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PROCEDURE TO EVALUATE SEISMIC SETTLEMENT IN DRY SAND BASED ON SHEAR WAVE VELOCITY

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ABSTRACT

Seismic settlement in dry sands is a major hazard especially in inland and desert areas where loose sandy deposits exist and groundwater is relatively deep. Other than research performed for liquefaction-induced settlement, studies on seismic settlement of dry sand are limited although Silver and Seed (1971) began to pioneer the work in the early 1970's. Moreover, most of the existing methods for estimating seismic settlement in dry sands are based on the standard penetration test. This paper presents a new method for evaluating seismic settlement in dry sand based on shear wave velocity. A detailed procedure, including the calculations of cyclic shear strain and volumetric strain, is discussed. Special attention is paid to the discussion of the relationship between relative density and shear wave velocity and the limiting volumetric strain. A case study is provided to illustrate the application of the procedure.

Introduction

As the world's population increases, human activities and urban developments have moved toward the inland and desert areas, such as those in southern California, where loose sandy deposits exist and groundwater is relatively deep. These loose sand deposits tend to densify when subjected to seismic shaking. In an earthquake event, the balanced structure of the sand skeleton is disturbed by repeated cyclic shear loading. The sand particles tend to move to a more stable position under a combination of cyclic shear loads and confining pressures. The rearrangement of sand grains leads to a densification of the loose dry sand deposits. Although few case histories have been reported for the seismic settlement observed in natural dry sand deposits (most probably due to the lack of human activities in these undeveloped areas), the occurrence of ground deformations in unsaturated engineered fills has been noted following a number of earthquakes. According to Steward et al. (2002), the earliest available reference to this type of event is by Lawson (1908) who summarized the observed ground cracking in hillside areas caused by the 1906 San Francisco earthquake. After the 1971 San Fernando earthquake, a dry sand settlement of 10 to 60 cm was reported to have occurred in unsaturated engineered fill (Fukuoka, 1971). After the 1994 Northridge earthquake, significant seismic settlement was also

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observed in hillside fills (Steward et al. 2001). Steward et al. pointed out that "such deformation did not typically damage structures to the extent that life safety was threatened. However, the economic losses … were large…"

The pioneering work in investigating seismic settlement in dry sand was done by Silver and Seed (1971), Seed and Silver (1972), and Pyke et al. (1975) in the early 1970's. There appears to be no more research for more than 10 years until the works of Tokimatsu and Seed (1987) and Pradel (1998). More recent efforts were contributed by Steward and Whang (2003) for evaluating the seismic compression deformation in compacted fill and Yi (2009a) for estimating seismic settlement in dry sand based on cone penetration test (CPT) data. These methods are either based on standard penetration test (SPT) data (Tokimatsu and Seed 1987; Pradel 1998; Steward and Whang 2003) or CPT data (Yi 2009a). Few published papers can be found that address the calculation of seismic settlement in dry sand based on shear wave velocity (V_s). Therefore, this paper presents a method for evaluating seismic settlement in dry sands based on shear wave velocity. The method includes a detailed procedure of calculations including cyclic shear stress, shear strain, volumetric strain, and seismic settlement. Special attention was paid to the discussion of the relationship between relative density and shear wave velocity and the limiting volumetric strain. The performance of the proposed method was evaluated by comparing it with the results calculated based on SPT and CPT data, utilizing current widely accepted methods.

Seismic Settlement of Dry Sand

The procedure presented herein is similar to the method originally proposed by Seed and Silver (1972) and later modified by Tokimatsu and Seed (1987) and Pradel (1998). Laboratory test results by Silver and Seed (1971) were utilized and extended to the relationship with shear wave velocity. A new concept of limiting volumetric strain is also proposed.

