Evaluation of liquefaction resistance of soils from Swedish weight sounding tests

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Abstract

A procedure to estimate the liquefaction resistance of soils by means of Swedish weight sounding tests is proposed. The empirical formulae estimating the liquefaction resistance of soils $R_l$ directly from the penetration resistance, $W_{sw}$ and $N_{sw}$, of Swedish weight sounding tests are firstly derived by combining the correlation between the values for $W_{sw}$ and $N_{sw}$ and relative density $D_r$ and the relation between liquefaction resistance $R_l$ and relative density $D_r$ obtained in past studies. As a result, two empirical formulae are derived, one for sand and the other for sand with silt and silty sand. The proposed procedure is then compared with the conventional procedure and examined in detail. The proposed procedure is found to work well for sand, sand with silt and silty sand, although due care should be taken when applying it to fine silts.

Keywords: Liquefaction resistance; Swedish weight sounding (IGC: D7/E7)

1. Introduction

Following the 2011 Great East Japan Earthquake that struck the eastern mainland of the Japanese archipelago on March 11, 2011, numerous residential houses were damaged due to settlement and tilt brought about by soil liquefaction. The houses were located mainly on reclaimed lands along Tokyo Bay. Some recent developments have been made in geotechnical investigation tools for evaluating the liquefaction susceptibility of soils in Japan, including a piezo-electric drive cone (PDC). Nevertheless, Swedish weight sounding (SWS) tests have been widely used and will still be used in Japan for geotechnical investigations of residential houses as robust and cost-effective tools. In addition, SWS tests are the investigation tools officially recommended for this purpose by the Ministry of Land, Infrastructure and Transport in Japan. Therefore, it is most important to pursue a more advanced use of SWS tests.

The procedure for SWS tests and the interpretation of the test results are described by Tsukamoto et al. (2004). The testing procedure consists of two phases, static penetration and rotational penetration. During the static penetration phase, the screw-shaped point attached to the tip of a rod weighing 49 N (5 kgf) is statically penetrated by adding several weights (10, 10, 25, 25, and 25 kgf) stepwise in increments until the total load becomes equal to 980 N (100 kgf), as shown in Fig. 1. At each load increment, the depth of the static penetration is then compared with the conventional procedure and examined in detail. The proposed procedure is found to work well for sand, sand with silt and silty sand, although due care should be taken when applying it to fine silts.

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The use of SWS tests was extensively examined in order to estimate the engineering properties of soils and various parameters associated with soil liquefaction, such as relative density, the SPT N-value, undrained shear strength, the factor of safety against liquefaction, post-liquefaction settlement and laterally spreading displacement (Tsukamoto et al., 2004, 2009a, 2009b, 2010, 2012, 2013; Tsukamoto, 2009, 2015; Tsukamoto and Ishihara, 2011). However, the estimations of the liquefaction resistance in those previous studies required some soil properties, such as the range in void ratio, \( e_{\text{max}} - e_{\text{min}} \), and the mean particle diameter, \( D_{\text{50}} \). Moreover, it was necessary to go through a complex sequential procedure involving the conversion of \( W_{\text{sw}} \) and \( N_{\text{sw}} \)-values to SPT N-values followed by the conversion of SPT N-values to the liquefaction resistance. In the present study, a more robust procedure is proposed to estimate the liquefaction resistance of soils by means of SWS tests.

### 2. Derivation of proposed procedure

The penetration resistance \( N_{\text{sw}} \) of SWS tests is known to be most largely dependent upon the effective confining stress, soil grading, soil density, etc. The results of laboratory calibration chamber tests on SWS tests are shown in Fig. 2 (Tsukamoto et al., 2009a). Herein, the values for \( N'_{\text{sw1}} \) are defined by taking into account the effects of the static penetration and the effective confining stress, as follows:

\[
N'_{\text{sw1}} = (N_{\text{sw}} + 40 \times W_{\text{sw}}) \sqrt{\frac{98}{\sigma'_{\text{v}}}} = N_{\text{sw}} \sqrt{\frac{98}{\sigma'_{\text{v}}}}
\]

