping ground correction factor (k_a). The Figure 2 show the $(N_1)_{60cs} - CSR_{7,5;1}$ relation for SPT test.

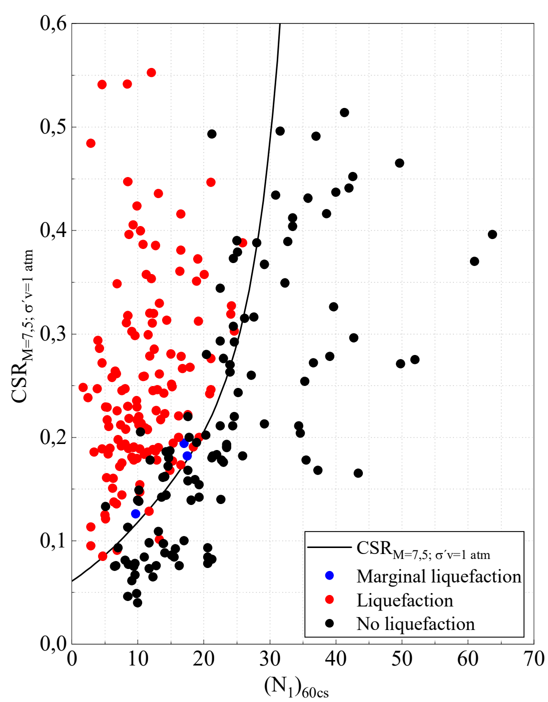

Figure 2. Database of historical CSR cases and relations between $CRR_{7,5;1\ atm}$ and $(N_1)_{60cs}$.

2.6. Cone Penetration Test (CPT)

The CPT is a penetration test, which consists of driving a tip instrumented with sensors. Being a static type, the penetrometer is operated at a constant speed and a continuous recording of the resistance generated in the tip of the equipment is made. It is also possible to measure the friction between the ground and the equipment by means of a friction sleeve in addition to the possibility of measuring the pore pressure by means of a pressure filter located in the device

The method developed for the CPT is based on the generation of a database of tests carried out at sites where liquefaction occurred or did not occur, after the occurrence of a seismic event (Figure 3), in addition CPT- SPT correlations have been developed.

The resistance q_c d must be normalized and corrected by the content of fines and seismic magnitude. Once the corrections to the tip resistance (q_{c1Ncs}), have been applied, it is possible to calculate $CRR_{7{,}5;1\,atm}$ with eq. 17 [19].

$$CRR_{7,5;1} = e^{\left(\frac{q_{c1Ncs}}{113}+\left(\frac{q_{c1Ncs}}{1000}\right)^2-\left(\frac{q_{c1Ncs}}{140}\right)^3+\left(\frac{q_{c1Ncs}}{137}\right)^4-2{,}8\right)} \quad (17)$$

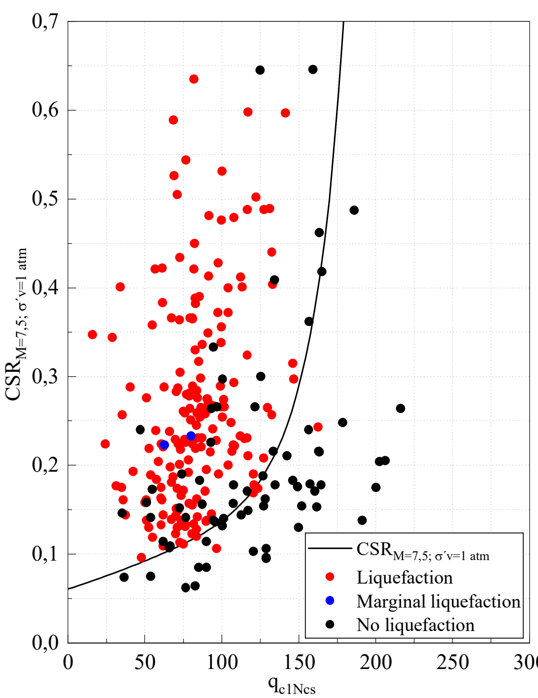

Figure 3. Database of historical CSR cases and relations between $CRR_{7{,}5;1\,atm}$ and q_{c1Ncs}.

3. Dynamic Cone Penetrometer PANDA®

The PANDA Variable Energy Lightweight Dynamic Penetrometer (FP P 94-105) [25] is a widely used penetrometer in France, Europe and some countries in Asia and South America.