Evaluation of Cyclic Shear Strain

Cyclic Shear Stress, τ_{av}

In the simplified procedure for liquefaction evaluation, Seed and Idriss (1971) formulated Eq. 1 for the calculation of the average equivalent uniform cyclic shear stress caused by an earthquake and assumed it to be 0.65 of the maximum induced stress.

$$\tau_{av} = 0.65 \cdot (a_{\max} / g) \cdot \sigma_{v0} \cdot r_d \tag{1}$$

where $a_{\max}$ is the peak horizontal acceleration at ground surface generated by the earthquake, g is the acceleration of gravity, σ_{v0} is the total and effective overburden stresses, and r_d is a stress reduction coefficient. Several equations have been proposed by individuals for the calculation of r_d (Seed and Idriss 1971; Lao and Whitman 1986). After investigating the effect of earthquake moment magnitude (M), Idriss (1999) proposed the following equations.

$$r_d = \exp[\alpha(z) + \beta(z) \cdot M] \tag{2a}$$

$$\alpha(z) = -1.012 - 1.126 \sin\left(z / 11.73 + 5.133\right) \tag{2b}$$

$$\beta(z) = 0.106 + 0.118\sin(z/11.28 + 5.142) \tag{2c}$$

in which z is the depth below ground surface in meters and the arguments inside the *sine* terms are in radians.

Cyclic Shear Strain, γ

According to Hooke's law, shear stress (τ) is calculated as the product of the shear modulus (G) and shear strain (γ) for a linear elastic material. Soil usually behaves as a nonlinear material under seismic loading. Utilizing the nonlinear relationship between the shear modulus ratio (G/G_0) and shear strain, the cyclic shear strain induced in the soil can be determined by the following equation.

$$\gamma = \frac{\tau_{av}}{G_0 \cdot (G/G_0)} \tag{3}$$

where G_0 is the maximum shear modulus of the soil and is determined from the mass density (ρ) and shear wave velocity (V_s) of the soil using the following equation.

$$G_0 = \rho \cdot V_s^2 \tag{4}$$

The calculation of γ in Eq. 3 usually requires iterations. Using the relationships between the shear modulus and the shear strain obtained experimentally by Iwasaki et al. (1978), Pradel (1998) formulated an equation which avoids the iterative process.

$$\gamma = \left[\frac{1 + a \cdot \exp(b \cdot \tau_{av}/G_0)}{1 + a}\right] \cdot \frac{\tau_{av}}{G_0} \times 100 \quad (\%) \tag{5a}$$

where

$$a = 0.0389(p/p_a) + 0.124 \text{ and } b = 6400(p/p_a)^{-0.6} \tag{5b}$$

where p is the average stress $(= \frac{1}{3}(1+2K_0)\sigma_{v0})$, p_a is the reference stress of 100 kPa or atmospheric pressure, and K_0 is the coefficient of lateral earth pressure at rest. Mayne and Kulhawy (1982) found that K_0 depends on the internal frictional angle (ϕ) of soil and the overconsolidation ratio (OCR) as shown in following equation.

$$K_0 = (1 - \sin\phi)OCR^{\sin\phi} \tag{6}$$

Evaluation of Cyclic Volumetric Strain

Relative Density, D_R

Relative density (D_R) is an important parameter in the laboratory testing of the dynamic properties of sandy soils. It represents the initial state of sands and is usually expressed as a function of soil resistance such as SPT blow counts (Terzaghi and Peck 1967) or CPT tip resistance (Yi, 2009a). To utilize V_s data, a relationship between D_R and V_s is necessary. The existing relationship between D_R and V_s is not available from published research. However, this relationship can be established by utilizing the relationships between relative density and SPT blow counts and between shear wave velocity and SPT blow counts.

Several relationships between relative density and SPT blow counts have been proposed in the past (Terzaghi and Peck 1967; Tokimatsu and Seed 1987; Idriss and Boulanger 2008). Data points collected by Mayne et al. (2002) and Tokimatsu and Seed (1987) are re-plotted in Fig. 1 showing the relationship between relative density and SPT blow counts corrected to an energy ratio of 60% with an overburden stress of 1 atm. $(N_1)_{60cs}$ is used as the abscissa in Fig. 1 to represent the equivalent clean sand $(N_1)_{60}$. Equations proposed by Terzaghi and Peck (1967) and Idriss and Boulanger (2008) as well as the curve by Tokimatsu and Seed (1987) are also plotted in Fig. 1. It can be seen that an average relationship can be better expressed by Eq. 7.

