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ABSTRACT

A popular method to evaluate earthquake induced settlements in dry sands is the approach proposed by Pradel (1998) which was based on standard penetration test (SPT) results and is only applicable to clean sands. A simple modification of the Pradel (1998) method is proposed based on cone penetration test (CPT) results and is extended to cover a wide range of unsaturated soils. A key parameter in the method by Pradel (1998) is the small strain shear modulus, G_o, which can be estimated from the CPT or measured using the seismic CPT. The CPT can provide a continuous evaluation of seismic compression that allows the expeditious analysis of complicated soil profiles and a framework for sensitivity analyses. Soil parameters, such as soil type, fines content, and equivalent SPT blow count interpolated from CPTs, were compared with adjacent borings and related laboratory test results from a ground improvement site. Both vibro-stone columns and compaction grouting were adopted to mitigate the site seismic settlement. The proposed simple modification of the Pradel method provided a valuable tool to evaluate the effectiveness of ground improvement work.

INTRODUCTION

Loose sands can compress during seismic loading. In saturated loose sands this can result in cyclic liquefaction and subsequent settlements as excess pore pressures dissipate. In dry or partly saturated sands, seismic loading can result in densification (seismic compression) that can also lead to settlements. Stewart et al (2001) documented post earthquake settlements in partly saturated hillside fills for the Northridge earthquake. Currently, a popular method to evaluate earthquake induced settlements in dry sands is the approach proposed by Pradel (1998) which was based on Tokimatsu and Seed (1987), uses Standard Penetration Test (SPT) results and is applicable only to clean sands. Stewart and Wang (2003) introduced an update to the Pradel (1998) method in an effort to extend the approach to compacted fills and to include non-plastic silty sands and low-plastic clays. However, the Stewart and Wang (2003) method requires samples. This paper introduces a simple modification of the Pradel (1998) method based on Cone Penetration Test (CPT) results and is extended to cover a wide range of unsaturated soils.

The CPT has major advantages over traditional methods of field site investigation such as drilling and sampling since it is fast, repeatable and economical. In addition, it provides near continuous data and has a strong theoretical background. These advantages have lead to a steady increase in the use and application of the CPT in North America and many other places around the world. Lunne et al. (1997) provided a detailed description of developments in CPT equipment, procedures, checks, corrections and standards. Most CPT systems today include pore pressure measurements (i.e. CPTu) and provide CPT results in digital form. The addition of shear wave velocity (Robertson et al., 1986) is also becoming increasingly popular (i.e. SCPTu). Hence, it is now more common to see the combination of cone resistance (q_c), sleeve friction (f_s), penetration pore pressure (u) and, sometimes, shear wave velocity (V_s) measured in one profile.

A common complaint about the CPT is that it does not provide a soil sample. Although it is correct that a soil sample is not normally obtained during the CPT, most commercial CPT operators also carry simple push-in soil samplers that can be pushed using the CPT installation equipment to obtain a small (typically 25 mm diameter) disturbed soil sample of similar size to that obtained from the SPT. From these samples, typical soil properties, such as Attberg limits, moisture content, and particle size distribution, can be obtained. The preferred approach and often more cost effective solution is to obtain a detailed continuous stratigraphic profile using the CPT, then to move over a short distance (< 1m) and push a small diameter soil sampler to obtain discrete selective soil samples in critical layers/zones that were identified by the CPT. The push rate to obtain the soil sample can be significantly faster than the 2 cm/s required for the CPT and sampling can be rapid and cost effective for a small number of discrete samples.

MODIFIED PROCEDURE OF PRADEL

The Pradel (1998) method is based on the Tokimatsu and Seed (1987) approach that involves the following basic steps:

1. Determine the average cyclic shear stress, τ_{av}, induced by the earthquake,
2. Determine the small strain (maximum) shear modulus, G_o of the soil,
3. Determine the cyclic shear strain, γ, and the shear modulus, G, which are compatible with τ_{av}, G_o, from a chosen set of experimental curves relating γ, to G/G_o,
4. Determine the volumetric strain, ε_{vol}, which is a function of the cyclic shear strain, γ, and the earthquake magnitude, M.

