Instability assessment of Loess using a CPT-based evaluation method

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ABSTRACT: In order to understand and evaluate the instability mechanism of French loess submitted to actions of high speed trains, a large campaign of CPT tests was conducted in both initially unsaturated and saturated boreholes. Results showed that the soil manifested a decrease in its penetration resistance once the soil is fully saturated. On the other hand, a numerical method based on shear wave velocity measurements was used in order to evaluate and predict such instability. The method consists of determining the initial shear wave velocity in soil using a CPT-V_s proposed correlation and some basic geotechnical parameters, such as fines, calcareous and water contents. The instability risk was evaluated by calculating a factor of safety representing the ratio of the soil's cyclic shear resistance after its full saturation to the soil's induced cyclic shear stress. A comparison made between the observed in situ behavior and the predicted results shows that the loess cyclic shear resistance is highly dependent on its initial geotechnical characteristics and its water content: a significant decrease in the measured penetration resistance and shear wave velocity were recorded with an increase in water content. The CPT-V_s based method has been found to be an efficient tool in identifying layers with high instability risk.

1 INTRODUCTION

Aeolian soils, like loess deposits, are geologically unstable by nature, and can be eroded very readily. Some recent laboratory tests conducted on loess taken from northern France confirmed the results in the literature regarding its nature: it is a homogeneous, typically non-stratified, porous, friable, slightly coherent, often calcareous, fine-grained, silty and pale yellow sediment (Marcial et al, 2002; (Author Marcial is not in ref list? Karam et al, 2006).

Between 1999 and 2004, numerous sinkholes were observed on the ground surface after long rainy periods along the Northern French TGV (high speed rail line). These holes can reach about 5 m diameter and 1 m depth. For the holes situated far from the railway line, their origin was identified and attributed to cavities that had existed since the First World War and other existing quarries. This represents 60% of the identified cases; the remaining 40% involved the zone of less than 25 m from the railway, for which the collapse couldn't be explained by cavity presence. It is worth mentioning that the high speed train railway is subjected to an equivalent 3,500 cycles per day and around 1270000 cycles per year. In 2003, numerical simulations by finite element modelling were performed by Geodynamics Company in order to evaluate the cyclic loads the soil is subjected to. It was reported that until 5 m away from the railway, the vertical cyclic shear stress varies from 11 to 23 kPa up to 4 m depth at a frequency of 20 Hz. The load amplitude considerably decreases beyond 5 m.

In this paper, the studied loess was taken from a particular site referenced Km 140 located at 140 km North of Paris and tested for instability assessment.

2 MATERIAL

The soil's sampling was carried out according to the French Standard (AFNOR 1994) in a trench of 1.5 x 9 m that is 25 m from the railways. Four different depths (1.2, 2.2, 3.5 and 4.9 m) were considered. The geotechnical properties of the samples are presented in Table 1. The liquid limit (w_L) varies from 26 to 30%; the plasticity index (I_p) varies from 6 to 9 %; the dry density is rather low (ρ_d = 1.39 to 1.55 Mg/m^3) that correspond to high porosity (n = 0.43 – 0.49) or high void ratio (e = 0.76 – 0.93); the carbonate content is quite high (% $CaCO_3$ = 5-15%). The grain size distribution analysis showed that the clay fraction (< 2 µm) ranges from 16 to 20%.

Table 1: Geotechnical properties of loess at Km140

Depth (m)	%< 2µm	w_L (%)	I_p (%)	ρ_d (Mg/m^3)	w_{nat} (%)	Ca (%)	S_r (%)
1.2	20	30	9	1.52	18.9	5	66
2.2	16	28	9	1.39	18.1	6	53
3.5	16	26	6	1.54	16.6	15	55
4.9	18	30	9	1.55	23.7	9	82

3 OVERVIEW OF THE INSTABILITY ASSESSMENT METHOD

The details of this method can be found in Karam (2006). The main input data required to proceed with the assessment method are the fines and carbonate percents, initial water content, dry density (or void ratio) and CPT measurements.

<u>Step 1:</u> Under initial in-situ conditions, CPT values are transformed into shear wave velocity profile using appropriate CPT-V_s correlation (see paragraph 4).

<u>Step 2:</u> Under small deformations, the passage from unsaturated shear wave velocity profile to fully saturated conditions can be expressed using the following proposed equation:

$$V_s = \frac{f_1 \cdot R \cdot (V_s)_{rec}}{f(e)} + 41 \cdot \ln\left(\frac{w_s}{w - w_r} - 1\right) \tag{1}$$

where $f_1 / f(e)$ is an adjustment factor that is a function of water content change, applied vertical stress and initial void ratio. It was assumed that the void ratio doesn't change significantly with water content variations.

w_s, w_r and w are the saturated, the residual and the measured water content respectively. $(V_s)_{rec}$ is the shear wave velocity measured in reconstituted soil (carbonate bonding were removed by crushing then sieving the soil sample).