$$D_R = 100\sqrt{(N_1)_{60cs}/52} \quad (\%) \tag{7}$$

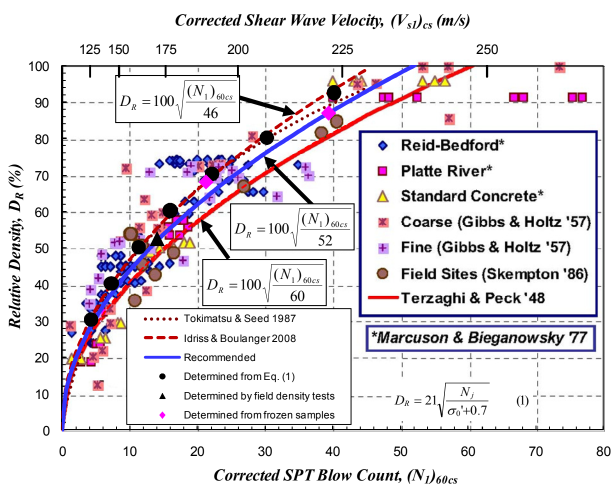


Figure 1 Relationship between relative density and corrected SPT blow counts and corrected shear wave velocity (data from Mayne et al. 2002 and Tokimatsu and Seed 1987)

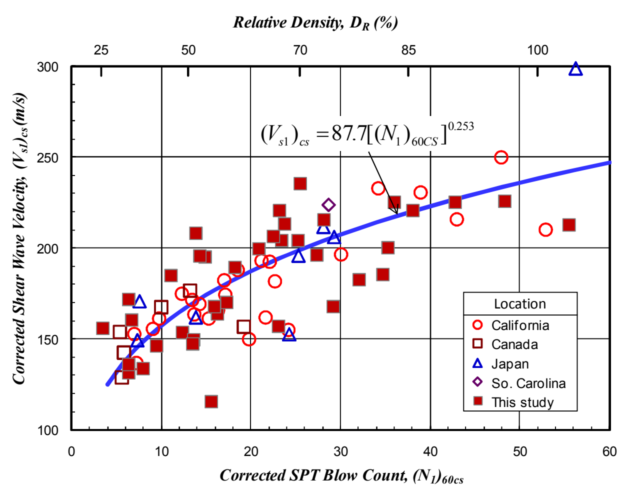


Figure 2. Relationship between corrected shear wave velocity and corrected SPT blow counts for uncemented, Holocene sands (modified from Andrus et al. 2004)

Figure 2 shows the relationship between $(N_1)_{60cs}$ and $(V_{s1})_{cs}$ based on the data pairs collected by Andrus et al. (2004) from different regions and by this author from California. By nonlinear regression analysis, Andrus et al. (2004) obtained a power curve as expressed by Eq. 8.

$$(V_{s1})_{cs} = 87.7[(N_1)_{60CS}]^{0.253} \tag{8}$$

where the subscript *cs* is the abbreviation for *<u>c</u>lean <u>s</u>and* (soils with 5% or less fines), and $(V_{s1})_{cs}$ is the overburden stress-corrected shear wave velocity as defined in Eq. 8 to account for the influence of the state of stress in soil.

$$(V_{s1})_{cs} = K_{cs}V_{s1} = K_{cs}V_s(p_a/\sigma_{v0}')^{0.25} \tag{9}$$

where V_{s1} is the overburden stress-corrected shear wave velocity of sandy soils and K_{cs} is a fines content (*FC*) correction factor. Juang et al (2002) suggested a relationship for estimating K_{cs} based on V_{s1} and *FC*. Generally, measured *FC* from soil samples is preferred for this correction. However, if the measured data is not available, *FC* estimated by other methods such as from CPT data (Yi, 2009a) can also be used.

By combining Eq. 7 and Eq. 8, the relationship between relative density and corrected shear velocity can be derived as shown in following equation.

$$D_R = 17.974[(V_{S1})_{cs}/100]^{1.976} \quad (\%) \tag{10}$$

By using this equation, the relationship between $D_R \sim (N_1)_{60cs} \sim (V_{s1})_{cs}$ is also plotted in Figs. 1 and 2.

