STUDIES ON SOIL LIQUEFACTION AND SETTLEMENT IN THE URAYASU DISTRICT USING EFFECTIVE STRESS ANALYSES FOR THE 2011 OFF THE PACIFIC COAST OF TOHOKU EARTHQUAKE

Kiyoshi FUKUTAKE\(^1\) and Jiho JANG\(^2\)

\(^1\)Member of JSCE, Institute of Technology, SHIMIZU Corporation
(Etchujima3-4-17, Koutou-ku, Tokyo, 135-8530, Japan)
E-mail: kiyoshi.fukutake@shimz.co.jp

\(^2\)Material & Energy Research Team, GS E&C Research Institute
(Cheoin-gu, Youngin-si, Gyeonggi-do, 449-831, Korea)
E-mail: jhjang03@gsconst.co.kr

The 2011 off the Pacific coast of Tohoku Earthquake caused soil liquefaction over a wide area. In particular, severe soil liquefaction was reported in the northern parts of the reclaimed lands around Tokyo Bay, even though the seismic intensity in this area was only about 5 on the JMA scale with low acceleration. The authors surveyed the residual settlement in the Urayasu district and then conducted effective stress analyses of sites affected and not affected by liquefaction. The analyses results were compared with the acceleration waves monitored with K-NET Urayasu or ground settlements surveyed. They were based on the acceleration observed on the seismic bedrocks in earthquake engineering in some other districts adjacent to Urayasu. Much of the settlement was due to the long duration of the earthquake, with further settlement resulting from the aftershocks. The study shows that the effects of aftershocks need to be monitored. The simplified liquefaction prediction methods using the factor of safety, \(F_L\), also need improvement.

**Key Words:** The 2011 off the Pacific coast of Tohoku Earthquake, liquefaction, settlement, effective stress analysis, aftershock

1. INTRODUCTION

The 2011 off the Pacific coast of Tohoku Earthquake (hereafter, Earthquake 3.11) caused liquefaction over a wide area. Focusing on the reclaimed land in the northern part of Tokyo Bay, the seismic intensity was about 5 (ground acceleration of about 100 to 150 Gal), thus the accelerations were not very high, but the duration of the seismic motions was long, and this caused liquefaction and settlement seen in many areas. On the other hand, even though the ground conditions were similar, some places did not undergo large settlements. As a result, the Japanese Geotechnical Society and the Ministry of Land, Infrastructure, Transport and Tourism Kanto Development Bureau jointly summarized the liquefaction and non-liquefaction areas on a map as shown in Fig.1.\(^1\) However, indices directly related to damage, such as settlement, etc., were not shown on the maps. Aerial laser surveying was carried out over Urayasu City before and after the earthquake, and the change in the ground surface was indicated by the difference in the elevation values.\(^2\) The errors, however, were large and the values did not necessarily correspond to the actual amount of settlement. Konagai et al.\(^2\) conducted a LiDAR survey and drew subsidence maps of the Tokyo Bay shore areas, and the measured results contained the effects of the structures.

The authors measured the amount of settlement in Urayasu, Shin Kiba, and Tatsumi a few days after the occurrence of the earthquake, and prepared drawings of the distribution of the settlement. They then investigated the causes of the variation in the distribution of the settlement, the amount of the settlement, and the difference in settlement.

In addition, focusing on the Urayasu area, typical ground models were prepared in each area from the \(N\)-values and borehole data of ground that liquefied...
and ground that did not liquefy. Response analysis by effective stress analysis was carried out based on the acceleration records measured at the engineering bedrock near these areas, including the main shock \((M = 9.0)\) and the largest aftershock \((M = 7.7)\), which occurred 29 minutes later.

The results were compared with the K-NET Urayasu acceleration records and the amount of settlement measured at the locations of liquefaction. In addition, an improvement to the \(F_l\) method of determining liquefaction in cases where the duration is long as in multi-segment earthquakes was proposed.

