### Seismically Induced Settlement of Ground Experiencing Undrained Shaking and Laterally Constrained Compression

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| Complete List of Authors: | Mesri, Gholamreza; University of Illinois at Urbana-Champaign, Dept. of Civil and Environmental Engineering
                           | Shahien, Marawan; Tanta University, Geotechnical and Foundation Engineering
                           | Kane, Thierno; University of Illinois at Urbana-Champaign, Civil and Environmental Engineering |
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Seismically Induced Settlement of Ground Experiencing Undrained Shaking and Laterally Constrained Compression

Gholamreza Mesri, Marawan Shahien, and Thierno Kane

Abstract: A method is proposed for estimating seismically induced settlement of saturated sands experiencing undrained shaking and laterally constrained compression. An empirical relationship is developed between the seismic coefficient of vertical compression, $m_{vs}$, and standard penetration test blow count, $N_{60}$, as a function of factor of safety against liquefaction, $F_\ell$, based on data interpreted from 18 sets of laboratory cyclic direct simple shear tests and 23 cases of field observations of seismic liquefaction. The proposed method is compared with seismic settlement observed at 78 sites subjected to 7.1 to 8.0 magnitude earthquakes, and with predictions by the previous well-known methods of settlement analysis for undrained shaking. For silty-clayey sands, the significant effect of the plasticity of fines on seismic settlement is illustrated. The use of pre- or post-earthquake penetration resistance for back-analyses of field seismic settlement observations is examined. A tentative correction factor is suggested for seismic settlement estimated based on the assumption of undrained shaking and laterally constrained compression, for liquefied saturated sublayers at small distances from drainage boundaries or under buildings with small breadths, that may experience volumetric compression during ground shaking.

Keywords: earthquakes, sands, seismic settlement, undrained shaking, porewater pressure

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G. Mesri. Ralph B. Peck Professor of Civil Engineering, University of Illinois, Urbana-Champaign, Illinois, U.S.A.
M. Shahien. Professor of Geotechnical Engineering and Foundations at Tanta University, Tanta, Egypt.
T. Kane. Graduate student, University of Illinois, Urbana-Champaign, Illinois, U.S.A.
Corresponding Author: G. Mesri (email: gmesri@illinois.edu).
Seismic shaking of granular level ground disengages interparticle contacts and produces a tendency for compression. Because of the finite permeability and rapid shaking, and in the absence of drainage boundaries, the response of saturated ground to seismic events is often undrained in the sense that no compression takes place during the shaking process. The tendency for compression generates excess porewater pressure, $u'$, resulting in a decrease in effective vertical stress $\sigma'_v$. Subsequent to the shaking, excess porewater pressure dissipates, leading to an increase in effective vertical stress, recompression of the granular soil, and settlement of the ground surface. The previous well-known methods of settlement analysis for undrained seismic shaking are by Tokimatsu and Seed (1987) and Ishihara and Yoshimine (1992). These methods have been subsequently modified and refined (e.g. Shamoto et al. 1996, 1998; Zhang et al. 2002; Wu and Seed 2004; Lee 2007; Cetin et al. 2009; Juang et al. 2013). We present a third method for settlement analysis of undrained shaking of saturated granular ground, in the absence of lateral deformation, with ground surface settlement computed according to (Shahien 1998)

\[ S = \sum_{j=1}^{n} \left[ L_o m_{vs} u'_j \right] \]

where, for sublayer $j$, $[L_o]_j$ is the thickness, $[m_{vs}]_j$ is the seismic coefficient of vertical compression subsequent to undrained shaking, and $u'_j$ is the excess porewater pressure generated by undrained shaking, with a maximum value equal to the in situ effective overburden pressure, $[\sigma'_{so}]_j$. 


Seismic Shaking

Seismic shaking is a cyclic shearing process, quantified in terms of maximum ground surface acceleration as a measure of the intensity of shaking, and earthquake magnitude as a measure of the duration of shaking. The shear stress level may be expressed by the factor of safety against initial liquefaction, $F_\ell$, at which instant $u'$ becomes equal to $\sigma'_\sigma$

$$F_\ell = \frac{\text{Cyclic Resistance Ratio}}{\text{Cyclic Stress Ratio}} = \frac{\text{CRR}}{\text{CSR}}$$

where cyclic resistance is the undrained seismic shear strength of a granular soil subjected to an earthquake of Richter magnitude $M$ often expressed by equivalent number of uniform seismic shear stress cycles, $N_c$. In other words, the cyclic resistance, determined by the granular soil characteristics such as relative density and fines content and preconsolidation pressure, is the seismic shear stress required for an earthquake of magnitude $M$ to cause initial liquefaction. The CRR may be evaluated from the CRR versus $(N_1)_{c60}$ relationship for different fines content and $M$, such as that by Seed et al. (1984, 1985), where the normalized Standard Penetration Test (SPT) blow count, $(N_1)_{c60}$ and fines content are measures of the granular soil characteristics.

The cyclic stress is the shear stress by ground acceleration produced by the earthquake, and may be estimated by (Seed and Idriss 1971)

$$\text{CSR} = 0.65 \frac{a_{\text{max}}}{g} \frac{\sigma'_{\sigma}}{\sigma'_{\sigma_0}} r_d$$

where $a_{\text{max}}$ is the maximum ground surface acceleration in gals, $g$ is the acceleration due to gravity (980 gals), $\sigma'_{\sigma_0}$ is the total overburden pressure at the depth under consideration, and $r_d$ is a depth-
related seismic shear stress reduction factor (e.g. Iwasaki et al. 1978; Cetin et al. 2004; Idriss and Boulanger 2004, 2006).