In France it has been used in the control of bases of railway lines, roads and other engineering works ([26]–[29]), where it has been tested as a reliable and versatile equipment and methodology, allowing to test soils up to 6 meters from depth.

The operating principle consists of the blow of a head instrumented with sensors, mounted on steel bars and at its end a conical tip of 2, 4 and up to 10 cm^2 is fixed. For each blow the tip resistance is recorded forming a penetrogram (qd-z).

The PANDA penetrometer has been constantly updated and has versions that are based on the calculation of resistance to penetration using the Dutch equation of dynamic driving (PANDA 2®) and the use of the theory of elastic waves in a bar and its interaction with the soil to obtain the relationship resistance to penetration versus the penetration generated [30] (PANDA 3®).

3.1. PANDA 2®

The PANDA 2® is based on the application of the Dutch formula (Eq.18) for the calculation of the tip resistance qd [30], [31]. Each blow given during the drilling for the tip (e), generates an energy that is registered and processed immediately.

$$q_d = \frac{1}{A} \cdot \frac{\frac{1}{2}MV^2}{e} \cdot \frac{1}{1+\frac{P}{M}} \quad (18)$$

With:
e = Tip penetration.
A = Tip section.
M = Hammer mass.
P = Mass of the set driven.
V = Impact speed.

The recorded data is stored in a dialogue terminal (TDD) that allows the visualization and analysis of the results immediately (Figure 4).

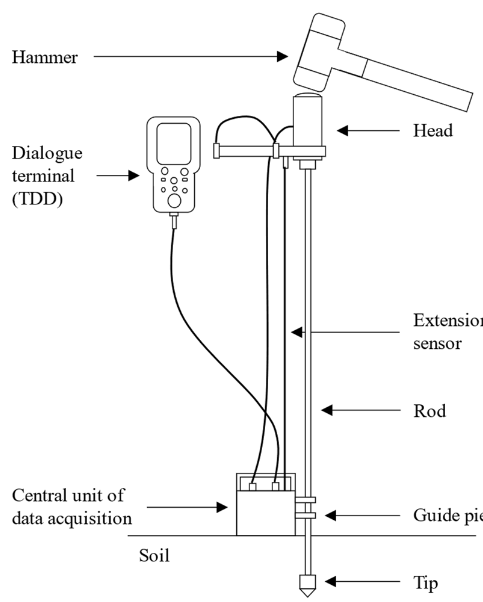

Figure 4. PANDA 2® dynamic variable energy light penetrometer scheme.

3.2. PANDA 3®

The PANDA 3® is a dynamic penetrometer of variable energy, in which the tip resistance analysis is based on the theory of elastic waves in a rod and its interaction with the soil (Eq. 19).

$$\frac{\partial^2 u}{\partial t^2} - c_t^2 \frac{\partial^2 u}{\partial t^2} = \frac{R(x,t)}{E_t A_t} \quad (19)$$

With:
$R(x,t)$ = External system resistance.
c_t = Wave velocity.

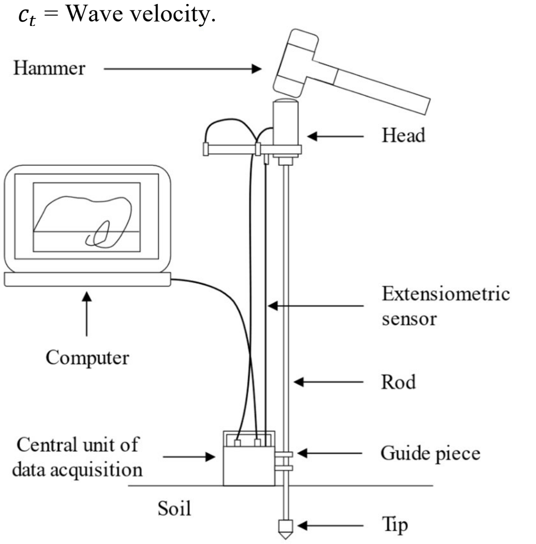

Figure 5. PANDA 3® dynamic variable energy light penetrometer scheme.