2. CHARACTERISTICS OF SETTLEMENT IN RECLAIMED GROUND IN THE NORTHERN PORTION OF TOKYO BAY

(1) Characteristics of settlement distribution

Figures 2 and 3 show the amount of horizontal ground surface settlement in the Tatsumi - Shin Kiba - Urayasu area located in the northern part of Tokyo Bay shown in Fig.1 within a few days after the earthquake. These values were measured using bearing pile structure as a benchmark. The numerical values of the colors are: blue, 0 to 10 cm; orange, 10 to 30 cm; and red, 30 cm or more. Measurement of the amount of settlement was carried out using points that had not moved, such as pile-supported buildings where no settlement occurred. Settlement of the whole area did not occur, but places where the settlement was large and places where there was almost no settlement were mixed.

Figure 4 shows the geological section along the survey line and the improved areas by preloading and sand drain\(^{17}\).
Figure 5 shows the distribution of settlement along the survey line indicated in Fig.3. Plots and arrows mean measured values and their ranges. Hatching zone covers all measured values. With the Tokyo Metro Tozai Line as the origin, there is almost no evidence of settlement up to 2000 m. However, according to local taxi drivers, the noise levels are greater when they drive now compared with before the earthquake; likewise, the vibrations seen to be greater, thus it is considered that a certain amount of settlement had occurred. From about

![Map showing settlement distribution](image)

**Fig.3** Settlement in the Urayasu area (cm).

![Geological section](image)

**Fig.4** Geological section along the survey line in Fig.3 and the improved areas by preloading and sand drain^{17).}
2400 m, the settlement becomes larger, and is largest at around 4000 m, reaching a maximum of about 50 cm. From that point until the sea wall, the settlement becomes smaller, thus overall, the distribution of settlement is shaped like a spoon. The settlement becomes smaller towards the sea wall because the ground has been compacted as a mitigation measure against liquefaction from about 4500 m to near the sea wall (near point A in Fig. 3). In addition, the construction methods taken to promote consolidation (sand drains + preloading) are considered to have a certain effect in reducing liquefaction near the sea wall.17) The effect of construction methods taken to promote consolidation on reducing liquefaction has also been reported at Port Island and Rokko Island after the 1995 Kobe earthquake.3), 4) If this type of mitigation measure had not been taken, then the amount of settlement in the area reclaimed between 1975 and 1978 would have appeared to be a bit larger compared with the area reclaimed between 1968 to 1971. However, the correlation between the amount of settlement and the year of reclamation is not very clear.

As can be seen from the settlement distribution in Figs. 2, 3 and 5, there is large variation in the amount of settlement. The amount of settlement varies considerably with 100 m separation, and there is large variation even within narrow areas. This variation in the amount of settlement is due to whether the ground has been improved or not as described earlier, and differences in the surface layer soils and thickness of the reclaimed ground. If the surface layer is a thick clay layer, the settlements tend to be small; for example, near point B in Fig. 3, the surface clay layer is thick, thus the amount of settlement is comparatively small. In addition, small differences in the shear stress that acted many times are considered to have had a major effect on the resulting phenomenon. In other words, once liquefaction has occurred, the soil thereafter has been subjected to many repetitions resulting in severe liquefaction. The next section considers the cause of this phenomenon.

(2) Liquefaction characteristics during cyclic small amplitude shearing at the element level
The characteristics of liquefaction in this case were caused by comparatively small accelerations that had continued for a long period of time, and as a result, small shear stresses were repeated a large number of times.
Therefore, the reason why the small differences in the acting shear stress could result in an extreme difference in the phenomenon was investigated from the behavior of soil elements.

A calculation was carried out for cyclic shearing with small shear stress amplitudes that were just sufficient to cause liquefaction. Figure 6 shows the calculation results for the liquefaction strength curve (relationship between the shear stress ratio when the double amplitude shear strain $\gamma_{DA}$ reaches 5% and the number of cycles) of loose sand and silty sand that contains much fines. The constitutive equation used in the calculation was the Ramberg-Osgood model expanded to three-dimensions with a multiple shear mechanism as the stress-strain relationship\(^5\), and the bowl model used as the strain-dilatancy relation-ship\(^6\),\(^7\). The difference between sand and silty sand is expressed by swelling index $C_s$ in the calculation (sand: $C_s/ (1+e_0)$=0.006, silty sand: $C_s/ (1+e_0)$= 0.010).