Seismically-Induced Excess Porewater Pressure

The magnitude of excess porewater pressure generated by seismic shaking, $u'$, is related to the factor of safety against liquefaction, $F_\ell$ (Yoshimi et al. 1975; Tatsuoka et al. 1984). This is illustrated in Fig. 1a using data from direct simple shear tests on Fuji River sand subjected to $F_\ell$ values in the range of 0.6 to 1.8 (Nagase and Ishihara 1988; Ishihara and Yoshimine 1992). Fuji River sand has subangular grains, a median grain size of 0.40 mm, a uniformity coefficient of 3.2, fines content less than 5%, and minimum and maximum void ratio of 0.529 and 1.064, respectively. A constant $m_{vo}$ is defined as $\Delta \varepsilon_v / \Delta \sigma_\ell$, however, the actual relationship between $\varepsilon_v$ and $\sigma_\ell$ is likely to be non-linear as illustrated by Fig. 1b.

Based on undrained cyclic shear tests, Lee and Albaisa (1974), DeAlba et al. (1975), and Seed et al. (1975), proposed a relationship between $u'/\sigma_{vo}$ and $N_c/N_\ell$, where $N_\ell$ is the equivalent number of uniform shear stress cycles required to cause initial liquefaction. The relationship between $u'/\sigma_{vo}$ and $N_c/N_\ell$ has been examined by Tatsuoka et al. (1980). The empirical equation proposed by Seed et al. (1975) has been simplified to the following form (Xu 1991; Tanaka et al. 1994)

$$\frac{u'}{\sigma_{vo}} = \frac{N_c}{N_\ell}$$

(4)
The relationship between \(\log \frac{CRR}{CSR}\) and \(\log \frac{N_c}{N_\ell}\) is linear with a slope \(a\) in the range of -0.1 to -0.3 (Annaki and Lee 1977; Youd and Perkins 1978; Ishihara et al. 1978; Tatsuoka et al. 1980). Such a relationship is used to express \(u'/\sigma_{vo}\) in terms of \(F_\ell\)

\[
(5) \quad \log \frac{CRR}{CSR} = a \log \frac{N_c}{N_\ell}
\]

Substituting Eqs. 2 and 4 into Eq. 5

\[
(6a) \quad \frac{1}{a} \log F_\ell = \log \frac{u'}{\sigma_{vo}}
\]

Or

\[
(6b) \quad \frac{u'}{\sigma_{vo}} = F_\ell^{1/a}
\]

Using \(a = -0.2\), we obtain for \(F_\ell \geq 1\)

\[
(7a) \quad \frac{u'}{\sigma_{vo}} = \frac{1}{F_\ell}
\]

and for \(F_\ell < 1\)

\[
(7b) \quad \frac{u'}{\sigma_{vo}} = 1
\]

Equation 7a predicts a \(u'/\sigma_{vo}\) versus \(F_\ell\) near that proposed by Ishihara (1985). Note that existing laboratory studies show that parameter \(a\) can widely vary, e.g. -0.08 for Duncan Dam sand (Pillai and Stewart 1994), to -0.54 for Niigata sand (Okamura et al. 2003).
Compressibility Subsequent to Seismic Shaking

Laboratory measurements of compression of granular soils subsequent to undrained shaking, Table 1, (e.g. Nagase and Ishihara 1988; Ishihara and Yoshimine 1992) were used to compute values of seismic coefficient of vertical compression subsequent to seismic shaking, $m_{vs}$, and were correlated to relative density, $D_r$, expressed in terms of the penetration resistance $N_{60}$. The values of $(N_1)_{60}$ were estimated for the laboratory specimens from empirical correlations between $(N_1)_{60}$ and $D_r$ (Holtz and Gibbs 1979; Tokimatsu and Seed 1984; Skempton 1986; Yoshida et al. 1988; Cubrinovski and Ishihara 1999; Boulanger and Idriss 2004), using

$$(8) \quad (N_1)_{60} = 0.005 D_r^2$$

where relative density, $D_r$, is in percent. Then, the relationship between $(N_1)_{60}$ and $N_{60}$ proposed by Liao and Whitman (1985) was used to compute $N_{60}$

$$(9) \quad N_{60} = (N_1)_{60} \left( \frac{\sigma'_{vo}}{100} \right)^{1/2}$$

where effective overburden pressure $\sigma'_{vo}$ is in kPa.

The data on $m_{vs}$ were interpreted by assuming at any $F_\ell$ (e.g. Burland and Burbidge 1985, Terzaghi et al. 1996)

$$(10) \quad m_{vs} = \frac{b}{N_{60}^c}$$
Both $m_{vs}$ and $N_{60}$ are functions of effective confining pressure, but because the influence of the confining pressure is empirically similar on both quantities, it is considered unnecessary to correct $N$ for the effective overburden pressure.

The data for Fuji River sand are shown in Fig. 2, and values of $b$ and $c$ are summarized in Figs. 3a and 3b, respectively. These values of $b$ and $c$ were used to construct the empirical correlation in Fig. 4 for $m_{vs}$ in terms of $N_{60}$ relationship and $F_\ell$.