The PANDA 3® penetrometer is instrumented with sensors that allow the recording of the variation of the strain $\varepsilon(x,t)$ and the acceleration $a(x,t)$ caused by the passage of the compressional wave generated by the dynamic blow. For the generated wave, the downward (ε_d) and returning (ε_r) wave is decoupled allowing the calculation of the tip penetration ($s_p(t)$) and the stress ($F_p(t)$) during the penetration work, then, it is possible to obtain the load-penetration curve ($\sigma_p - s_p$) [30], [32] (Figure 6).

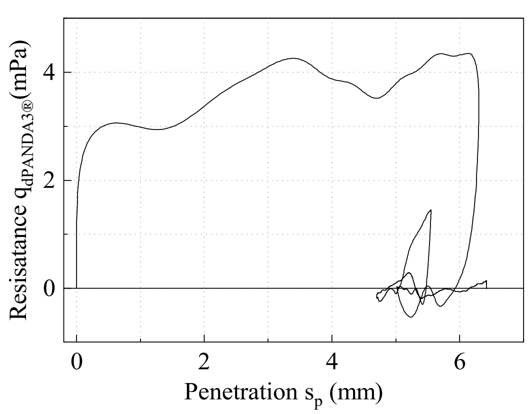

Figure 6. Load - penetration curve in tip PANDA 3®.

By analyzing the load-penetration curve and the soil-tip interaction, it is possible to obtain different geotechnical parameters, such as: Dynamic stiffness (k_d^{P3}), resistance to dynamic penetration (q_d^{P3}) and static (q_c^{P3}) deformation module (E_d^{P3}), Smith's linear damping coefficient (j_S) and ground wave velocities (C_p^{P3} and C_s^{P3}) [26], [27], [30], [32].

4. PANDA® penetrometer liquefaction potential assessment methods

4.1. Lepetit (2002)

Lepetit (2002) [11], proposed a method of evaluation of the liquefaction potential based on the use of PANDA 2® equipment and the permeameter. For the development of the methodology, the correlation between the dynamic tip resistance (q_d) and the tip resistance q_c of the CPT was used (eq. 20) ([33], [34]).

$$q_c \cong (0{,}93 \ to \ 1{,}05) q_d \quad (20)$$

Lepetit [11] considered $q_d = q_c$ and the permeability measurements to define the soil behavior index (I_c) and operate according to Robertson's original method [35], [36]. To obtain the relationship k-I_c, s the values previously proposed for hydraulic conductivity [37] and the classification of soils proposed in the works developed by Robertson [38], [39] (Table 1) are used. With the relationship values it is possible to obtain the relationship between both parameters (Figure 7).

Table 1. Relationship between permeability and soil behaviour [37], [40].

Soil type	I_c	$k/(m/s)$
Gravel	<1,31	$10^{-3} - 10^{0}$
sand	1,31< I_c<2,05	$10^{-5} - 10^{-3}$
Sandy mixtures	2,05< I_c<2,60	$10^{-7} - 10^{-5}$
Silty mixtures	2,60<I_c<2,95	$10^{-9} - 10^{-7}$
Clays	2,95<I_c<3,60	$10^{-10} - 10^{-9}$

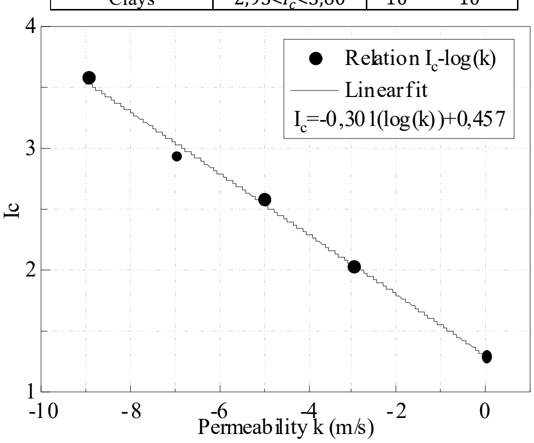

Figure 7. Relationship $I_c - k$ [37], [40].

By obtaining the relationship k-I_c it is possible to replace the soil behavior index with permeability, and present the equations for the calculation of the correction factor k_c as a function of permeability (k) (eq. 21 for $k > 1{,}4 \cdot 10^{-4}(m/s)$ and eq.22 for $k < 1{,}4 \cdot 10^{-4}(m/s)$).

$$\mathrm{k}_c = 1 \quad (21)$$

$$\mathrm{k}_c = 0{,}26 \cdot (\log(k))^2 + 2{,}6 \cdot (\log(k)) + 5{,}49 \quad (22)$$

The scheme of the method is shown in Figure 8.