This equation shows that V_s can be decomposed into two terms. The first one reflects changes in the measured shear wave velocity due to macrostructure changes due to loading, volume reduction or reduction of carbonates' effects. The second term reflects the effect of water content.

The term R reflects the hydro-mechanical behavior of loess due to the presence of carbonates and clay bonding. It has been shown (Karam et al., 2009) that the clay bonding is weak but has a direct effect on the measured shear wave velocity: indeed V_s increases with an increase in the inter-particles contact surface. However, when the applied pressure increases beyond the pre-consolidation pressure or when the water content increases significantly, the bonding are eliminated and the pore volume is reduced. R can be expressed by the following equations:

$$\mathrm{R} = \begin{cases} -a' \cdot P + b' & , P \le 75kPa \\ \max(k \cdot P^{-m}, 1) & , P > 75kPa \end{cases} \tag{2}$$

where a', b' are function of the equivalent fines content $(FC)_e$ and P is the applied vertical pressure. The pre-consolidation pressure of the studied loess is around 75 kPa. $(FC)_e$ represents an equivalency in bonding created by both carbonates and clay particles. It can be expressed as follows:

$$\%(\mathrm{FC})_e = \begin{cases} \max\left(\%C + \dfrac{\%Fines}{\left(\dfrac{P_c}{P_{pre}}\right)} \cdot (1 - S_r), 3.5\right) & , P \le 75kPa \\ \%C & , P > 75kPa \end{cases} \tag{3}$$

where $\%C$ is the percent of carbonates content, S_r is the degree of saturation and P_c is the applied pressure.

<u>Step 3:</u> The initial shear modulus can be calculated using the following equation:

$$G_{0\max} = \rho V_s^2 \tag{4}$$

Step 4 : After a large number of undrained cyclic triaxial tests conducted on saturated samples, a decrease in effective confining stress was observed due to an increase in the pore pressure (Karam et al. 2009). Large deformations were also registered. Using the liquefaction resistance curves, a correlation between the shear modulus and the equivalent number of cycles N_{equ} was proposed. N_{equ} is defined as the number of cycles needed to reduce the shear strength of the soil by 90% (usually this phenomenon is observed at small deformations). The proposed correlation can be expressed under the following equation:

$$N_{equ} = \left(\frac{\lambda}{G_{0\max}} \right)^{-\xi} \tag{5}$$

where λ and ξ are function of the cyclic loading q_{cyc} and the percent of cementation of the soil. A schematic representation of the shear modulus degradation during cyclic loading is shown in Figure 1.

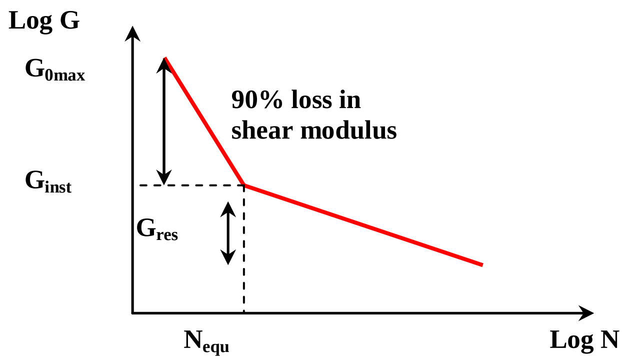

Figure1: Schematic representation of cyclic degradation of shear modulus in triaxial test

Step 5: The cyclic resistance ratio can then be calculated using the following equation

$$R_{CR(trx)} = \varsigma \times N_{equ}^{-\theta} \tag{6}$$

where ζ and θ are constants depending on the equivalent fine content of the soil.

The triaxial resistance ratio can be transformed into in-situ cyclic resistance ratio by using the expression established by Castro et al. (1975) and which was used by other authors, such as, Finn et al. (1971) and Seed et al. (1975):

$$R_{CR(in-situ)} = 0.9 \cdot \left(\frac{1+2k_0}{3\sqrt{3}} \right) \cdot 2 \cdot R_{CR(trx)} \tag{7}$$

Step 6: The cyclic stress ratio can easily be calculated using the common equation proposed by Seed and Idriss (1971):

$$R_{CS} = 0.65 \times \left(\frac{a_{\max}}{g} \right) \times \left(\frac{\sigma_{vo}}{\sigma'_{vo}} \right) \times r_d \tag{8}$$