Relationship between volumetric strain and shear strain of dry clean sand

Silver & Seed (1971) conducted a series of unidirectional cyclic shear tests on dry quartz sand (Crystal Silica No. 30) with relative densities of approximately 45, 60, and 80%, and obtained relationships between volumetric and cyclic shear strains. Fig. 3 summarizes these relationships after 15 uniform strain cycles which is equivalent to an earthquake with a

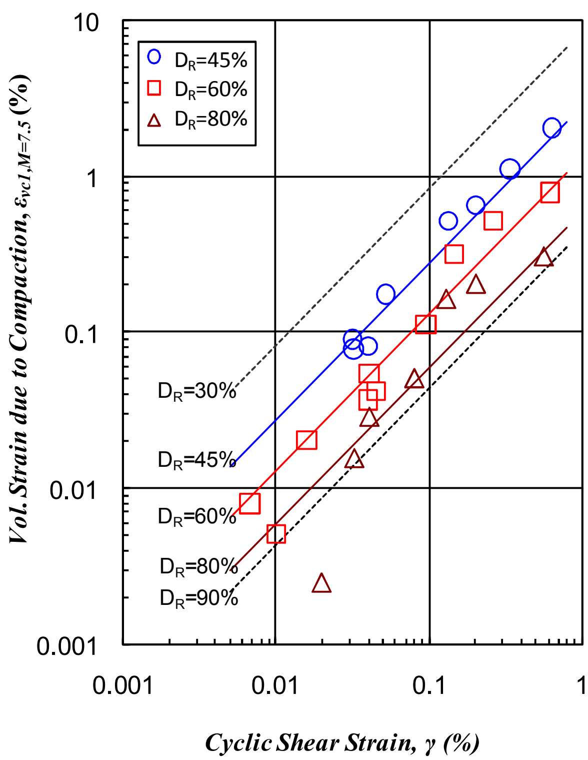


Figure 3. Relationship between volumetric strain and shear strain for dry clean sands (after Silver & Seed 1971)

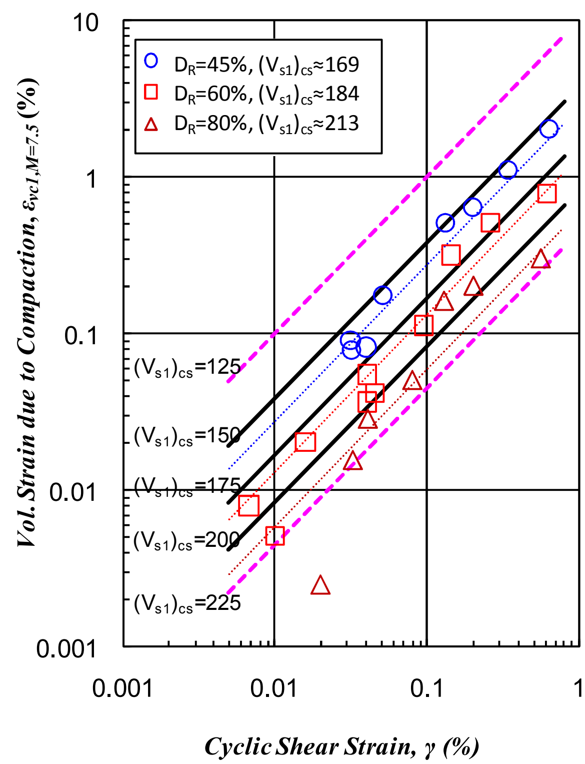


Figure 4. Relationship between volumetric strain, shear strain and corrected shear wave velocity for dry clean sands

moment magnitude of 7.5. Coupling the relationships presented in Fig. 3 and Eq. 10 leads to the relationship between volumetric strain and shear strain for clean sands with different corrected shear wave velocity $(V_{s1})_{cs}$ as shown in Fig. 4. These relationships can be expressed by following equation.

$$\varepsilon_{vc1,M=7.5} = 0.329(D_R/100)^{-2.68}\gamma = 32.715[(V_{s1})_{cs}/100]^{-5.296}\gamma \tag{11}$$

where, $\varepsilon_{vc1,M=7.5}$ represents the volumetric strain obtained by unidirectional cyclic excitation with intensity equivalent to an earthquake with a moment magnitude of 7.5.