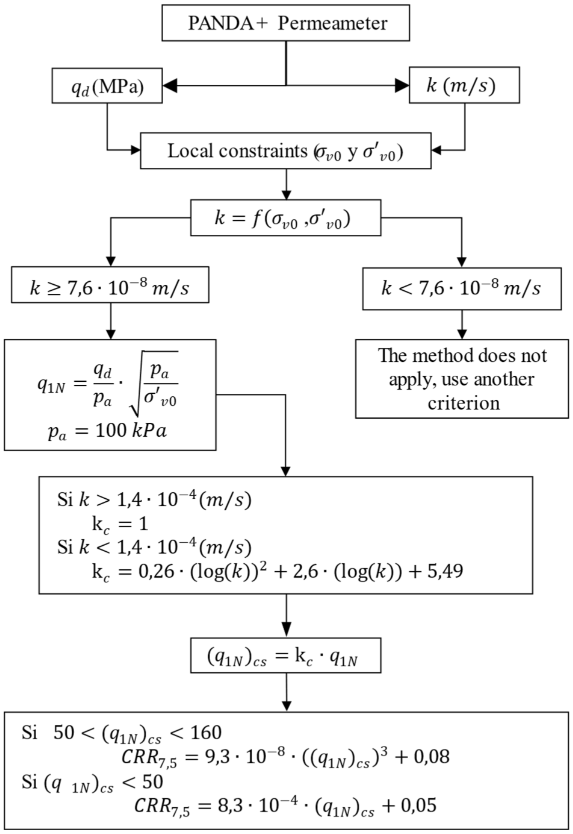

Figure 8. Scheme of the method proposed by Lepetit (2004) [11].

4.2. Jara 2013

Jara (2013)[12] proposed two methodologies for the evaluation of the liquefaction potential through the use of the PANDA 3® lightweight dynamic penetrometer and the Geondoscope® equipment [41]. The main idea of the methods is to obtain the dynamic resistance and deformation parameters by means of the PANDA 3® test and the soil characteristics by means of analysis of geoendoscopic images with the Geondoscope®.

The methods are based on the consideration of equality between q_c and q_d^{P3}, allowing to apply the liquefaction potential analysis method proposed for the CPT [35] and on the equality between shear waves velocities Cs_{P3} y V_s, allowing apply the method based on shear waves [42].

From the Geoendoscope® test it is possible to obtain the percentage of fines and subsequently the index of soil behavior I_c.

4.2.1. Method 1

The first proposed method is based on the application of Robertson's procedures [35] to mining tailings, for which the value of the standardization exponent n is considered equal to 0.5 [12]. The normalization of the tip resistance is carried out by means of eq. 23.

$$q_{d1N} = \frac{q_d}{P_a}\left(\frac{P_a}{\sigma'_{v0}}\right)^{055} \tag{23}$$

The original Robertson method [35] allows obtaining a fine content value through the behavior index. For this method the value of the soil behavior index it is caculated through the fine content that is obtained by means of Geoendoscopy. The equation for soil beahaviour index through the fine content is eq. 24 (for $FC \in [0{,}01\%; 99\%]$).

$$I_c = \left(\frac{FC+3{,}7}{1{,}75}\right)^{\frac{1}{3{,}25}} \tag{24}$$

With the value of I_c it is possible to calculate the correction factor by content of fines. The CRR corrected by magnitude and fine content is calculated using eq. 25 [43].

$$CRR_{7{,}5;1\,atm} = e^{\left(\frac{q_{c1Ncs}}{540}+\left(\frac{q_{c1Ncs}}{67}\right)^2-\left(\frac{q_{c1Ncs}}{80}\right)^3+\left(\frac{q_{c1Ncs}}{114}\right)^4-3\right)} \tag{25}$$

Below is an scheme of the evaluation method of the liquefaction potential proposed by Jara [12] (Figure 9).