The limiting values of $m_{vs}$ corresponding to liquefaction observed in the laboratory as well as in the field are shown in Fig. 5 together with the computed $m_{vs}$ versus $N_{60}$ relationship for $F_\ell = 1.0$ and for the limiting condition in Fig. 4 (Shahien 1998). Thus, the highest values of $m_{vs}$ in Fig. 4 correspond to the liquefaction condition observed in the laboratory and in the field. The Burland and Burbidge (1985) relationship for coefficient of vertical compression and blowcount is shown in Fig. 5 for reference.

**Settlement Subsequent to Seismic Shaking**

Subsurface conditions at 78 sites of observed ground surface settlement subjected to undrained seismic shaking (Table 2, Shahien 1998) were used to evaluate the proposed method in terms of observed settlements, Fig. 6. Settlement analyses were also conducted using the methods by Tokimatsu and Seed (1987) and Ishihara and Yoshimine (1992). The seismic settlement by the proposed method was computed using Eq. 1 together with Eqs. 2, 3, 7, and 10. The proposed method applies a fines correction to $(N_1)_{60}$, to be described subsequently. The same fines content
correction was applied to the \((N_1)_{60}\) used for the methods by Tokimatsu and Seed (1987) and 

The ground surface settlement subsequent to seismic shaking, computed by the three methods, 
are compared in Fig. 7. For most cases, the settlements computed by the three methods are 
generally comparable. With some exceptions, the Ishihara and Yoshimine (1992) predicts 
settlements up to 100 mm more than the settlements by the proposed method, and up to 150 mm 
more than the settlement by Tokimatsu and Seed (1987). Thus, in general, the proposed method 
predicts a settlement somewhere in between those of Tokimatsu and Seed (1987) and Ishihara and 

Further Examination of the Three Methods

A series of settlement calculations were carried out using the three methods, together with 
\(N_{60}\) in the range of 5 to 20, \(\sigma_{vo}^*\) in the range of 50 to 150 kPa, and \(F_\ell\) in the range of 0.6 to 1.4. The 
objective of this exercise was to (a) examine the influence of these variables on the vertical strain 
resulting from undrained seismic shaking, and (b) provide an insight into differences, in some 
cases, in the computed settlements by the three methods.

For the proposed method, \(m_{vs}\) was obtained from Fig. 4 using values of \(N_{60}\) and \(F_\ell\). The 
increase in effective vertical stress subsequent to undrained shaking, equal to \(u'\), was computed 
using Eq. 7. The Ishihara and Yoshimine (1992) method requires \((N_1)_{60}\) and CSR to obtain vertical 
strain. For both Tokimatsu and Seed (1987) and Ishihara and Yoshimine (1992) methods, \((N_1)_{60}\) 
was computed from \(N_{60}\) and \(\sigma_{vo}^*\) using Eq. 9. For the Tokimatsu and Seed (1987) method, the
value of CRR was obtained from Seed et al. (1984, 1985) using \( (N_t)_{60} \), and then CSR was
determined using Eq. 2 and the value of \( F_\ell \).

The vertical strains computed by the three methods and compared in Fig. 8, suggest that (a) the
vertical strain versus factor of safety relationships by the three methods indicate a similar trend, (b)
with the exception of the loose sand with \( N_{60} = 5 \), the vertical strain predicted by Ishihara and
Yoshimine (1992) is the highest, by Tokimatsu and Seed (1987) is the lowest, and by the proposed
method often is somewhere in between; the predicted vertical strain by the proposed method is
lowest for \( F_\ell \) greater than 1.0 at \( \sigma_{vo}' = 50 \) kPa (c) the differences in predicted vertical strain among
the three methods become insignificant with the increase in \( N_{60} \) and \( F_\ell \), (d) the proposed method
predicts a significant influence of \( \sigma_{vo}' \) on vertical strain for loose sands (e.g. \( N_{60} = 5 \)) because \( u' \) is
directly related to \( \sigma_{vo}' \). The high values of \( \sigma_{vo}' \) may result from in situ effective overburden
pressure plus an increase in effective vertical stress by a building load. The laterally constrained
seismic compression may result either from large building breadths or sublayers at large depths.
This prediction is reasonable because loose sands that exist at high effective overburden pressures,
upon seismic shaking should be expected to experience large compression. On the other hand, the
influence of \( \sigma_{vo}' \) on vertical strain decreases as the sand becomes denser (e.g. \( N_{60} = 20 \)) because in
the absence of particle damage (Mesri and Vardhanabhuti 2009) the overburden pressure under
which a dense sand exists is not expected to significantly influence vertical strain subsequent to
seismic shaking.
Influence of Procedure to Calculate $F_\ell$

The proposed method for computing settlement subsequent to undrained seismic shaking computes $F_\ell$ using CRR in terms of $\left(N_1\right)_{60}$ and $M$, $r_d$ as a function of depth $z$, and the fines content correction for $\left(N_1\right)_{60}$ described in Terzaghi et al. (1996) and substantially adapted from Seed et al. (1984, 1985). Alternative interpretations of CRR versus $\left(N_1\right)_{60}$ and $M$, $r_d$ as a function of $z$ and $M$, and fines content correction for $\left(N_1\right)_{60}$, have been subsequently proposed (Youd et al. 2001; Cetin et al. 2004; Idriss and Boulanger 2006). These proposals are illustrated in Fig. 9 and are used to compute the seismic-induced settlements shown in Fig. 10 for the cases listed in Table 2. According to Fig. 10, the alternative procedures for calculating $F_\ell$ tend to estimate, in some cases, a somewhat higher seismic-induced settlement than by the $F_\ell$ computed according to Seed et al. (1984) as described in Terzaghi et al. (1996). For example, a higher $r_d$ computes a higher CSR and therefore, a lower $F_\ell$ and higher vertical strain. In a few cases, the higher CRR at low range of $\left(N_1\right)_{60}$, as in Fig. 9a, is expected to increase $F_\ell$ and decrease the vertical strain.