where \( W_{\text{sw}} \) is in kN and \( \sigma'_{\text{v}} \) is in kPa. In the static penetration phase, it yields to \( N'_{\text{sw1}} = 40 \times W_{\text{sw}} \) [kN], since \( N_{\text{sw}} = 0 \). On the other hand, in the rotational penetration phase, it yields to \( N'_{\text{sw}} = N_{\text{sw}} + 40 \), since \( W_{\text{sw}} = 1 \) [kN]. It was demonstrated by Tsukamoto et al. (2009a, 2009b) and Tsukamoto (2009) that the linear relation of \( N'_{\text{sw1}} = \alpha_{\text{sw}} D_{\text{r}}^2 \) would hold for fine clean
sand, although the relations for sand with silt and silty sand would be more suitably represented with the offset in the relative density, \( D_{ro} \), as follows:

\[
N'_{sw} = \alpha_{sw}(D_r - D_{ro})^2
\]

where \( D_r \) and \( D_{ro} \) are defined to take values ranging from 0 to 1 and are not percentages. The value for \( \alpha_{sw} \) changes with the inclusion of silt. The data given in Fig. 2 clearly show the strong dependency of the \( N'_{sw} \) values on soil grading, which is represented by the value for \( \alpha_{sw} \). Based on the data shown in Fig. 2, the offset is found to be equal to \( D_{ro} = 25\% \). The values for \( \alpha_{sw} \), inferred from Fig. 2, are summarised in Table 1. The presence of the offset in relative density, exclusively for sand with silt and silty sand, might indicate that there could be a range in void ratios that compressible silt-containing sand would not be able to attain under the presence of confining stress.

In Fig. 3, the data for liquefaction resistance, \( R_l = \sigma_d/(2\sigma'_o) \), collected from many sources of past studies, are plotted against relative density \( D_r \) (Nakazawa 2007). Herein, the data shown in Fig. 3 are liquefaction resistance \( R_l \) obtained from isotropically consolidated undrained cyclic triaxial tests on saturated soil specimens, \( \sigma'_o \) the effective confining stress, and \( \sigma_d \) the single amplitude of cyclic stress used to achieve the double amplitude axial strain of \( DA_{sl} = 5\% \) in 20 cycles. Therefore, any other effects, including earthquake magnitude, multi-directional shaking and confining stress, are out of the scope of the present study. These data include reconstituted samples as well as undisturbed samples, and range from sand to silt. It is well known that the liquefaction resistance increases almost linearly with the relative density up to \( D_r = 80\% \). However, the resistance tends to increase rapidly when the relative density exceeds \( D_r = 80\% \) (Ishihara 1973; Tatsuoka et al., 1982; Tokimatsu and Yoshimi 1983). Focusing on the range in data up to \( D_r = 80\% \), shown in Fig. 3(a), the empirical relation of \( R_l = 0.42 \times D_r \) might be appropriately assumed for clean sand (Ishihara 1973). On the other hand, it would be still controversial to describe the tendency of liquefaction resistance by changing the inclusion of silt (Papadopoulou and Tika 2008; Dash et al., 2010; others). From the viewpoint of better characterising the soils of sand and silt mixtures, the use of usual definitions for void ratio and relative density may not be adequate, and the concept of a skeleton void ratio, and similar concepts, have recently been introduced and examined (Thevanayagam 1998; Thevanayagam and Mohan 2000; Thevanayagam et al., 2002; Yang et al., 2015; and others). However, the applicability of such concepts to a characterisation of the liquefaction resistance of sand and silt mixtures needs to be examined in further detail. Nevertheless, although the data shown in Fig. 3(b) are quite limited, they clearly indicate that the ratios of liquefaction resistance to relative density, \( R_l/D_r \), would tend to decrease with the increase in silt. Following the linear relation of \( R_l = 0.42 \times D_r \) for clean sand with relative density up to \( D_r = 80\% \), as described above, it would similarly be appropriate to assume linear relations for silt-containing sand with relative density up to \( D_r = 80\% \). In addition, it is worthwhile to see if the presence of the offset in relative density would also fit into the relations between \( R_l \) and \( D_r \), as shown in Fig. 3(b). It is seen that the following relations can be assumed for sand with silt, silty sand and silt.