Step 7: Finally, a safety factor (F_S) profile against cyclic instability can be calculated using:

$$F_s = \frac{R_{CR(in-situ)}}{R_{CS}} \tag{9}$$

4 EFFECT OF SATURATION ON PENETRATION RESISTANCE OF LOESS

In order to evaluate the proposed testing method and to better understand the effect of water content on the mechanical behaviour of loess, two types of CPT measures were conducted at a site referenced Km 140. The first test, CPT101, was conducted at initial conditions with a water content comprised between 18 and 20%. The second test, CPT1, was conducted after wetting of the borehole area. The in-depth water content and the cone penetration resistance profiles are shown in Figure 2. As results show, the water content increased after wetting to around 26% at 2m depth and to 23% at 4m depth. It is worth mentioning that the saturation water content of tested loess at 2.2m is 34%. To water content change corresponds a significant decrease in the penetration resistance: $q_c \approx 0.1$ MPa between 2.2 m and 3.1 m depth after wetting. Beyond 5m, the water content does not change significantly, neither did the cone penetration resistance's profile.

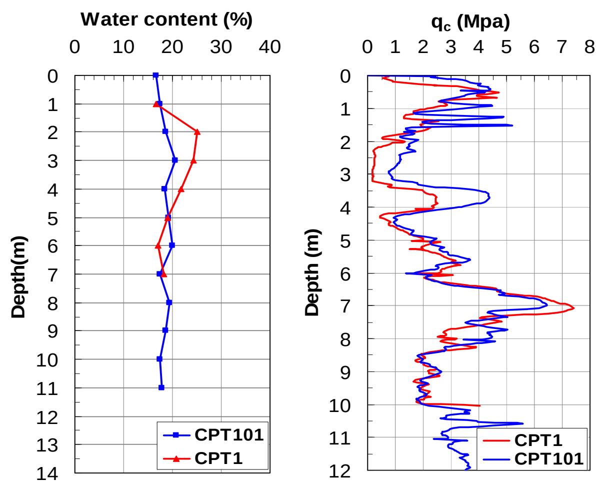

Figure 2: Water content and CPT profiles at initial conditions (CPT101) and after wetting (CPT1) (site Km 140, SNCF)

5 PROPOSED CORRELATION BETWEEN V_S AND CPT PARAMETERS

Many correlations between cone penetration parameters and shear wave velocity were proposed in literature. Below are two equations proposed by Andrus et al. (2003) to determine the shear wave velocity using CPT values measured in uncemented sand and clay:

$$V_s = 26.3(q)^{0.199}(f_s)^{0.003}.ASF \qquad \text{for sands} \qquad (10)$$

$$V_s = 14.3(q)^{0.428}(f_s)^{0.108}.ASF \qquad \text{for clays} \qquad (11)$$

where ASF is the aging scaling factor.

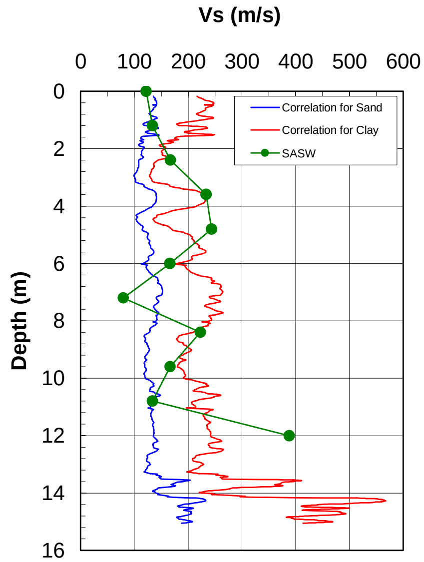

Figure 3: Comparison between measured V_s by SASW and by using CPT-V_s correlating equations proposed by Andrus et al, 2003.

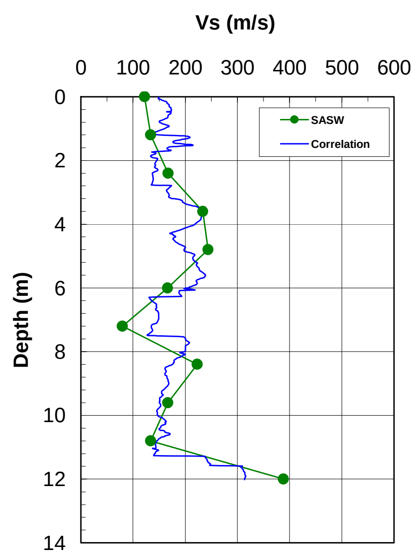

Figure 4: Calculated V_s profile in loess soil using a new CPT-V_s correlation equation.