Corrections for multidirectional shaking and earthquake magnitude

Pyke et al. (1975) continued the research of Silver and Seed by performing multidirectional as well as unidirectional cyclic shear tests and concluded that "the the settlement caused by combined horizontal motions are about equal to the sum of the settlement caused by the components acting alone." As such, the volumetric strain for any magnitude should be calculated using the following equation to account for the effects of multidirectional shaking and earthquake magnitude.

$$\varepsilon_{vc,M} = K_{vc,M} \cdot [2 \cdot \varepsilon_{vc1,M=7.5}] \tag{12}$$

where $K_{vc,.M}$ is the earthquake magnitude correction factor. By reviewing previous studies, Tokimatsu & Seed (1987) summarized the volumetric strain ratio related to number of representative cycles at $0.65 \cdot \tau_{av}$. Data by Tokimatsu and Seed (1987) was replotted in Fig. 5 versus earthquake magnitude. The average values can be expressed by the following equation.

$$K_{vc,M} = \varepsilon_{vc,M} / \varepsilon_{vc,M=7.5} = 0.26M - 0.96 \tag{13}$$

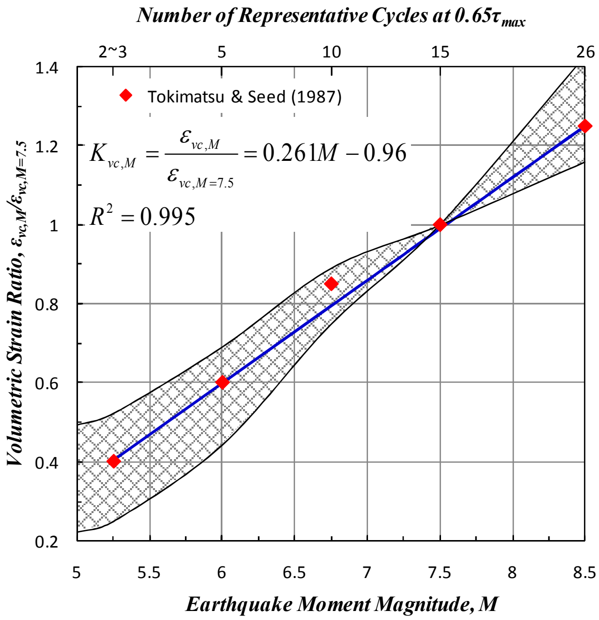
Ultimate Volumetric Strain, $\varepsilon_{v,ult}$, and Limiting Volumetric Strain, $\varepsilon_{v,lim}$

In the application of the existing methods (Tokimatsu and Seed 1987; Pradel 1998), one possible scenario is that under certain conditions, such as a very large earthquake or very loose sand strata, the calculated seismic settlement of dry sand is too large to be reasonable. Therefore, knowing the anticipated maximum value is very important in any kind of empirical method. It is well known in geotechnical laboratory testing that the void ratios, e, e_{max}, and e_{min}, of a coarse grained soil represent its natural (in situ), loosest, and densest states, respectively. Theoretically, the volume change from its natural state to its densest state represents the limitation of any anticipate volume change. Therefore, the maximum value of the volumetric strain (termed as *ultimate volumetric strain*, $\varepsilon_{v,ult}$, hereafter) can be calculated by the following equation.

$$\varepsilon_{v,ult} = \frac{e - e_{min}}{1 + e} = \frac{(1 - D_R)(e_{max} - e_{min})}{1 + e_{max} - D_R(e_{max} - e_{min})} \times 100\% \tag{14}$$

Because the densest state is obtained from laboratory testing, the real maximum value of volumetric strain (termed as *limiting volumetric strain*, $\varepsilon_{v,lim}$, hereafter) in the field is less than $\varepsilon_{v,ult}$. Ishihara and Yoshimine (1992) observed that the volumetric strain that occurred during post-liquefaction reconsolidation of Fuji River sand was limited to a value that is uniquely related to the initial D_R (Yoshimine et al 2006) or $(V_{s1})_{cs}$ (Yi 2009b) as shown in following equation.

$$\varepsilon_{v,\lim} = 1.5 \cdot \exp(-0.025 D_R) = 12 \cdot \exp\left[-0.449\left((V_{s1})_{cs}/100\right)^{1.976}\right] \tag{15}$$