\[
R_l = \beta_{\text{ext}}(D_r - D_{ro})
\]

where the value for \( \beta_{\text{ext}} \) again changes with the inclusion of silt. Similar to the strong dependency of the penetration resistance of SWS tests on soil grading, shown in Fig. 2, the liquefaction resistance is also found to exhibit a clear dependency on soil grading, as shown in Fig. 3, which is represented by the value for \( \beta_{\text{ext}} \). In producing a more robust empirical relation between the liquefaction resistance and the penetration resistance in the

### Table 1

<table>
<thead>
<tr>
<th>Soil Type</th>
<th>( D_{50} ) (mm)</th>
<th>( F_c ) (%)</th>
<th>( e_{max} - e_{min} )</th>
<th>( \alpha_{sw} )</th>
<th>( D_{ro} ) (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Toyoura sand</td>
<td>0.17</td>
<td>0</td>
<td>0.366</td>
<td>690</td>
<td>0</td>
</tr>
<tr>
<td>Image sand</td>
<td>0.15</td>
<td>23.8</td>
<td>0.512</td>
<td>890</td>
<td>25</td>
</tr>
<tr>
<td>Omigawa sand</td>
<td>0.15</td>
<td>10.7</td>
<td>0.611</td>
<td>470</td>
<td>25</td>
</tr>
<tr>
<td>Narita sand</td>
<td>0.13</td>
<td>29.1</td>
<td>0.765</td>
<td>235</td>
<td>25</td>
</tr>
</tbody>
</table>

Fig. 3. Relations on plots for \( R_l \) against relative density \( D_r \) (a) sand and (b) sand with silt, silty sand & silt (data collected by Nakazawa 2007).
SWS tests, it is in fact a primary issue whether these two effects would compromise each other, as described in what follows.

Parameters $e_{sw}$ and $D_0$, in Eqs. (2) and (3), are introduced to take into account the effects of soil grading. There are some useful soil properties that would be suited for classifying the effects of soil grading, such as fines content $F_c$ and mean particle diameter $D_{50}$. The range in void ratio, $e_{max}-e_{min}$, is also one of the soil properties that can represent the effects of soil grading. This was proposed by Cubrinovski and Ishihara (2004) and was illustrated to work fine for analysing the penetration resistance of in-situ tests (Cubrinovski and Ishihara 1999; Tsukamoto et al., 2004). Herein, based on the data shown in Fig. 2, the ratios of $N'_{sw1}/D_t^2$ for clean sand and $N'_{sw1}/(D_t-0.25)^2$ for sand with silt and silty sand are plotted against the range in void ratio, $e_{max}-e_{min}$, in Fig. 4. The data for sand with silt and silty sand are then fitted with the following equation:

$$\frac{N'_{sw1}}{(D_t-0.25)^2} = \frac{96}{(e_{max} - e_{min})^{1.2}}$$  

(4)

With the same principle in mind, the ratios of $R_l/D_t$ for clean sand and $R_l/(D_t-0.25)$ for sand with silt and silty sand, shown in Fig. 3, are plotted against the range in void ratio, $e_{max}-e_{min}$, shown in Fig. 5. The data for sand with silt and silty sand are then fitted with the following equation:

$$\frac{R_l}{D_t-0.25} = \frac{0.2}{(e_{max} - e_{min})^{1.6}}$$  

(5)

From the data and the empirical equations fitted in Figs. 4 and 5, the correlations between $R_l$ and $N'_{sw1}$ can be derived for clean sand and for sand with silt and silty sand, as follows:

$$R_l = 0.016 \sqrt{N'_{sw1}} \text{ (for clean sand of } R_l \leq 0.3)$$  

(6)

$$R_l = 0.02 \sqrt{N'_{sw1}} \text{ (for clean sand with silt and silty sand of } R_l \leq 0.3)$$  

(7)

It is noteworthy that soil parameter $e_{max}-e_{min}$, representing the effects of soil grading, was removed and is no longer present in Eqs. (6) and (7), although the two empirical equations were derived as Eqs. (6) and (7) for clean sand and for sand with silt and silty sand, respectively. It should be noted here that Eq. (4) was developed for silty sand with a fines content $F_c$ of less than 30%. In addition, based on the plots of data correlating $F_c$ and $e_{max}-e_{min}$ produced by Cubrinovski and Ishihara (2004), there is a strong tendency for $e_{max}-e_{min}$ to increase with $F_c$ up to $F_c=30\%$ (corresponding to $e_{max}-e_{min}=0.7$), above which the change in $e_{max}-e_{min}$ with an increasing $F_c$ becomes smaller. Therefore, a close look at the data shown in Fig. 5 suggests that the use of Eq. (5) requires some caution and should be constrained to silt and silty sand. After all, the use of Eq. (7) should be constrained to sand with silt and silty sand.