Fines Content Correction

Most of the early empirical concepts of ground response to shaking and liquefaction were developed for clean sands, generally assumed to have a fines content less than 5% (Casagrande 1948). However, the influence of content and composition of fines on dynamic response of granular soils has been recognized for a long time. The importance of the fines content in relation to settlement due to seismic shaking especially came to focus after the 1989 magnitude 7.1 Loma Prieta earthquake, and the behavior of hydraulic fills with fines content of at least 15% observed in
the Marina District (O’Rourke et al. 1991, 1992; Ishihara 1993). It was recognized that the silt content significantly decreased settlement due to seismic shaking, and for the prediction of settlement, for example using the Tokimatsu and Seed (1987), a $\left(N_1\right)_{60}$ corrected for fines content was considered (O’Rourke et al. 1992). However, in predicting settlement using Tokimatsu and Seed (1987) or Ishihara and Yoshimine (1992) the presence of silt was taken into account by adapting an adjusted thickness that excluded the thickness of thin silt and/or clay layers detected by the piezocone penetration measurements (O’Rourke et al. 1992; Ishihara 1993). Shahien (1998) and Wu (2002) computed settlement due to seismic shaking using standard penetration resistance corrected for the fines content. The correction for $\left(N_1\right)_{60}$ of silty sands recommended by Seed et al. (1985) has been used in the settlement analyses proposed here.

A unique characteristic of granular soils is that the soil particles readily separate upon vibration. The loss of interparticle contacts and increase in porewater pressure may lead to liquefaction. The introduction of plastic fines increases the resistance to separation of the sand particles. The resistance increases with the increase in the plasticity of the fines, providing adhesion at interparticle contacts. Thus, as the plasticity index of a sand with fines increases, resistance to particle separation and liquefaction increases, and seismic settlement decreases. Ishihara (1993), using data especially for tailing materials, proposed a correction factor for CRR of sands, as a function of plasticity index $I_p$. For $I_p >10\%$

\[ \mu(I_p) = 0.025 I_p + 0.75 \]

where $\mu(I_p) = s_u(I_p)/s_u(I_p = 10\%)$, and $I_p$ is in percent. As is illustrated by the following example, taking into account the plasticity of the fines leads to a significant reduction in the computed settlement.
The southern part of Rokko island in Osaka Bay was reclaimed by the sedimentary rock debris of the Kobe group that includes sandstones, mudstones, and tuff. Shibata et al. (1996) and Tokimatsu et al. (1996) have described the sandy fill for the sedimentary rock debris as having median particle size in the range of 0.035 to 10 mm, fines content in the range of 10 to 55% and clay content of the fines having high plasticity. The 7.2 magnitude Hyogoken-Nambu earthquake of January 17, 1995, produced a maximum ground surface acceleration of about 0.35g in the southern part of Rokko island (Shibata et al. 1996). Tokimatsu et al. (1996) reported subsurface conditions, including a SPT N profile obtained before the earthquake at a site in the southern part of Rokko island where the earthquake produced a ground surface settlement of about 200 mm (Yasuda et al. 1996). A settlement analysis using the proposed method together with N values, and taking into account the fines content but not the plasticity of the fines leads to ground surface settlement of 500 mm shown in Fig. 11, which is significantly larger than the observed settlement of 200 mm. Even though the clay content at the site was described having high plasticity (Tokimatsu et al. 1996), data on Atterberg limits are not available. A settlement analysis using the Ishihara (1993) correction factor in Eq. 11, and assuming plasticity index of 40% leads to a computed ground surface settlement of 259 mm, comparable with the observed settlement of 200 mm.

In summary, not only the content but also the composition of the fines, for example as expressed by the plasticity index, may have a significant effect on seismic settlement of silty-clayey sands.

**Pre- and Post-Earthquake Penetration Resistance**
The back-analyses of settlements that have been observed during earthquakes have been frequently carried out using post-earthquake in situ penetration resistance measurements. Therefore, there is a considerable interest in the relationship between penetration resistance measured after the earthquake and the resistance that determined the ground response during the earthquake. For example, Goto (1968a, b) compared SPT N values measured at 28 sites before and after the 1964 magnitude 7.5 Niigata earthquake, measuring an increase in N for N(before) values less than 20, and a decrease in N for N(before) greater than 20. Goto (1968 a, b) concluded that the sands with N(before) less than 20 contracted in response to earthquake, leading to higher N(after), whereas sands with N(before) greater than 20 dilated, leading to lower N(after). The general relationship between N(after) and N(before) is rather complex because at any depth N(before) is influenced by the pre-earthquake condition of the sand such as relative density, content and composition of fines, and age of the deposit, and N(after) is in addition affected by the intensity and duration of ground shaking, and elapsed time after earthquake when N is measured (Mesri et al. 1990; Leon et al. 2006; Mesri and Vardhanabhuti 2009). The complex relationship between N(after) and N(before) was illustrated by Shahien (1998) using data in connection to a number of earthquakes (Hayashi et al. 1966; Koizumi 1966; Watanabe 1966; Kawakami and Asada 1966; Goto 1968a, 1968b; Ohsaki 1970; Ishihara et al. 1980; Shibata et al. 1996). The majority of data assembled by Shahien (1998) may be expressed as