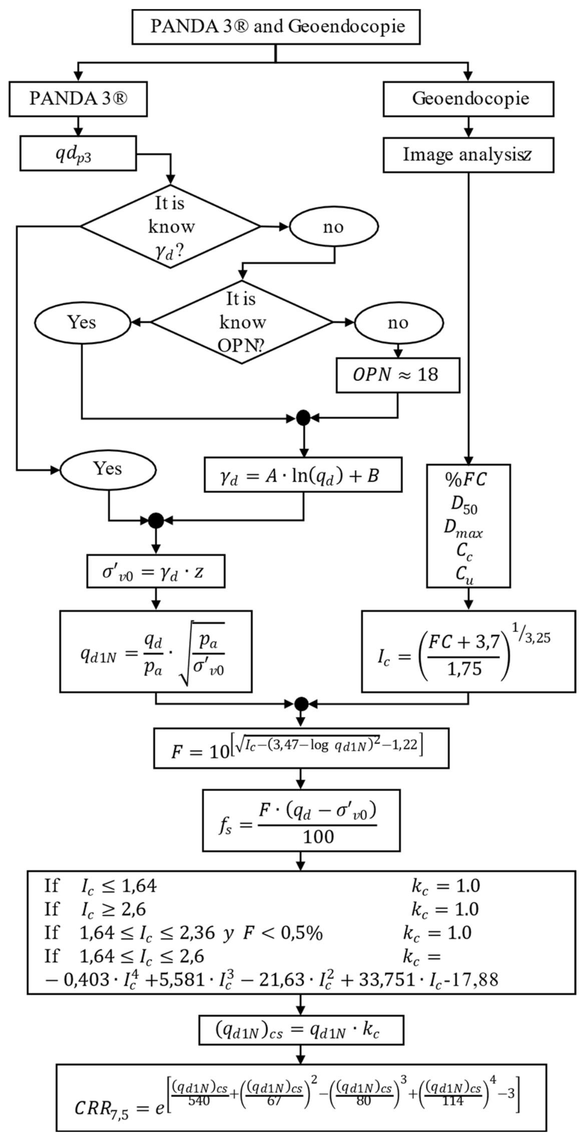

Figure 9. Method 1 scheme proposed by Jara (2013) [12].

4.2.2. Method 2

The second proposed method is based on the consideration of equivalence between the shear waves velocities obtained by the PANDA 3® penetrometer (Cs_{P3})) and the shear waves velocity (V_s). If $Cs_{P3} = V_s$, is considered, and the method of Andrus and Stokoe (2000) [42], the equation for the normalized and corrected shear wave velocity is eq. 26.

$$(Cs_{P3})_1 = Cs_{P3}\left(\frac{P_a}{\sigma'_{v0}}\right)^{0,25} \tag{26}$$

The percentage of fines content is obtained through a geoendoscopy test and based on this the correction factor k_{cs} according to Juang et al. (2002) [44] (eq. 27 para $FC \leq 5\%$, eq. 28 para $5\% < FC < 35\%$ y eq. 29 para $FC \geq 35\%$)

$$k_{cs} = 1 \tag{27}$$
$$k_{cs} = 1 + T(FC - 5) \tag{28}$$
$$k_{cs} = 1 + 30T \tag{29}$$

With:

$$T = 0{,}009 - 0{,}0109\left(\frac{(Cs_{P3})_1}{100}\right) + 0{,}0038\left(\frac{(Cs_{P3})_1}{100}\right)^2$$

Finally, the value of $CRR_{7,5cs}$ can be obtained by eq. 30:

$$CRR_{7,5cs} = 0.022\left(\frac{(Cs_{P3})_{1CS}}{100}\right)^2 + 2{,}8\left(\frac{1}{V_{s1c}-(Cs_{P3})_{1CS}} - \frac{1}{V_{s1c}}\right) \tag{30}$$

Where V_{s1c} is the upper limit value for the occurrence of liquefaction as indicated in method [42], and which is calculated according to Eq. 31.

$$V_{s1c} = 200 + \frac{35-FC}{30}\cdot 15 \tag{31}$$

The method based on shear waves is schematized in Figure 10.