3. Verification of proposed procedure

The most reliable scheme for verifying the proposed procedure is to undergo comparisons with observed field behaviour. This requires an assertion of whether soil liquefaction truly occurred or not as well as a reliable estimate of the amplitude of cyclic shear stress at a particular site during a particular earthquake event. This is entirely beyond the authors’ ability; and thus, the proposed procedure is instead verified by making a comparison with the conventional procedure used by the authors.

In the conventional procedure, the evaluation of the liquefaction resistance of soils has relied entirely upon the SPT N-values, as indicated in Fig. 6. Therefore, in order to make use of SWS tests, the penetration resistance of SWS tests, $W_{sw}$ and $N_{sw}$, is firstly converted to SPT N-values. The empirical formulae proposed by Inada (1960) are most frequently used, although the following empirical formula has also been proposed by Tsukamoto et al. (2004) and is used in the
The use of Eq. (8) is advantageous for taking account of the effects of soil grading in a manner relevant to the present study. It then follows that the liquefaction resistance of soils would be estimated from SPT $N$-values, based on empirical formulae, such as those proposed by Tatsuoka et al. (1980), as follows:

$$R_i = \frac{\sigma_{d1}}{2\sigma_0} = 0.0676\sqrt{N_1} + \Delta R_0(D_{50})$$

where $\Delta R_0(D_{50}) = 0.225 \times \log_{10}(0.35/D_{50})$ and $D_{50}$ is in mm. It was noted that Eq. (9) is valid for soils having $D_{50}$ ranging from 0.04 to 0.6 mm. It is not practical or convenient to use the two parameters, namely, $\epsilon_{\text{max}}-\epsilon_{\text{min}}$ in Eq. (8) and $D_{50}$ in Eq. (9). Herein, the values for $\epsilon_{\text{max}}-\epsilon_{\text{min}}$ can be converted to $D_{50}$ by adopting the correlation of $\epsilon_{\text{max}}-\epsilon_{\text{min}} = 0.23 + 0.06/D_{50}$ proposed by Cubrinovski. The empirical formula of Eq. (9) is one of the most useful equations; it satisfactorily incorporates the effects of soil grading. However, when it comes to the use of SWS tests without soil sampling, the use of Eqs. (6) and (7) is preferable due to their insensitivity to soil grading.

In order to verify the procedure proposed in the present study, a comparison with the conventional procedure is conducted by using some case history studies. A case history study in Itako City is newly introduced, and other case history studies described in past studies are used in what follows.

### 3.1. Case history study in Itako City

Itako City is located at the downstream area of the Tone River, as shown in Fig. 7, and is one of the areas where extensive soil liquefaction was observed during the main shock and aftershocks of the 2011 Great East Japan Earthquake (Tsukamoto et al., 2012). Following the extensive damage to infrastructures, as well as to residential houses, incurred due to soil liquefaction, Itako City initiated field geotechnical investigations, in which a trench survey was carried out at a couple of sites. A portion of the results of the geotechnical investigations has been described by Tsukamoto et al. (2015).

The two trenches, herein called A and B, are located relatively close to JR Itako Station. The history of land use certainly affects the layering of the soil deposits. In this area, paddy fields prevailed over long periods of times, in which networks of small water channels were formed for agricultural purposes, such as irrigation and transport. This area was then reclaimed, upon which residential houses and infrastructures were built. In fact, one of the major purposes of the trench survey was to see if any soil materials reclaimed in such old small water channels were responsible for the extensive soil liquefaction observed around this area. In trench A, some remains of old water channels were discovered, as shown in Fig. 8. It was found that those old water channels were reclaimed with clayey soils, at least in this area, and are less responsible for the soil liquefaction. Instead, some traces of soil liquefaction were observed in the underlying stratum of fine sand, which is a grey-coloured natural deposit, as shown in Fig. 8. A close look at trench B revealed that the feeder dikes for the sand boils were very thin, as indicated by red arrows in Fig. 9, and originated in the underlying natural sand deposits. Samples of natural sand deposits, that had liquefied, were retrieved from the trenches, and the grain size distributions are shown in Fig. 10. Some physical properties relevant to the present study are also shown in Table 2. A series of Swedish weight sounding tests were carried out at two locations near trench A, Sws-1 and Sws-2, and at two locations near trench B, Sws-3 and Sws-4.