\[
N_{(after)} = \mu_N \times N_{(before)}
\]

where for values of N in the range of 2 and 50, \(\mu_N\) ranges between \(\frac{1}{2}\) to 2; however, \(\mu_N\) could be as small as \(\frac{1}{4}\) and as large as 4 (Shahien 1998).
The use of \( N(\text{after}) \), which may be significantly different from \( N(\text{before}) \) especially for loose young sands that are likely to be substantially modified by ground shaking, partly contributes to the difference between computed and observed settlements. An instructive example of ground behavior observed during the magnitude 7.9 Tokachi-Oki earthquake of 16 May 1968 at a site where \( N(\text{before}) \) and \( N(\text{after}) \) data are available, has been reported by Ohsaki (1970). The site, consisting of fine sand layers down to a depth of more than 20 m, is located in the city of Hachinohe, about 560 km north of Tokyo. Because the ground contains a large quantity of iron sand, part of the site was mined to a depth of about 6 m and then backfilled with waste sand. The \( N \) profiles before and after earthquake are shown in Fig. 12 at both locations without and with excavation and backfilling. The date of the excavation and backfilling was not reported, however, the loose backfill is a young fine sand deposit with a fines content of less than 10%.

After the Richter magnitude 7.9 Tokachi-Oki earthquake, that resulted in maximum ground surface acceleration of 0.23g at the site, a differential settlement of about 500 mm was observed between the natural areas and the area that had been excavated to the depth of about 6 m and backfilled with loose waste sand. The proposed method of settlement analysis together with \( N(\text{after}) \) predicts a settlement of only 57 mm for the backfilled area, whereas \( N(\text{before}) \) predicts a settlement of 462 mm. The latter computed settlement is rather close to the observed behavior because the predicted settlement for the natural ground using \( N(\text{before}) \) and \( N(\text{after}) \) is 5 and 4 mm, respectively. In other words, the computed differential settlement of 457 mm is quite comparable to the observed differential settlement of about 500 mm. The settlement calculations for the backfilled area in Fig. 12 using the \( N(\text{before}) \) and \( N(\text{after}) \) are tabulated in Appendix A to illustrate the proposed method of settlement analysis.
In summary, for certain ground conditions and earthquakes, the $N_{\text{after}}$ may not accurately characterize the pre-earthquake sand deposit. The understanding of the general relationship between $N_{\text{after}}$ and $N_{\text{before}}$ for a variety of ground conditions, earthquakes, and elapsed times post-earthquake when $N$ is measured, is of considerable interest for reliable analyses of ground settlement resulting from seismic shaking.

Settlement During Ground Shaking

Centrifuge test results (Liu and Dobry 1997; Hausler 2002; Dashti 2009; Bray and Dashti 2010; Dashti et al. 2010) on seismic settlement of buildings on liquefied saturated sands suggest that some volumetric compression may take place during ground shaking, and this type of volumetric compression is associated with $m_{vs}$ values larger than those determining post-earthquake recompression. The volumetric compression of liquefied saturated sands during ground shaking is possible for sand layers with small drainage distances, such as small $z$, or small building breadths, $B$. The centrifuge tests data suggest that seismic settlement computed assuming laterally constrained compression for the liquefied condition for sublayers at small $z$ or buildings with small $B$ may be increased by a factor of 1.4. However, these methods based on laterally constrained compression do not include settlement resulting from lateral escape of soil from under the building. Needless to mention that the 1.4 factor is not applicable to liquefied soils beneath wide foundations or liquefied soil sandwiched between stiffer layers.
Conclusions

The following conclusions are based on data, analyses and interpretation presented in this paper.

1. A new method is presented for predicting seismically-induced settlement of saturated granular level ground experiencing undrained shaking and laterally constrained compression. The vertical compression of each sublayer is directly related to the magnitude of excess porewater pressure generated by ground shaking, $u'$, and the seismic coefficient of vertical compression, $m_{vs}$, in response to the dissipation of excess porewater pressure subsequent to ground shaking.

2. Based on reported laboratory data on 18 sands, an empirical relationship is proposed between $m_{vs}$ and $N_{60}$ as a function of the factor of safety against liquefaction, $F_l$, defined as the ratio of the undrained shear strength at yield determined from CRR versus $(N_1)_{60}$ of Seed et al. (1984, 1985), to seismic shear stress computed from CSR relation of Seed and Idriss (1971) in terms of maximum ground surface acceleration.

3. The limiting range of the proposed $m_{vs}$ versus $N_{60}$ relationships is further examined using laboratory and field data for the liquefaction condition of 23 sands.

4. The ground surface seismic settlement predicted by the proposed method is compared to the settlement observed at 78 sites subjected to 7.1 and 8.0 magnitude earthquakes.
with a wide range of values of $D_{50}$, FC, N, and $\sigma'_{vo}$. There is acceptable agreement between the settlement predicted by the proposed method and the observed settlements.