$X_l$ is the lower limit of shear stress ratio (lower bound value of the liquefaction strength)\(^6\) that will not cause liquefaction even with multiple repetitions. The figure shows the case where $X_l = 0.1$. If the acting shear stress ratio is $X_l$ or less, liquefaction does not occur, but if it is greater than $X_l$, liquefaction will eventually occur due to multiple repetitions. In conventional design, the liquefaction strength $R_{15}$ or $R_{20}$ under 15 or 20 cycles, respectively, is used (the value near A in Fig.6). In the Earthquake 3.11, a liquefaction strength (for example, $R_{100}$, in which liquefaction is reached after 100 cycles, etc.) should be used, such as near B. In the case of sand, $X_l$ is approached with fewer cycles compared with silty sand, thus this is still more reason to take this approach. Incidentally, the following relationship is shown in Fig.6.

$$X_l \approx R_{100} \approx 0.8 \times R_{20}$$

Figure 7 shows a stress-strain relationship close to $X_l$ (for a case where liquefaction just barely occurs). In the element calculation, the shear stress ratio $X_l$ was set to 0.11, which is slightly larger than $X_l$. In the case of sand, it required 62 cycles for the strain to start to suddenly increase, but once liquefaction occurred the strain accumulated with few cycles. This corresponds to severe liquefaction even though the acting shear stress is small, as in the Earthquake 3.11. In the silty sand, the number of cycles until the increase in strain is reached is greater, and the rate of increase in the strain after liquefaction is slower compared with sand. Therefore, if the acting shear stress ratio is slightly larger than $X_l$, there is a possibility of liquefaction occurring accompanied with large strain, but if the shear strain ratio is smaller than $X_l$, liquefaction does not occur. In other words, the subsequent behavior greatly differs depending on whether the acting shear stress ratio is greater than or less than $X_l$. This is considered one reason why the locations where settlement occurred and the locations where settlement did not occur were mixed. In other words, even in a small area, the acting shear stress ratio fluctuated greatly depending on small variations in the ground stiffness and the circumstances of the nearby topography and structures. As a result, at
certain locations where the acting amplitude was smaller than \( X_0 \), the settlement was small, and at other locations where the acting amplitude was larger than \( X_0 \), the settlement was large. Therefore, in addition to the differences in the surface soil stratum, the magnitude of the shear stress was considered one cause of the extremes in the phenomenon of liquefaction and the variation in the settlement.

This discussion has been based on a theoretical model having a lower limit value of liquefaction strength at \( X_0 \). Therefore, although it cannot be said that the model as it is represents the behavior of the soil in Urayasu, it can be said to be one explanation for variations in settlement. In the future it is necessary to verify this experimentally using on-site test specimens.

3. EVALUATION AND ANALYSIS OF LIQUEFACTION BY EFFECTIVE STRESS ANALYSIS

(I) Analysis conditions

Data were collected from several boreholes in the Urayasu area, and two types of FEM ground models were produced: location-1 where there was no liquefaction, and location-2 where there was liquefaction. Location-1 was the ground at K-NET Urayasu near the Urayasu City Office. Location-2 was set based on the data from several boreholes in the southeast of the reclaimed area (the Takasu area and the Hinode area). The settlement at both locations was 30 to 50 cm. Effective stress analysis was carried out with the ground models for locations-1 and 2, and the acceleration response and the extent of liquefaction (amount of settlement, etc.) were investigated.

Table 1 shows the stratigraphic composition at the two assumed locations. The blue portion indicates strata that could liquefy. For location-1 (non-liquefaction-prone ground), the K-NET Urayasu borehole data were used to the depth of GL-20m, and deeper strata were set by reference to nearby borehole in-company data. At location-2 (liquefaction-prone ground), an average ground model was determined based on data from several boreholes: Chiba prefecture data and in-company data. The differences between the ground at the two locations were as follows: at location-2 the \( N \)-values of the sand strata were small, the liquefaction-prone stratum in the silty sand stratum F was thick, and the strata were deep.