By using the data of $W_{sw}$ and $N_{sw}$, corresponding to the soil layers that experienced liquefaction, the values for liquefaction resistance $R_i$ are calculated from the proposed procedure using Eq. (6) or (7), depending upon the values of fines content $F_c$. Herein, when the value for $F_c$ is lower than about 5%, the soil is classified as sand and Eq. (6) is used. On the other hand, when the value for $F_c$ is in the range of about 5–15%, the soil is classified as sand with silt and Eq. (7) is used. When the value for $F_c$ is higher than about 15%, the soil is classified as silty sand and Eq. (7) is used. The values for liquefaction resistance $R_i$ are also calculated from the conventional...
procedure using Eqs. (8) and (9); they are plotted against relative density $D_r$ in Fig. 11. The white points indicate the data from the proposed procedure, while the dark points indicate the data from the conventional procedure. The values for relative density $D_r$ are calculated from the following empirical formula proposed by Tsukamoto et al. (2004):

$$D_r = \sqrt{\frac{(N_{sw} + 40)\left(\epsilon_{max} - \epsilon_{min}\right)^2}{90}} \left(\frac{98}{\sigma_v}\right)^{0.22}$$

where the values for $\epsilon_{max}$-$\epsilon_{min}$ can also be estimated from the correlation of $\epsilon_{max}$-$\epsilon_{min}$ = $0.23 + 0.06/D_{50}$, proposed by Cubrinovski and Ishihara (2004). The proposed procedure is found to give slightly lower estimates for $R_l$ than the conventional procedure for the data of Sws-1 and Sws-2 near trench A. On the other hand, the proposed procedure is found to give slightly higher estimates for $R_l$ than the conventional procedure for the data of Sws-3 and Sws-4 near trench B. A direct comparison of the data for $R_l$, calculated from the proposed procedure and the conventional procedure, is shown in Fig. 12. The data are linearly fitted with dashed lines in this figure.

3.2. Other case history studies from past field investigations

The authors have been continuing to use SWS tests for earthquake reconnaissance investigations, and some of the past field investigations are used as case history studies in the present study, as shown in Table 2. In this table, some of the soil properties pertinent to the present study and the types of soil samples are also shown.

The sites of reclaimed deposits in Itako City, Katori City and Hitachinaka City experienced soil liquefaction during the 2011 Great East Japan Earthquake (Tsukamoto et al., 2012, 2013, 2015). The site of reclaimed deposits in Sakai-minato...
City experienced soil liquefaction during the 2000 Tottori-ken Seibu Earthquake (Tsukamoto and Ishihara, 2010). The soil sample retrieved from this case history study contained silt of as much as $F_c = 98\%$; and thus, the use of Eq. (7) is not relevant. However, this case history study is also included for comparison purposes. The site in Kashiwazaki Regional Sewage Centre experienced soil liquefaction during the 2007 Niigata-ken Chuetsu-oki Earthquake (Tsukamoto and Ishihara, 2010). The site of an alluvial sand deposit in Karatsu City in Saga Prefecture in Kyushu has not experienced soil liquefaction during the recent earthquakes. However, some field tests were conducted for ultra microfine cement permeation grouting together with SWS tests (Hashimoto et al., 2015).

In Figs. 13 and 14, comparisons of $R_l$ calculated from the proposed and the conventional procedures are shown for the sites in Katori City and Hitachinaka City. In these diagrams and those that follow, the highest and lowest ratios of $R_l$, calculated from the proposed procedure to those from the conventional procedure, are indicated with dashed lines. The proposed procedure appears to give lower estimates for $R_l$ than the conventional procedure for the sites in Katori City, shown in Fig. 13, where the fines content $F_c$ is as high as about 20%. On the other hand, the proposed procedure appears to give reasonable estimates for $R_l$ similar to the conventional procedure for the sites in Hitachinaka City, shown in Fig. 14, where the fines content $F_c$ is 16%. Some discussions will be given later on the differences in the estimates.