The procedure is essentially a simplified version of the method proposed by Seed and Silver (1972) that was based on the findings of Silver and Seed (1971) which suggested that the settlement of dry sand is a function of the cyclic shear strain, γ, the number of strain cycles, N_c, and the relative density of the sand.

This paper presents a modification of the Pradel (1998) method by following the same basic steps but using CPT results. Each step is described in detail below.

DETERMINATION OF CYCLIC SHEAR STRESS

Pradel (1998) suggested using the simplified approach to estimate the average cyclic shear stress, τ_{av}, induced at a depth z, first proposed by Seed and Idriss (1971). The average cyclic shear stress induced during an earthquake can also be estimated via the cyclic stress ratio (CSR), as defined by Youd et al (2001), as follows;

$$\tau_{av} = CSR\ \sigma'_{vo} \qquad (1)$$

$$\tau_{av} = 0.65\ (a_{max}/g)\ \sigma_{vo}\ r_d \qquad (2)$$

The r_d value is related to the earthquake magnitude and the depth, according to Youd et al (2001) or by Boulanger & Idriss (2004, 2006).

DETERMINATION OF SMALL STRAIN SHEAR MODULUS

The small strain shear modulus, G_o, can be directly measured using a seismic CPT (SCPT) to obtain the shear wave velocity, V_s, where;

$$G_o = \rho\ (V_s)^2 \qquad (3)$$

Where $\rho = \gamma/g$

γ = unit weight

g = acceleration due to gravity

If a seismic CPT is not available, it is possible to estimate the shear wave velocity and small strain shear modulus from the CPT using recently updated correlations, suggested by Robertson (2009), using;

$$G_0 = 0.0188\ [10^{(0.55 I_c + 1.68)}]\ (q_t - \sigma_{vo}) \qquad (4)$$

Note that G_o is in same units as the net cone resistance. I_c is the soil behavior type index determined using normalized CPT parameters, Q_{tn} and F_r, as follows:

$$I_c = [(3.47 - \log Q_{tn})^2 + (\log F_r + 1.22)^2]^{0.5} \qquad (5)$$

Where:

$$Q_{tn} = [(q_t - \sigma_v)/p_a](p_a/\sigma'_{vo})^n \qquad (6)$$

$$F_r = [(f_s/(q_t - \sigma_{vo})]\ 100\% \qquad (7)$$

$(q_t - \sigma_v)/p_a$ = dimensionless net cone resistance,

f_s = cone sleeve friction,

$(p_a/\sigma'_{vo})^n$ = stress normalization factor

$n = 0.381\ (I_c) + 0.05\ (\sigma'_{vo}/p_a) - 0.15$

p_a = atmospheric pressure in same units as q_t and σ_v

Equation 4 was developed for uncemented, Holocene-age soils (Robertson, 2009). The estimated values of G_o will be conservatively low for older soils. Clearly, it is preferred if the in-situ shear wave velocity is measured during the CPT.

DETERMINE CYCLIC SHEAR STRAIN

Pradel (1998), based on the experimental results of Iwasaki et al (1978), and confirmed by Stewart and Wang (2003), developed a simplified relationship between cyclic shear strain, γ, and ratio of average shear stress and small strain shear modulus, τ_{av}/G_o, as follows;

$$\gamma = \left[\frac{1 + a \cdot e^{b \cdot R}}{1 + a}\right] \cdot R \cdot 100 \tag{8}$$

$$R = \frac{\tau_{av}}{G_0} \quad (\text{Note } \tau_{av} \text{ and } G_0 \text{ same units})$$

$$a = 0.0389 \cdot \left(\frac{p}{p_a}\right) + 0.124$$

$$b = 6400 \cdot \left(\frac{p}{p_a}\right)^{-0.6}$$

$$p = 1/3(1 + 2K_0)\sigma_{vo}$$

Stewart and Wang (2003) suggested that the parameters ‘a’ and ‘b’ would be somewhat soil-type dependant.

DETERMINE VOLUMETRIC STRAINS

Pradel (1998), based on the results of Silver and Seed (1971) and Tokimatsu and Seed (1987), suggested that the volumetric strain after 15 cycles, $\varepsilon_{vol(15)}$, can be estimated from normalized SPT penetration resistance, $(N_1)_{60}$, in clean sands, using;

$$\varepsilon_{vol(15)} = \gamma \cdot \left[\frac{(N_1)_{60,cs}}{20}\right]^{-1.20} \tag{9}$$

This can be modified to use CPT penetration resistance, using well established correlations between SPT and CPT penetration resistance, suggested by Lunne et al (1997).