<table>
<thead>
<tr>
<th>GL m</th>
<th>Thickness of Strata m</th>
<th>Soil Profile</th>
<th>Fine Content Fc (%)</th>
<th>( \gamma ) kN/m^3</th>
<th>( V_s ) m/s</th>
<th>( N ) value</th>
</tr>
</thead>
<tbody>
<tr>
<td>1.0</td>
<td>1.0</td>
<td>Fill (F)</td>
<td>17.5</td>
<td>140</td>
<td>3</td>
<td></td>
</tr>
<tr>
<td>2.3</td>
<td>1.3</td>
<td>Silt (Asc)</td>
<td>17.5</td>
<td>140</td>
<td>4</td>
<td></td>
</tr>
<tr>
<td>4.6</td>
<td>2.3</td>
<td>Sand (As)</td>
<td>17.5</td>
<td>140</td>
<td>19</td>
<td></td>
</tr>
<tr>
<td>7.9</td>
<td>3.3</td>
<td>Silt (Asc)</td>
<td>17.5</td>
<td>140</td>
<td>7</td>
<td></td>
</tr>
<tr>
<td>20.0</td>
<td>12.1</td>
<td>Clay (Ac)</td>
<td>17.5</td>
<td>125</td>
<td>3</td>
<td></td>
</tr>
<tr>
<td>27.0</td>
<td>7.0</td>
<td>Silt (Asc)</td>
<td>17.5</td>
<td>230</td>
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<td>33.0</td>
<td>6.0</td>
<td>Silt (Asc)</td>
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<td>370</td>
<td>30</td>
<td></td>
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<tr>
<td>35.0</td>
<td>2.0</td>
<td>Sand (Ds)</td>
<td>19.0</td>
<td>400</td>
<td>&gt;50</td>
<td></td>
</tr>
</tbody>
</table>

(b) Location-2: liquefaction-prone ground based on the boring data of the southeast of the reclaimed area of Urayasu

The constitutive equations used in the analysis were the R-O model as described in Section 2.(2) in combination with a bowl model. Figure 8 shows the results calculated from the constitutive equation for the liquefaction strength curve of the strata that have the potential for liquefaction as determined from the \( N \)-values and the fine fraction content \( Fc \). The \( N \)-values and \( Fc \) values for the borehole data for the ground showed variations with depth, thus there was also variation in the liquefaction strength. Therefore, for the sand strata at location-2, \( R_l \) was calculated as it is with variation based on the Japanese design specifications for highway bridges, and the minimum value (about 0.2) was taken to be \( R_{20} \). Based on this, the lower limit value of liquefaction strength, \( X_0 \), was set to 0.8×0.2 = 0.16 in Equation (1), to provide the curved line of the liquefaction strength in the figure. The other strata were also set by the same method. The difference in liquefaction strength curves in Fig.8(a) and (b) are based on \( Fc \) and \( N \)-values.

Also, the dynamic deformation properties (the so-called \( G/G_0 - \gamma \), \( h - \gamma \) relationship) were set to standard curves for sand, silt and clay based on Reference 8 (approximately).
The input seismic motion was the acceleration record for the main shock and aftershock measured at engineering bedrock (GL-4.1m, $V_s = 430$ m/s) at the Shimizu Institute of Technology. The distance between the measurement point and Urayasu City Office is 9.4 km. Figure 9 shows the acceleration record and the response spectrum measured at engineering bedrock. The NS component and the EW component of the engineering bedrock wave at the bottom surface of the analysis were input simultaneously as E+F waves. The primary natural period at the point of measurement was 1.18 seconds, which is close to the natural period at location-2. (Gal=cm/s²)

While liquefaction caused by the main shock was continuing (after 29 minutes from the main shock), the largest aftershock occurred ($M = 7.7$). Therefore in the analysis, it was assumed that during the period from the main shock to the largest aftershock, there was no pore water pressure dissipation, and the main shock and the aftershock were input as continuous. The groundwater level was the same in the main shock and the aftershock.