In Fig. 15, the values for $R_l$, calculated from the proposed and conventional procedures, are plotted against $D_r$ for the site of the Takenouchi Industrial Complex in Sakai-minato City. This site corresponds to the reclaimed deposits of seafloor sludge, containing silt up to as much as $F_c = 98\%$. The data for the liquefaction resistance, $R_l = \sigma_d/(2\sigma_o)$, are also plotted in Fig. 15. They were obtained from saturated isotropically consolidated undrained cyclic triaxial tests conducted on reconstituted samples (Imamura et al., 2004) and on undisturbed samples (Nakazawa et al., 2004). Some of the earlier studies suggested that the liquefaction resistance of undisturbed samples of silty sands and silt is larger than that of laboratory reconstituted samples (Huang and Huang 2007, and

### Table 2

<table>
<thead>
<tr>
<th>Site</th>
<th>$D_{50}$ (mm)</th>
<th>$F_c$ (%)</th>
<th>$e_{max}$</th>
<th>$e_{min}$</th>
<th>Soil sample</th>
<th>Eq. fitted</th>
<th>Reference</th>
</tr>
</thead>
<tbody>
<tr>
<td>Trench A, Itako city</td>
<td>0.33</td>
<td>1.4</td>
<td>1.11</td>
<td>0.72</td>
<td>Trench sample</td>
<td>Eq. (6)</td>
<td>Tsukamoto et al. (2015)</td>
</tr>
<tr>
<td>Trench B, Itako city</td>
<td>0.31</td>
<td>12.7</td>
<td>1.58</td>
<td>0.94</td>
<td>Trench sample</td>
<td>Eq. (7)</td>
<td></td>
</tr>
<tr>
<td>Tonegawa river, Katori city</td>
<td>0.13</td>
<td>19.6</td>
<td>1.34</td>
<td>0.82</td>
<td>Sand boil sample</td>
<td>Eq. (7)</td>
<td>Tsukamoto et al. (2012)</td>
</tr>
<tr>
<td>Jukken-gawa river, Katori city</td>
<td>0.14</td>
<td>22.5</td>
<td>1.74</td>
<td>1.01</td>
<td>Sand boil sample</td>
<td>Eq. (7)</td>
<td>Tsukamoto et al. (2012)</td>
</tr>
<tr>
<td>Nakaminato, Hitachinaka city</td>
<td>0.21</td>
<td>16.0</td>
<td>0.926</td>
<td>0.532</td>
<td>Sand boil sample</td>
<td>Eq. (7)</td>
<td>Tsukamoto et al. (2013)</td>
</tr>
<tr>
<td>Takenouchi Industrial Complex, Sakai-minato city</td>
<td>0.045</td>
<td>98</td>
<td>1.586</td>
<td>0.858</td>
<td>Sand boil sample</td>
<td>Eq. (7)</td>
<td>Tsukamoto and Ishihara (2010, 2011)</td>
</tr>
<tr>
<td>Kashiwazaki Regional Sewage Centre</td>
<td>0.30</td>
<td>12.6</td>
<td>1.057</td>
<td>0.626</td>
<td>Sand boil sample</td>
<td>Eq. (7)</td>
<td>Tsukamoto and Ishihara (2010)</td>
</tr>
<tr>
<td>Karatsu city</td>
<td>0.40</td>
<td>6.0</td>
<td>–</td>
<td>–</td>
<td>SPT sample</td>
<td>Eq. (6)</td>
<td>Hashimoto et al. (2015)</td>
</tr>
</tbody>
</table>

Fig. 11. Plots of liquefaction resistance $R_l$ estimated from proposed procedure and conventional procedure against relative density $D_r$, (a) Trench A and (b) Trench B in Itako City.

Fig. 12. Plots of liquefaction resistance $R_l$ estimated from proposed procedure against conventional procedure (Itako City).
However, the data for the liquefaction resistance of the undisturbed samples, shown in Fig. 15, are found to be even lower than those from the proposed procedure. Some discussions will be given later on the discrepancies in the estimates.

The comparisons of $R_l$, calculated from the proposed and conventional procedures, are shown in Fig. 16.