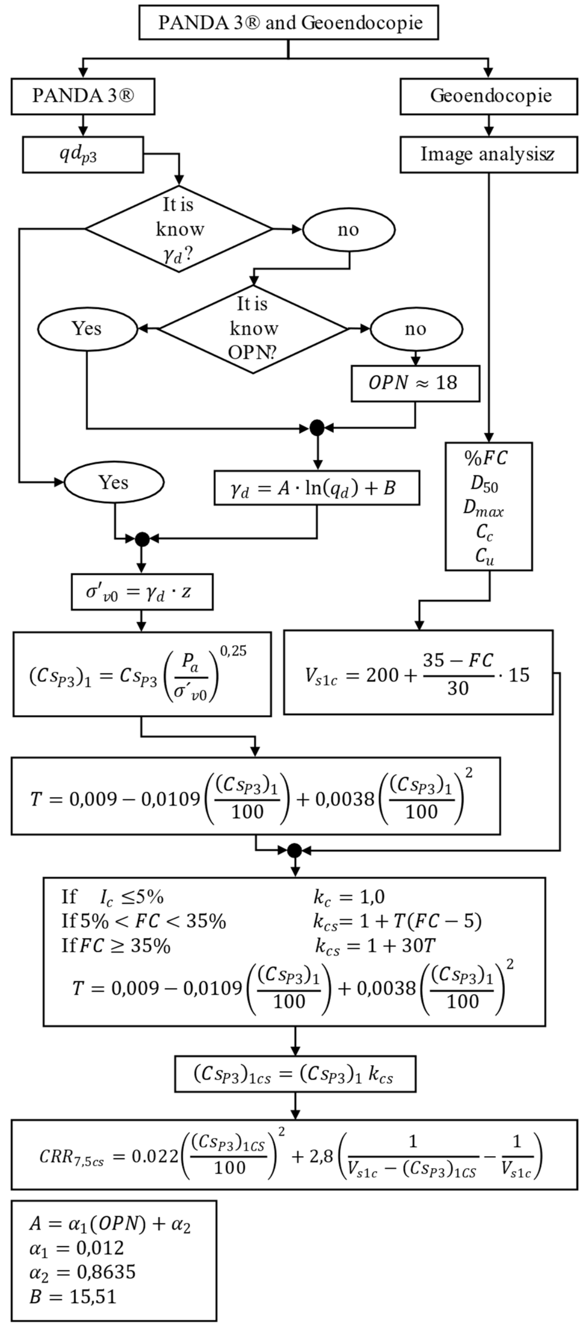

Figure 10. Method 2 scheme proposed by Jara (2013) [12].

4.3. Villavicencio 2009

Villaviciencio (2009) [45], proposed a method for obtaining the cyclic resistante ratio normalized and corrected ($CRR_{7,5;1}$) through the use of the PANDA 2® penetrometer [45]. The methodology considered the equality between q_d and q_c and from this apply the equations of the simplified method developed for the CPT penetrometer, similar to the method proposed by Lepetit (2002) [11]. This method considered the application of the equation proposed by Boulanger (2003) (eq. 25) [46].

5. Comentarios y conclusiones

Penetrometers are one of the equipment currently available to predict soil liquefaction. However, there are still some difficulties in the field analysis that are associated with the execution of the currently available test. Due to the above that the generation of new methods based on new equipment is necessary.

The methods presented in this article are based on correlations with widely used equipment in the analysis of the liquefaction potential, and in fine granular soils (mining tailings), however, the results based on the PANDA® equipment can be used for preliminary analysis. of liquefaction potential in similar soils.

The PANDA® penetration equipment allows the evaluation of the soil liquefaction potential, however, it is necessary to generate a proprietary database that allows predicting the cyclic resistance of the soil based on the dynamic tip resistance qd.

References

[1] B. Seed and I. Idriss, "Simplified Procedure for Evaluating Soil Liquefaction Potential," *J. Soil Mech. Found. Div.*, vol. 97, no. 9, pp. 1249–1273, 1971.

[2] L. Youd *et al.*, "Summary Report," in *NCEER Workshop on Evaluation of Liquefaction Resistance of Soils*, 1997, vol. Rept. NCEE, pp. 1–40.

[3] T. L. Youd and I. M. Idriss, "Proceedings of the NCEER Workshop on Evaluation of liquefaction resistance of soils," in *NCEER Workshop on Evaluation of Liquefaction Resistance of Soils*, 1997.

[4] M. Hashemi and M. R. Nikudel, "Application of Dynamic Cone Penetrometer test for assessment of liquefaction potential," *Eng. Geol.*, 2016.

[5] AFNOR, *XP P 94-105 Contrôle de la qualité du compactage*. Francia, 2000, p. 36.

[6] R. Espinace A *et al.*, "pénétromètre Panda ®Stability of chilean's tailings dams with the Panda® penetrometer. Experiences of the last 10th," in *18th International Conference on Soil Mechanics and Geotechnical Engineering*, 2013, pp. 519–522.