Jefferies and Been (2006) showed that soils with the same state parameter (ψ) have the same response to loading. Robertson (2009) showed that soils with the same ψ have the same normalized clean sand equivalent penetration resistance, $Q_{tn,cs}$. Hence, equation 9 can be modified in terms of CPT $Q_{tn,cs}$ to extend the relationship to a wide range of soils. The correlation between SPT and CPT (Lunne et al, 1997) can be extended to apply to clean sand equivalent values, since the concept of equivalence with state parameter (Jefferies and Been, 2006) applies to both clean sand equivalent SPT and CPT values. Hence, the modified version is;

$$(N_1)_{60,cs} = \frac{Q_{tn,cs}}{8.5 \cdot \left[1 - \frac{I_c}{4.6}\right]} \tag{10}$$

$Q_{tn,cs}$ is determined as follows (Robertson and Wride, 1998):

$$Q_{tn,cs} = K_c \, Q_{tn} \tag{11}$$

Where K_c is a correction factor that is a function of grain characteristics (combined influence of fines content, mineralogy and plasticity) of the soil that can be estimated using I_c as follows:

$K_c = 1.0$ if $I_c \leq 1.64$ (12)

$K_c = 5.581\, I_c^3 - 0.403\, I_c^4 - 21.63\, I_c^2 + 33.75\, I_c - 17.88$

if $I_c > 1.64$ (13)

Pradel (1998) showed that the volumetric strain, ε_{vol}, during a design earthquake is defined as;

$$\varepsilon_{vol} = \varepsilon_{vol(15)} \cdot \left[\frac{N_c}{15}\right]^{0.45} \qquad N_c = (M - 4)^{2.17} \tag{14}$$

Where N_c is the equivalent number of cycles for an earthquake with magnitude, M. Hence, combining equations 8 and 9 with 10 and 14, provide a method to estimate the seismic compression volumetric strain from CPT results for a wide range for soils.

CALCULATION OF SETTLEMENT

Based on the above concepts, the vertical settlement, S, due to seismic compression, can be determined using the modified Pradel (1998) method, where the volumetric strain is doubled in order to take into account the multidirectional nature of earthquake shaking (Pyke et al., 1997);

$$S = 2 \cdot \int_0^{GWT} \varepsilon_{vol} \cdot dz \tag{15}$$

EXAMPLE

Based on the above procedure, the authors analyzed the seismic compression settlement for a ground improvement project below an underground structure in southern California.

The site has a design peak ground surface acceleration of a_{max} = 0.83g for M = 6.8 magnitude earthquake. The subsurface ground conditions at the site are fill and native soils, composed of predominantly silty sands with inter-bedded sands and some sandy clay to depths of approximately 21m (70 feet) below the ground surface. The geotechnical investigation indicated that the upper 9m (30 feet) of the native soils are generally loose to medium dense sand and the sandy clay soils are generally stiff to very stiff with a plasticity index (PI) of about 19%. Dense to very dense cemented soils were encountered at depths greater than 9m (30 feet). Ground water during the site investigation was at about 12m (40 feet) below ground surface.

The soil improvement program was focused on mitigation of the soil liquefaction hazard and to reduce seismic settlement of the sandy soils above the water table.

The ground improvement contractor constructed 0.91m (3 feet) diameter vibro-compacted stone columns utilizing a rectangular spacing of 2.5m x 2.4m (8.3 feet), to achieve a replacement ratio of 10.6%. In the post-construction analysis, an equivalent 2.48m x 2.48m (8.16 feet) square pattern was used to simulate the 10.6% replacement ratio. Due to underground utilities in parts of the site, compaction grouting was also performed, with diameters up to 0.68m (2.25 feet) in order to inject as much grout volume as possible into the loose sandy soil without excessive ground heave. With a primary and a secondary grid of compaction grouting column at the midpoint, the grouting area replacement ratio achieved was 12%. The compaction grout material had a minimum unconfined compressive strength of 2,000kPa (300 psi) at the 28 days.