In Figs. 17 and 18, comparisons of $R_l$, calculated from the proposed and the conventional procedures, are shown for the sites at the Kashiwazaki Regional Sewage Centre and in Karatsu City.
3.3. Some implications and limitations of proposed procedure

Based on the comparisons of $R_l$, calculated from the proposed and the conventional procedures and shown in Figs. 12–18, the ranges in ratios for $R_l$ calculated from the proposed procedure to those from the conventional procedure, can be deduced. In Fig. 19, the values for $R_l$(proposed)/$R_l$(conventional) are then plotted against fines content $F_c$. The values for $R_l$(proposed)/$R_l$(conventional) are found to stay around 1, up to a fines content $F_c$ of 15%, although they tend to decrease with an increasing fines content $F_c$ above $F_c=15%$.

From the data shown in Figs. 12–18, it is also possible to examine the differences in the estimates of the proposed and the conventional procedures by calculating the values for $R_l$(proposed)/$R_l$(conventional) are plotted against mean particle diameter $D_{50}$. Large discrepancies are found to arise at lower values for $D_{50}$, corresponding to a fines content $F_c$ of over 20%. In order to examine such discrepancies, it would be worthwhile to see that the empirical formula of Eq. (9), for the conventional procedure, is comprised of two terms. The second term of $\Delta R_l(D_{50})$ is not dependent upon the SPT $N$-values; it is solely dependent upon mean particle diameter $D_{50}$. In Fig. 20, the values for $\Delta R_l(D_{50})$ are also plotted against $D_{50}$. It is found that the discrepancies in the estimates of $R_l$ from the proposed and the conventional procedures come primarily from the second term of $\Delta R_l(D_{50})$. Some of the earlier studies also suggested that the liquefaction resistance of undisturbed samples of silty sands and silt is larger than that of laboratory reconstituted samples (Huang and Huang 2007; and others), implying the presence of ageing effects. The effects of ageing are not straightforward, although the authors estimate that the presence of fines in silty sand and silt would foster developments of ageing within soil fabric structures, compared to coarse soils, and that silty sand and silt would have greater potential for developing ageing effects. However, no silty sand or silt would necessarily develop ageing within the soil fabric structures. Hence, it is not clear if the conventional procedure is truly applicable even to loose silt with lower values for $W_{sw}$ and $N_{sw}$, which are addressed in the present study. In any case, due care should be taken when applying the proposed procedure to fine silt.

Fig. 21 shows the relations of liquefaction resistance $R_l$ and $N_{sw1}'$, which are derived from Eqs. (6) and (7) and are proposed...
in the present study. It should be noted here that the use of the relations shown in Fig. 21 is restricted to satisfy \( R_l \leq 0.3 \), since these relations are expected to be curved inwards in the range of \( R_l \) larger than 0.3 to 0.4. The presence of the two relations of \( R_l \) and \( N_{sw1} \) for sand and for sand with silt would imply that the soils of concern need to be classified at least as sand or silt-containing sand, although SWS tests would not currently accompany any soil sampling procedures. Hence, it is desirable that soil sampling geotechnology be developed using boreholes of SWS tests. Otherwise, the lack of soil sampling in SWS tests might be overcome by acquiring borehole data from standard penetration tests (SPTs) conducted nearby.

The depth-wise distributions of critical \( N_{sw} \)-values, separating the possibility of soil liquefaction into small or large, can also be derived from the present study, as shown in Fig. 22. The critical \( N_{sw} \)-values derived from the present study are found to be larger than those conventionally found in the literature.

4. Conclusions

An empirical procedure was proposed to estimate the liquefaction resistance \( R_l \) of soils by means of Swedish weight sounding tests. Two empirical formulae, Eqs. (6) and (7), were firstly derived to estimate \( R_l \) directly from the penetration resistance, \( W_{sw} \) and \( N_{sw} \), of Swedish weight sounding tests, namely, one for clean sand and the other for sand with silt and silty sand. The proposed procedure was found to work well, although a comparison of the proposed procedure with the conventional procedure, which is based on SPT N-values and \( D_{50} \), revealed that discrepancies arise when the liquefaction resistance of fine silt is estimated. Therefore, due care should be taken when applying it to fine silt.

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