[7] C. Sanhueza and G. Villavicencio, "Estimación de parámetros resistentes a partir del ensayo de penetración PANDA y su aplicación en el cálculo de la capacidad de soporte y asentamientos del suelo de fundación (1a parte)," *Rev. la Constr.*, vol. 9, no. 1, pp. 120–131, 2010.

[8] G. Villavicencio, P. Breul, R. Espinace, and P. Valenzuela, "Control de compactación con penetrómetro ligero en tranques de relaves, considerando su variabilidad material y estructural," *Rev. la Constr.*, vol. 11, no. 1, pp. 119–133, 2012.

[9] G. Villavicencio A, P. Breul, C. Bacconnet, A. Fourie, and R. Espinace A, "Liquefaction potential of sand tailings dams evaluated using a probabilistic interpretation of estimated in-situ relative density," *Rev. La Constr.*, vol. 15, no. 2, pp. 9–18, 2016.

[10] C. Sanhueza and G. Villavicencio, "Influencia de la cohesión aparente generada por raíces sobre la estabilidad de un talud natural en las dunas de Reñaca," *Rev. la Constr.*, vol. 11, no. 1, pp. 16–31, 2012.

[11] L. Lepetit, "Etude d'une méthode de diagnostic de digues avec prise en compte du risque de liquéfaction," Université Blaise Pascal, 2002.

[12] F. Jara, "Etude d'une nouvelle méthodologie pour la détermination du potentiel de liquéfaction des sols à l'aide du Panda 3® et de la géoendoscopie: Cas des «tranques de Relave» au Chili," Polytech'Clermont Ferrand, 2013.

[13] I. M. Idriss and R. W. Boulanger, "Semi-empirical procedures for evaluating liquefaction potential during earthquakes," *Soil Dyn. Earthq. Eng.*, vol. 26, no. 2-4 SPEC. ISS., pp. 115–130, 2006.

[14] S. S. C. Liao and R. V Whitman, "A Catalog of Liquefaction and Non-liquefaction Occurrences During Earthquakes," Department of Civil Engineering, MIT, 1986.

[15] I. Idriss, "An Update to the Seed-Idriss Simplified Procedure for Evaluating Liquefaction Potential," in *Proceedings of TRB Workshop on New Approaches to Liquefaction*, 1999.

[16] Y. Huang and Y. Miao, *Hazard Analysis of Seismic Soil Liquefaction*, 1st ed. Springer Singapore, 2017.

[17] T. L. Youd and I. M. Idriss, "Liquefaction Resistance of Soils: Summary Report from the 1996 NCEER and 1998 NCEER/NSF Workshops on Evaluation of Liquefaction Resistance of Soils," *J. Geotech. Geoenvironmental Eng.*, vol. 127, no. 4, pp. 297–313, 2001.

[18] R. W. Boulanger and I. M. Idriss, "Evaluation of cyclic softening in silts and clays," *J. Geotech. Geoenvironmental Eng.*, vol. 133, no. 6, pp. 641–652, 2007.

[19] R. W. Boulanger and I. M. Idriss, "CPT and SPT based liquefaction triggering," California, 2014.

[20] I. M. Idriss and R. W. Boulanger, *Soil Liquefaction during Earthquakes*, vol. 1. Oakland, California, USA, 2008.

[21] H. Bolton Seed, "Earthquake-Resistant Design of Earth Dams," in *Seismic Design of Embankments and Caverns*, 1983, pp. 41–64.

[22] H. B. Seed and I. M. Idriss, "A simplified procedure for evaluating soil liquefaction potential. Report No. EERC 70-9," Berkeley, California, U.S.A., 1970.

[23] R. W. Boulanger, D. W. Wilson, and I. M. Idriss, "Examination and Reevalaution of SPT-Based Liquefaction Triggering Case Histories," *J. ofGeotechnical Geoenvironmental Eng.*, vol. 138, no. 8, pp. 898–909, 2012.

[24] A. Sy and R. G. Campanella, "Becker and standard penetration tests (BPT-SPT) correlations with cbnsideration of casing friction," *Can. Geotech. J.*, vol. 31, no. 3, 1994.

[25] AFNOR, *XP P 94-105 Suelos : Reconocimiento y pruebas Control de calidad de compactación Norma experimental Correspondencia Ejecución de excavaciones BNSR-CTT*. Francia, 2000, pp. 1–33.