Following installation of the stone columns and compaction grouting columns, verification tests were performed by 32 cone penetration test (CPT) soundings, complemented by two boreholes with SPT and samples. The CPT soundings extended at least to the bottom of the stone columns or compaction grouting columns at about 12m (40 feet). The post improvement SPT soil samples were used for basic soil property laboratory tests. These laboratory tests included sieve analysis, hydrometer tests, Atterberg limits, and natural soil moisture contents.

The CPT results were used to evaluate the post-earthquake settlements. The design criteria required a differential seismic induced settlement less than 12.5mm (0.5 inch) and total seismic settlement less than 25mm (1 inch).

Above the water table, the seismic compression settlement was estimated based on the above CPT method. No liquefaction was predicted below the ground water level and the focus of the analysis was on the seismic compression of the dry soils above the water table.

Figure 1 presents a summary of the results from CPT-7 compared to an adjacent borehole SPT-2. The CPT soil behavior type (SBT) index, I_c, was below 2.60 for most of the deposit where the soils were sandy, except between a depth of 7 m to 8.5 m (23 feet to 28 feet) and again at 11.9 m (39 feet), where the soils were sandy clay. Figure 1 also compares the measured SPT values of $(N_1)_{60,cs}$, corrected for fines content using the method described by Youd et al (2001), with the equivalent values of $(N_1)_{60,cs}$ obtained from the CPT using equation 10. In general, there is good agreement between the measured $(N_1)_{60,cs}$ values and the CPT-based values. Also included in Figure 1 is a comparison between the measured fines content from SPT samples and the estimated apparent fines content based on the CPT method described by Robertson and Wride (1998). Again there is general good agreement. Based on the above CPT method to estimate seismic compression of dry soils, the cyclic shear strains during the design earthquake are between 0.05% and 0.20% and the average induced volumetric strains are between 0.01% and 0.15%.

The stone columns (or compaction grouting columns) have a higher shear modulus than the surrounding soils, and will attract more seismic shear load, as "shear reinforcement". According to Baez (1995) and Baez and Martin (1993), the CSR value in the surrounding soil (CSR_s) will be reduced to:

$$CSR_s = K_G * CSR \tag{16}$$

Where:

$$K_G = \frac{1}{G_r}\frac{1}{\left[A_r + \frac{1}{G_r(1-A_r)}\right]} \tag{17}$$

The cyclic shear stress in the soil will be reduced as a function of the stone column replacement ratio (A_r) , as well as the shear modulus ratio, ($G_r = G_{sc}/G_{soil}$). The site ground improvement design replacement ratio is 0.106. The authors conservatively used a shear modulus ratio of 3.0. Therefore, the soil CSR reduction ratio, K_G is 0.825.

The reduced shear stress in the treated soil profile can be expressed as:

$$\tau_{av} = CSR_s * \sigma_v^{'} \qquad (18)$$

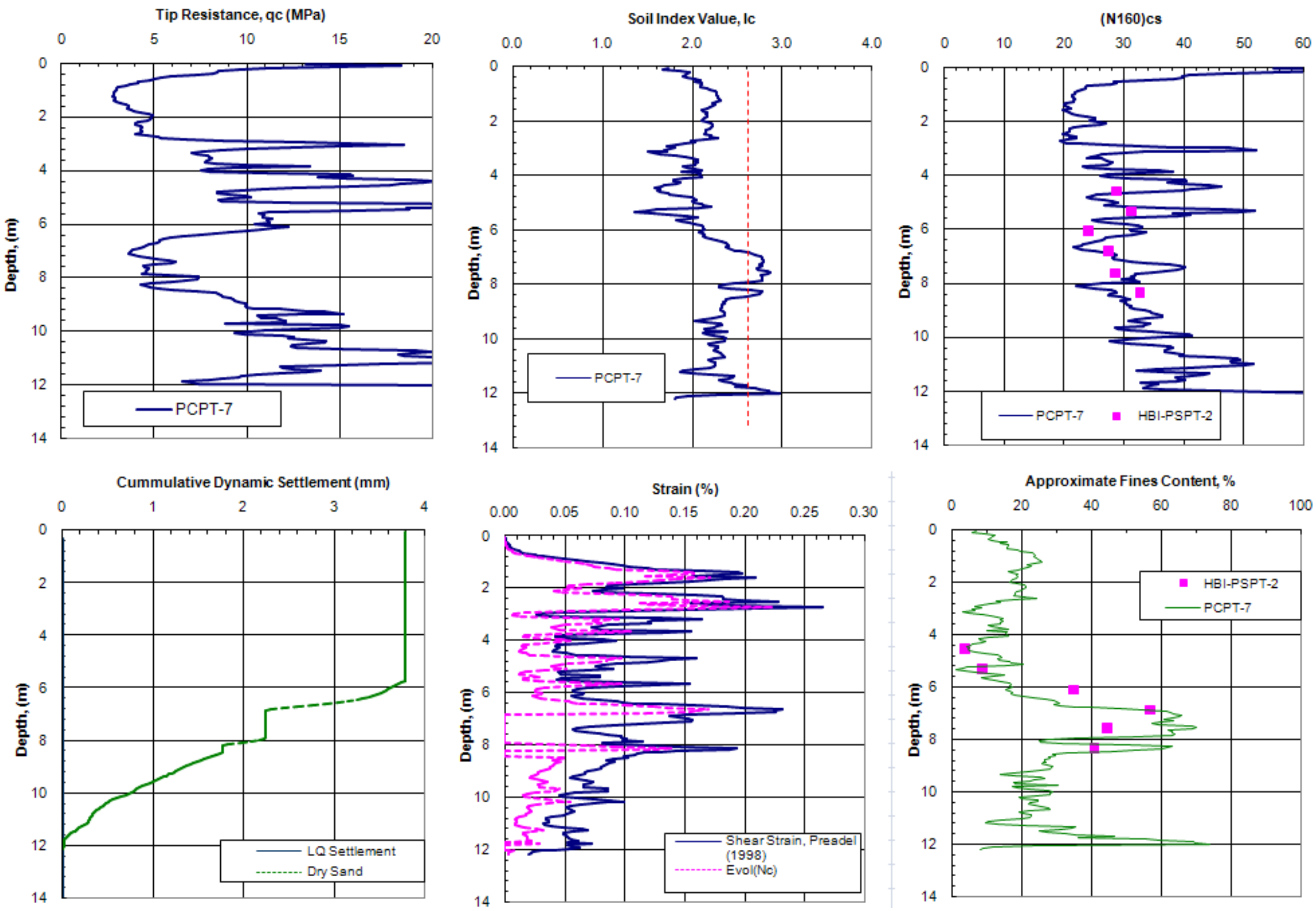

Fig. 1. Example comparison between CPT and adjacent SPT

With both vibro-compaction stone column and compaction grouting treatment, the expansion of the gravel column or the low mobility grout forces the surrounding soils away from the column, and this action results in an increased lateral soil stress (Kirsch, 2006). Hence, the treated soil no longer has the original in-situ horizontal K_0 stress condition as described in Pradel's paper. The authors used σ_v instead of the average pressure (p), and hence, assumed $K_0 = 1.0$.

To be conservative and to simplify the analysis, the authors ignored the stone column (or compaction grouting column) reinforcement along the vertical direction, and calculated the unsaturated seismic compression settlement in the soil profile.

The authors also calculated the seismic settlement below the ground water table. The liquefaction induced settlement calculation was performed according to Youd et al (2001) using the CPT method described by Zhang et al (2002).

Since the site design water table depth is relatively deep, and in some areas deeper than CPT penetration depth, the dry seismic settlement dominates the calculated dynamic settlement. The total seismic settlement was calculated from the bottom penetration depth of each CPT to the bottom elevation of the underground structure. In the seismic settlement analysis, the thin layer correction was not used.

SUMMARY

A simple CPT-based modification of the Pradel (1998) method is proposed to calculate the seismic compression settlement in unsaturated soils. The method provides a continuous evaluation of seismic compression of complicated soil profiles and a framework for sensitivity analyses. The method was evaluated at site where vibro-compacted stone columns and compaction grouting were adopted to mitigate seismic settlement. The proposed simple modification of the Pradel method provided a valuable tool to evaluate the effectiveness of the ground improvement work.

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