5. For a large number of sites subjected to earthquakes, the settlement predicted by the proposed method and those by the methods of Tokimatsu and Seed (1987) and Ishihara and Yoshimine (1992), are comparable. Considering all the sites, in general, the proposed method predicts a settlement between the low value by Tokimatsu and Seed (1987) and the high value by Ishihara and Yoshimine (1992). The difference in the predicted vertical strain among the three methods becomes insignificant with increase in $N_{60}$ and $F_C$. The proposed method predicts a significant influence of $\sigma'_{vo}$ on vertical strain for loose sands (e.g. $N_{60} = 5$). The high values of $\sigma'_{vo}$ may result from in-situ effective overburden pressure plus an increase in effective vertical stress by a building load resulting in laterally constrained seismic compression either because of large building breadths or sublayers at large depths.

6. The influence on the predicted ground surface seismic settlement, of alternative assumptions on the relationship between CRR and $(N_{1})_{60} \cdot [CRR]_{M}/[CRR]_{7.5}$ and M, $\Delta(N_{1})_{60}$ and FC, and $r_d$ versus depth and M, is examined using the proposed method. For many cases, the alternative assumptions, together do not have a significant effect on the magnitude of predicted settlement. However, in some cases, the alternative assumptions may decrease predicted settlement by 20 to 40 mm for the total settlement in the range of 80 to 500 mm.
7. The plasticity of the fines content may have a significant effect on settlement of sands subjected to undrained seismic shaking. This factor is illustrated by settlement analyses for a site in Rokko island, Japan, together with different assumptions on plasticity of fines influencing the SPT N values. Without a correction for the plasticity of fines, a ground surface settlement of 500 mm is predicted which is significantly larger than the observed settlement of 200 mm. With a correction factor for the plasticity of fines, a ground surface settlement of 259 mm is computed.

8. The back-analyses of settlements observed during earthquakes have been frequently carried out using post-earthquake in situ penetration resistance measurements. Data on SPT penetration resistance measured before and after earthquakes suggests that in most cases, N(after)/N(before) may range between ½ and 2; however, N(after)/N(before) may be as low as ¼ and as high as 4. The influence of using N(after) as opposed to N(before) is dramatically illustrated for a young loose sand backfill at a site in Hachinohe, Japan, subjected to the magnitude 7.9 Tokachi-Oki earthquake. For a loose young sand, N(after) predicts a settlement of 57 mm, whereas N(before) predicts a settlement of 462 mm, quite comparable to the observed settlement.

9. Limited centrifuge observations on seismic settlement of buildings on saturated sand suggest that laterally constrained volumetric compression of liquefied sands that may take place during shaking may mobilize values of seismic coefficient of vertical compression as high as 1.4 times those observed subsequent to shaking.
References


Notation

- $a_{\text{max}} =$ maximum ground surface acceleration
- $B =$ breadth of building
- $\text{CRR} =$ Cyclic resistance ratio $= \frac{s_{\text{uo}}(\text{yield})}{\sigma'_{\text{vo}}}$
- $\text{CSR} =$ Cyclic stress ratio $= \frac{\tau(\text{seismic})}{\sigma'_{\text{vo}}}$
- $D_{50} =$ median grain size
- $D_r =$ relative density
- $e_{\text{max}} =$ maximum void ratio
- $e_{\text{min}} =$ minimum void ratio
- $\text{FC} =$ fines content
- $F_t =$ factor of safety against liquefaction
- $g =$ acceleration due to gravity (980 gals)
- $I_p =$ plasticity index
- $[L_o]_j =$ thickness of sublayer j
- $[m_{\text{vs}}]_j =$ seismic coefficient of vertical compression of sublayer j subsequent to undrained shaking.
- $M =$ Richter Magnitude Earthquake
- $N =$ SPT blow count
- $N(\text{after}) =$ SPT blow count determined post-earthquake
- $N(\text{before}) =$ SPT blow count determined pre-earthquake
- $N_{60} =$ blow count corresponding to energy ratio of 60%
- $(N_i)_{60} =$ normalized SPT blow count
- $N_c =$ equivalent number of uniform seismic shear stress cycles
$N_t =$ equivalent number of uniform shear stress cycles required to cause initial liquefaction

$r_d =$ depth-related seismic shear stress reduction factor

$S =$ ground surface settlement

$u'_j =$ excess porewater pressure generated by undrained shaking at sublayer $j$

$SPT =$ standard penetration test

$s_u(yield) =$ seismic shear stress required for an earthquake of magnitude $M$ to cause initial liquefaction

$z =$ depth of sublayer

$\varepsilon_v =$ vertical strain

$\mu(I_p) =$ plasticity index correction factor for undrained shear strength of granular material containing plastic fines with $I_p > 10$

$\mu_N =$ ratio between blow count determined Post- and Pre-earthquake

$\sigma_{vo} =$ in situ total overburden pressure

$[\sigma_{vo}]_j =$ in situ effective overburden pressure at mid-depth of sublayer $j$

$\sigma'_v =$ effective overburden pressure

$\tau(seismic) =$ shear stress by ground acceleration produced by an earthquake
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Fig. 1  Excess porewater pressure generation in response to ground shaking, and vertical strain in response to dissipation of excess porewater pressure for (a) loose Fuji River Sand (Interpreted from data by Nagase and Ishihara 1988 and Ishihara and Yoshimine 1992), (b) loose clean sand (Interpreted from data by Chern and Lin 1994).