[26] E. Escobar, M. B. Navarrete, R. Gourvès, P. Breul, and B. Chevalier, "Dynamic Characterization of the Supporting Layers in Railway Tracks using the Dynamic Penetrometer Panda 3 ®," *Int. Conf. Transp. Geotech. (ICTG 2016)*, vol. 143, no. Ictg, pp. 1024–1033, 2016.

[27] E. Escobar *et al.*, "Reconnaissance dynamique des sites ferroviaires a l'aide du penetrometre PANDA 3®," in *Journées nationales de géotechnique et de géologie de l'ingénieur*, 2014.

[28] Y. Haddani, P. Breul, G. Saussine, M. A. Benz, F. Ranvier, and R. Gourvès, "Trackbed Mechanical and Physical Characterization using PANDA ® / Geoendoscopy Coupling," in *International Conference on Transportation Geotechnics (ICTG 2016)*, 2016, vol. 143, no. Ictg, pp. 1201–1209.

[29] L. Chaigneau, R. Gourves, and D. Boissier, "Compaction control with a dynamic cone penetrometer," in *International Workshop on Compaction of Soils, Granulates and Powders*, 2000, pp. 103–109.

[30] M. Benz, "MESURES DYNAMIQUES LORS DU BATTAGE DU PENETROMETRE PANDA 2," 2009.

[31] B. Frazer, "Formulas de hinca dinámica," *Rev. Obras Pública*, pp. 977–990, 1971.

[32] E. J. Escobar Valencia, "Mise au point et exploitation d'une nouvelle technique pour la reconnaisance des sols: le PANDA 3," Université Blaise Pascal, 2015.

[33] L. Chaigneau, "Caracterisation des mileux granulaires de surface al'aide d'un penetrometre," Université Blaise Pascal - Clermont II, 2001.

[34] D. Langton, "The Panda lightweight penetrometer for soil investigation and monitoring material compaction," *Gr. Eng.*, no. September, 1999.

[35] P. K. Robertson and C. E. Wride, "Evaluating cyclic liquefaction potential using the cone penetration test," *Can. Geotech. J.*, vol. 35, no. 3, pp. 442–459, 1998.

[36] P. Robertson and R. Campanella, "Liquefaction potentia of sands using the CPT," *J. Geotech. Eng.*, vol. 111, no. 3, pp. 384–403, 1985.

[37] M. Manassero, "Hydraulic conductivity assessment of slurry wall using piezocone test," *J. Geotech. Eng.*, vol. 120, no. 10, pp. 1725–1746, 1994.

[38] P. K. Robertson, R. G. Campanella, D. Gillespie, and J. Greig, "Use of Piezometer Cone Data," in *IN-SITU '86 Use of in-situ testing in geotechnical engineering, ASCE specialty conference*, 1986, pp. 1–18.

[39] P. K. Robertson, "Soil classification using the cone penetration testing," *Can. Geotech. J.*, vol. 27, no. 1, pp. 151–158, 1990.

[40] T. Lunne, P. K. Robertson, and J. J. M. Powell, *Cone penetration testing in geotechnical practice*. 1997.

[41] Y. Haddani, "Caractérisation et classification des milieux granulaires par géoendoscopie," Clermont Ferrand 2, Clermont Ferrand, 2004.

[42] R. Andrus and K. Stokoe, "Liquefaction Resistance of Soils from Shear-Wave Velocity," *J. Geotech. Geoenvironmental Eng.*, vol. 126, no. 11, pp. 1015–1025, 2000.

[43] I. M. Idriss and R. W. Boulanger, "Semi-empirical Procedures for Evaluating Liquefaction Potential During Earthquakes," in *3rd International Conference on Earthquake Geotechnical Engineering*, 2004, pp. 32–56.

[44] C. H. Juang, T. Jiang, and R. D. Andrus, "Assessing

probability-based methods for liquefaction potential evaluation," *J. Geotech. Geoenvironmental Eng.*, vol. 128, no. 7, pp. 580–589, 2002.

[45] G. Villavicencio A, "Methodologie pour evaluer la stabilite des barrages de residus miniers," Université Blaise Pascal Clermont II, 2009.

[46] R. W. Boulanger, "High overburden stress effects in liquefaction analyses," *J. Geotech. Geoenvironmental Eng.*, vol. 129, no. 12, pp. 1071–1082, 2003.