In order to evaluate these equations, Spectral Surface Wave Analysis (SASW) was performed in order to determine the shear wave velocity in initial site conditions. A STRATAVIZOR central acquisition of 48 channels was installed and connected to 48 geophones of 10 Hz. A 10 kg hammer was used as a dynamic source. The distance between the source and the first geophone was 10 m. The shear wave velocity was measured every 1.2m depth.

A comparison between the measured shear wave velocity and the calculated ones (based on equations 10 and 11) is shown in Figure 3. The results show that for the first 2m, V_s follows the sand-correlation profile and then changes to the clay-correlation profile between 2 and 4 m depth. For larger depth, the results are erratic and no conclusion can be made. This may be explained by the cemented nature of the loess and also to the uncertainties in the measured values of V_s by the SASW method beyond 6 m depth.

This is why a new correlation was proposed in this paper in order to include the particular cemented nature of loess. The adjusted equation can be expressed as follows:

$$V_s = V_{s1}\frac{R}{f(e)}(aZ+b) \tag{12}$$

where V_{s1} is the equation (10) proposed by Andrus et al (2003) for uncemented sand, Z is the depth in meters. The new calculated V_s profile is shown in Figure 4. Note that for the soil layers beyond 5 m depth, the initial geotechnical characteristics have not been determined and a back analysis was conducted in order to match the calculated and the measured shear wave velocity profiles.

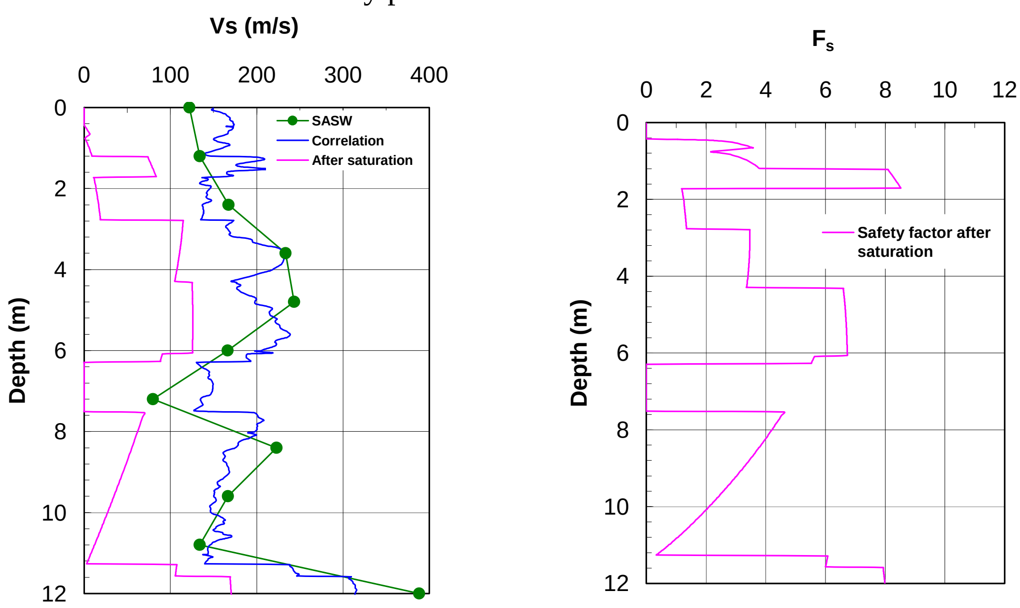

Figure 5: V_s profile in wetted conditions and safety factor against instability calculated using the proposed assessment method

6 APPLICATION OF THE INSTABILITY ASSESSMENT METHOD

The assessment method was applied directly. First the V_s profile was determined under saturated conditions, then a cyclic loading of 23 kPa was applied and a safety factor profile was calculated (Figure 5). The results show that (except the first 0.50 m depth) the soil layers located between 1.8 m – 2.4 m, 6.2 m – 7.8 m and 10.4 m – 11.6 m have the lowest safety factors ($0 \leq F_s \leq 1$). The cyclic triaxial tests that were conducted on soil specimen taken at 1.2 m, 2.2 m, 3.5 m and 4.9 m respectively showed that the soil at 2.2 m has the lowest cyclic shear resistance (Karam et al. 2009).

7 CONLUSION

A CPT-V_s based instability assessment method was applied on French loess in order to evaluate its cyclic instability along the Northern TGV Line. When effect of particle bonding created by clay and carbonate were taken into consideration, results have shown good agreement between the evaluating method and the laboratory cyclic triaxial tests: the soil situated at 2.2m has the lowest cyclic shear resistance and the soil located at 4.9m has the largest cyclic shear resistance. The instability detected in loess beyond 6m should be considered with precaution since no laboratory data was available and a back analysis was conducted using the SASW profile in order to determine the initial soil parameters.

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