Fig. 2  Relationship between $m_{vs}$, $N_{60}$ and $F_t$ based on direct simple shear test data for Fuji River Sand (Nagase and Ishihara 1988; Ishihara and Yoshimine 1992). The open circles correspond to limiting values of $m_{vs}$ as interpreted from complete set of laboratory data by Nagase and Ishihara (1988) and Ishihara and Yoshimine (1992).

Fig. 3  Empirical relationship between (a) parameter $b$ and factor of safety against liquefaction and (b) parameter $c$ and factor of safety against liquefaction.

Fig. 4  Proposed relationship between $m_{vs}$ and $N_{60}$, as a function $F_t$, based on all available data interpreted using Eq. 10.

Fig. 5  Observed $m_{vs}$ versus $N_{60}$ data for liquefaction condition observed in the laboratory and in the field together with the computed $m_{vs}$ versus $N_{60}$ relationship for $F_t = 1.0$ in Fig. 4. The Burland and Burbidge (1985) relationship from Terzaghi et al. (1996) for coefficient of vertical compression versus blowcount is shown for reference. The numbers correspond to the sites in Table 2.

Fig. 6  Ground surface settlement due to seismic shaking predicted by the proposed method compared to observed settlement. The solid points correspond to the median of the observed settlements. The dotted horizontal lines show the range of settlements at the site.

Fig. 7  Ground surface settlement by seismic shaking predicted by the proposed method compared to settlements predicted by Tokimatsu and Seed (1987) and Ishihara and Yoshimine (1992).
Fig. 8. Influence of $N_{60}$ and $\sigma'_{vo}$ on the $\varepsilon_v$ versus $F_\ell$ relationship predicted by the Ishihara and Yoshimine (1992), Tokimatsu and Seed (1987) and the proposed method.

Fig. 9. Alternative assumptions on (a) Cyclic resistance ratio versus $\left(N_1\right)_{60}$, (b) $[\text{CRR}]_M/[\text{CRR}]_{7.5}$ versus earthquake magnitude $M$, (c) $\Delta(N_1)_{60}$ versus fines content, and (d) $r_d$ versus depth for different earthquake magnitude $M$.

Fig. 10. Settlement predicted by the proposed method using alternative procedures to compute factor of safety against liquefaction.

Fig. 11. Predicted subsurface and surface settlement by the proposed method for a site in Rokko island, together with $N$ values taking or not taking into account plasticity of fines content.

Fig. 12. At a site in the city of Hachinohe subjected to Tokachi-Oki earthquake (a) vertical profiles of $N$ (before) and $N$ (after) for natural ground consisting of a fine sand with iron content, (b) vertical profiles of $N$ (before) and $N$ (after) for an area of the site mined to a depth of 6 m and refilled with loose waste sand, and (c) settlement predicted by the proposed method for the mined area using $N$ (before) and $N$ (after) values.
List of Tables

Table 1  Index properties of sands used in the laboratory tests to determine seismic coefficient of vertical compression for undrained shaking, $m_{vs}$.

Table 2  Data for sites subjected to earthquake shaking used to examine the proposed $m_{vs}$ versus $N_{60}$ relationship.
Table 1. Index properties of sands used in the laboratory tests to determine seismic coefficient of vertical compression for undrained shaking, $m_{vo}$.

<table>
<thead>
<tr>
<th>Sand</th>
<th>$D_50$ (mm)</th>
<th>Uniformity Coefficient</th>
<th>Fines Content (%)</th>
<th>$e_{max}$</th>
<th>$e_{min}$</th>
<th>Relative Density (%)</th>
<th>$\sigma_{vo}$ (kPa)</th>
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1a Air pluviated tests on dry sand
1b Air pluviated tests on saturated sand
2 Strain controlled simple shear tests
3 Uni-Multi-directional shaking
Table 2. Data for sites subjected to earthquake shaking used to examine the proposed $m_{sv}$ versus $N_{60}$ relationship. $F_i$.  

<table>
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<tr>
<th>Earthquake</th>
<th>No</th>
<th>Cases</th>
<th>References</th>
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<th>Observed settlement (mm)</th>
<th>Soil Profile including N Before/After</th>
<th>Depth of water table (m)</th>
<th>Total thickness considered for Calc. (mm)</th>
<th>$D_{sv}$ (mm)</th>
<th>FC (%)</th>
<th>$\sigma_{vo}$ (kPa)</th>
<th>$F_i$</th>
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<td>Kishida (1969) &amp; Iwasaki (1986)</td>
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<td>Iwasaki (1986)</td>
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Table 2. (Continued)