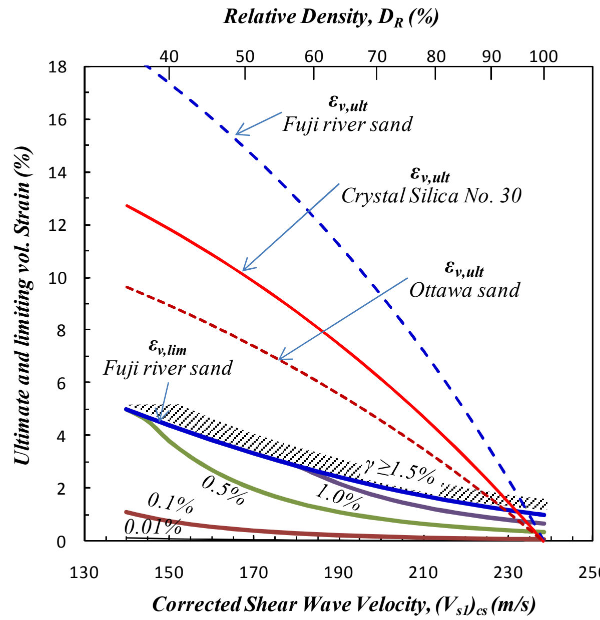


Figure 5. Relationship between volumetric strain ratio and earthquake moment magnitude (modified from Tokimatsu and Seed, 1987)

Figure 6. Relationship between ultimate and limiting volumetric strain and corrected shear wave velocities including the variation at a given shear strain

Figure 6 compares the variation of $\varepsilon_{v,ult}$ with $(V_{s1})_{cs}$ or D_R for Fuji river sand (e_{max}=1.064, e_{min}=0.529), Crystal Silica No. 30 (e_{max}=1.042, e_{min}=0.668), and Ottawa 20/30 sand (e_{max}=0.747, e_{min}=0.501). The $\varepsilon_{v,lim}$ of Fuji River sand (Eq. 15) and the variation of $\varepsilon_{vc,M}$ for M=7.5 (Eqs. 13 and 12) are also shown in the figure. For Fuji River sand, the limiting volumetric strain is approximately 25% of the ultimate volumetric strain for D_R ranging between 40 and 80%. Although, $\varepsilon_{v,lim}$ of Fuji River sand is obtained under saturated conditions, the limiting volumetric strain under natural condition should not exceed this value because of the existence of moisture content in the natural sand deposit and the anticipated suction caused by this moisture content as well as the lubricating act of water in saturated sands. It seems that both $\varepsilon_{v,ult}$ and $\varepsilon_{v,lim}$ are soil type dependent. Therefore, further research is necessary to evaluate the relationship between $\varepsilon_{v,ult}$ and $\varepsilon_{v,lim}$ for various sands. However, it is the opinion of this author that if further information is not available, Eq. 15 can be used conservatively as the limiting value of Eq. 12.

Seismic Settlement of Dry Sand

Similar to the calculation of liquefaction-induced settlement, the seismic settlement of dry sand can be evaluated by equating the vertical strain to the volumetric strain and then integrating the vertical strains over the depth interval of concern:

$$S_{v,1D} = \int_0^{Z_{\max}} \varepsilon_{vc,M} \cdot dz \tag{16}$$

Performance of the Proposed Method

Although some case histories of seismic settlement in unsaturated engineered fills have been reported, few case histories are available for natural dry sand. Due to the other factors affecting the seismic settlement in unsaturated engineered fills, the simulation of this kind of case history is beyond the scope of this study. To demonstrate the performance of the proposed method, calculations were made on a site where SPT and seismic cone penetration test (SCPT) were performed in such proximity as to reasonably conclude that the subsurface soils are identical. Fig. 7 illustrates the results of calculated shear strain, volumetric strain, and seismic settlement distribution based on V_s data utilizing the proposed method and compares with those based on CPT and SPT data utilizing existing methods. For comparison, the measured SPT blow counts and CPT tip resistance were converted to shear wave velocity as shown in the first graph of Fig. 7. The interpreted relative densities are also shown in the figure. It can be seen that the calculated settlement based on V_s data utilizing the proposed method generally agrees with the results obtained from CPT and SPT data utilizing existing methods, although the results based on SPT data demonstrated higher settlements near surface. This difference is considered to be attributable to the inconsistence between measured field resistance utilizing different methods as shown in the near surface measurements of the first graph.

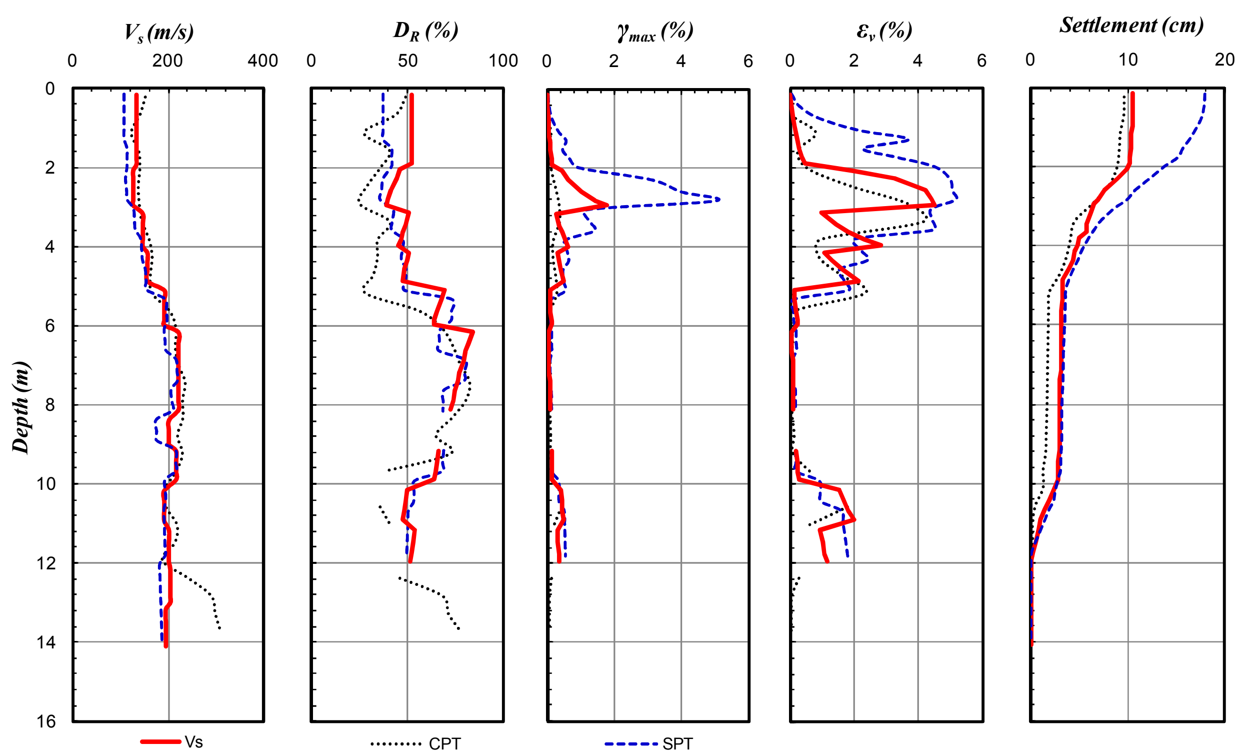


Figure 7. Example of calculated shear strain, volumetric strain, and settlement of dry sand for Site A based on V_s data, comparing with the results based on CPT and SPT data (a_{max}=0.4g)

Conclusions

SCPT is a cost effective method to obtain both cone penetration resistance and shear wave velocity with a small increase in the field investigation cost over the cost of conventional CPT soundings. Other than by the research for SPT and CPT data, the current application of V_s data is limited to the evaluation of liquefaction potential (Andrus and Stokoe 2000). This author made an effort to apply the V_s data in the evaluation of seismic settlement in both dry and saturated sand. The method to evaluate liquefaction-induced settlement based on V_s data has been proposed in a separated paper by this author (Yi, 2009b). This paper presented a procedure to evaluate seismic settlement in dry sand based on V_s data. Based on the results of previous studies, relationships with V_s have been proposed. The procedure has been presents in the order of the calculation sequence by a set of equations. The performance of the proposed method was verified by comparing the results obtained using existing methods for different field investigation data from the same location of a site. The outcome indicates that the proposed method is capable of providing consistent results with the existing methods applied to other field investigation data, such as SPT and CPT data. However, it should be noted that the predictions in this paper should not be taken to imply that level of accuracy obtained in the sample calculation can achieved for any given site with different exploratory data due to the anticipated inconsistence between field investigation data even from the same test location. Furthermore, the verification of the proposed method may be necessary when the case histories are available. Moreover, further laboratory verification of the relationship between ultimate and limiting volumetric strains for different soil types is considered to be necessary.

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