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<th>Observed settlement (mm)</th>
<th>Soil Profile including N Before/After</th>
<th>Depth of water table (m)</th>
<th>Total thickness considered for Calci. D&lt;sub&gt;50&lt;/sub&gt; (mm)</th>
<th>FC (%)</th>
<th>SPT N</th>
<th>σ'&lt;sub&gt;vo&lt;/sub&gt; (kPa)</th>
<th>F&lt;sub&gt;i&lt;/sub&gt;</th>
<th>Calculated Settlement (mm)</th>
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<td>78</td>
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<td>Martin et al. (2004)</td>
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*Estimated acceleration
1Site improved using rod-vibro compaction
2Site improved by preloading using sand drains
3Site improved using sand drains
Fig. 1. Excess porewater pressure generation in response to ground shaking, and vertical strain in response to dissipation of excess porewater pressure for (a) loose Fuji River Sand (Interpreted from data by Nagase and Ishihara 1988 and Ishihara and Yoshimine 1992), (b) loose clean sand (Interpreted from data by Chern and Lin 1994).
Fig. 2. Relationship between \( m_{vs} \), \( N_{60} \), and \( F_l \) based on direct simple shear test data for Fuji River Sand (Nagase and Ishihara 1988; Ishihara and Yoshimine 1992). The open circles correspond to limiting values of \( m_{vs} \) as interpreted from complete set of laboratory data by Nagase and Ishihara (1988) and Ishihara and Yoshimine (1992).
Fig. 3. Empirical relationship between (a) parameter \( b \) and factor of safety against liquefaction and (b) parameter \( c \) and factor of safety against liquefaction.
Fig. 4. Proposed relationship between $m_{vs}$ and $N_{60}$, as function of $F_\ell$, based on all available data interpreted using Eq. 10.
Fig. 5. Observed $m_{vs}$ versus $N_{60}$ data for liquefaction condition observed in the laboratory and in the field together with the computed $m_{vs}$ versus $N_{60}$ relationship for $F_t = 1.0$ in Fig. 4. The Burland and Burbidge (1985) relationship from Terzaghi et al. (1996) for coefficient of vertical compression versus blowcount is shown for reference. The numbers correspond to the sites in Table 2.
Fig. 6. Ground surface settlement due to seismic shaking predicted by the proposed method compared to observed settlement. The solid points correspond to the median of the observed settlements. The dotted horizontal lines show the range of settlements at the site.
Fig. 7. Ground surface settlement by seismic shaking predicted by the proposed method compared to settlements predicted by Tokimatsu and Seed (1987) and Ishihara and Yoshimine (1992).
Fig. 8. Influence of $N_{60}$ and $\sigma'_{vo}$ on the $\varepsilon_v$ versus $F_l$ relationship predicted by the Ishihara and Yoshimine (1992), Tokimatsu and Seed (1987) and the proposed method.
Fig. 9. Alternative assumptions on (a) Cyclic resistance ratio versus \( (N_1)_{60} \), (b) \( \frac{[CRR]_M}{[CRR]_{7.5}} \) versus earthquake magnitude \( M \), (c) \( \Delta(N_1)_{60} \) versus fines content, and (d) \( r_d \) versus depth for different earthquake magnitude \( M \).
Fig. 10. Settlement predicted by the proposed method using alternative procedures to compute factor of safety against liquefaction.
Fig. 11. Predicted subsurface and surface settlement by the proposed method for a site in Rokko island, together with N values taking into account plasticity of the fines.
Fig. 12. At a site in the city of Hachinohe subjected to Tokachi-Oki earthquake (a) vertical profiles of N(before) and N(after) for natural ground consisting of a fine sand with iron content, (b) vertical profiles of N(before) and N(after) for an area of the site mined to a depth of 6 m and refilled with loose waste sand, and (c) settlement predicted by the proposed method for the mined area using N(before) and N(after) values.
Appendix A

To illustrate the proposed method of settlement analysis, the settlement calculations for the backfilled area at a site in the city of Hachinohe subjected to the magnitude 7.9 Tokachi-Oki earthquake, shown in Fig. 12, performed using the N(before) and N(after), are tabulated in tables A1 and A2, respectively.

An average unit weight, $\gamma$, of 17.5 kN/m$^3$ is used to calculate the total overburden pressure, $\sigma_{vo}$ and the effective overburden pressure $\sigma'_{vo}$. An energy ratio of 0.8 is assumed to convert the reported blowcounts, N to $N_{60}$.

The Seed et al. (1984) relationships summarized in Fig. 9 are used to determine the cyclic resistance ratio, CRR, the magnitude scaling factor, $[CRR]_M/[CRR]_{7.5}$ as well as $r_d$ for each sublayer. Cyclic shear stress ratio, CSR, and the factor of safety against liquefaction, $F_\ell$, respectively, are calculated using Eqs. 3 and Eq. 2. Values of $N_{60}$, together with the $F_\ell$, are then used in Fig. 4 to determine the seismic coefficient of vertical compression, $m_{vs}$. Each sublayer settlement is then calculated using Eq. 1.
### Table A1. Settlement calculations for the backfilled area in Fig. 12 using N(before).

<table>
<thead>
<tr>
<th>Sub layer</th>
<th>Depth (m)</th>
<th>N (before)</th>
<th>N$_{60}$ (before)</th>
<th>$\sigma_{vo}$ (kPa)</th>
<th>$\sigma'_{vo}$ (kPa)</th>
<th>C$_N$</th>
<th>(N$<em>1$)$</em>{60}$</th>
<th>$r_d$</th>
<th>CRR</th>
<th>CSR</th>
<th>$F_t$</th>
<th>$[L_{so}]$ (m)</th>
<th>$m_{vs}$ (1/Mpa)</th>
<th>$S_j$ (mm)</th>
<th>$\sum S_j$ (mm)</th>
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<td>21.0</td>
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### Table A2. Settlement calculations for the backfilled area in Fig. 12 using N(after).

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<th>$\sigma'_{vo}$ (kPa)</th>
<th>C$_N$</th>
<th>(N$<em>1$)$</em>{60}$</th>
<th>$r_d$</th>
<th>CRR</th>
<th>CSR</th>
<th>$F_t$</th>
<th>$[L_{so}]$ (m)</th>
<th>$m_{vs}$ (1/Mpa)</th>
<th>$S_j$ (mm)</th>
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