In this analysis, the vibrations in two directions were applied simultaneously in a three-dimensional model. Therefore in addition to the shear strain for each component, the resultant shear strain $\Gamma$ and the cumulative shear strain $G^*$ given by the following equations were used:

$$\Gamma = \sqrt{2\gamma^2 + \gamma^2 + \gamma^2 + (\epsilon_\alpha - \epsilon_\beta)^2 + (\epsilon_\gamma - \epsilon_\delta)^2 + (\epsilon_\delta - \epsilon_\gamma)^2}$$

$$G^* = \sum \Delta G^*$$

$$= \sum \sqrt{\Delta \gamma^2 + \Delta \gamma^2 + \Delta \gamma^2 + (\Delta \epsilon_\alpha - \epsilon_\beta)^2 + (\Delta \epsilon_\gamma - \epsilon_\delta)^2 + (\Delta \epsilon_\delta - \epsilon_\gamma)^2}$$

(2) Analysis results and discussion

a) Non-liquefied ground (location-1)

Figures 10 and 11 show a comparison of the effective stress analysis results for the acceleration wave form and the acceleration response spectrum for the main shock at K-NET Urayasu measured at location-1. Liquefaction was not confirmed in the soils at this location. In the analysis, the excess pore water pressure in the sand strata increased by about 50%, but did not liquefy. Both wave forms are similar, and although in the acceleration response spectrum the analysis values become small at a period of about 2 seconds, there is general agreement. From this it can be seen that the engineering bedrock wave from Shimizu Institute of Technology can be applied to the engineering bedrock wave in the Urayasu area.

b) Liquefaction-prone ground (location-2)

Figures 12 and 13 show the maximum excess pore water pressure distribution, the maximum resultant shear strain distribution, the acceleration wave form, and the excess pore water pressure ratio wave form at location-2. The liquefied strata were the sand stratum (As) and the silty sand (reclaimed stratum F). In stratum F, a $\Gamma$ of 9% or greater was produced, and in the As stratum it was 4 to 6%.
Looking at the excess pore water pressure ratio wave form, it can be seen that the speed of increase in the water pressure is slow, and liquefaction occurs after the main seismic motion has passed (in other words, liquefaction did not occur for about 80 seconds after the seismic motions). Thereafter the vibration continued, thus a large shear strain was produced. This is consistent with the element calculation results shown in Fig. 7. This process towards liquefaction is different from past records and simulation results.

Fig. 9 Measured acceleration wave form and response spectrum at bedrock ($V_s = 430$ m/s) at Shimizu Institute of Technology (Gal=cm/s²).

Fig. 10 Comparison of acceleration wave form of the main shock at K-NET Urayasu measured at location-1.

(b) Calculated wave by effective stress analysis

Fig. 11 Comparison of acceleration response spectra for the main shock at K-NET Urayasu measured at location-1.

Looking at the excess pore water pressure ratio wave form, it can be seen that the speed of increase in the water pressure is slow, and liquefaction occurs after the main seismic motion has passed (in other words, liquefaction did not occur for about 80 seconds after the seismic motions). Thereafter the vibration continued, thus a large shear strain was produced. This is consistent with the element calculation results shown in Fig. 7. This process towards liquefaction is different from past records and simulation results.
For example, in the 1995 Kobe Earthquake, which was an active-fault-type earthquake, the "killer pulse" of the main motion caused sudden liquefaction, producing the maximum shear strain, and thereafter the vibration time was short, and there was not much accumulation of strain after the main motion (for example, see the analysis results of Reference 6).

For unidirectional components (NS or EW component only) liquefaction was limited to a part of the sand stratum, and it was not possible to explain the severe liquefaction that occurred. In the seismic motion conditions here, the acceleration amplitude is not very large, and the phenomenon is close to the lower limit value for liquefaction, thus the effect of the bi-directional input on the occurrence of liquefaction is considered to be particularly large.

(3) Effect of aftershock on the liquefaction and settlement

In interviews with the local public regarding the largest aftershock on this occasion (15:15 hours March 11th, $M = 7.7$), the following evidence was obtained. "At Shin Kiba, mainly water was ejected during the main shock, but sand was also ejected in the aftershock." "In eastern Kanto (the Tone River catchment area), sand was ejected in the main shock, and the ground flowed during the aftershock." "In the backfilled ground of the old iron sand quarry, sand was ejected in both the main shock and the after-shock."

From this, the largest aftershock was considered to have contributed to the liquefaction and settlement. To verify the effect of the aftershock, the analysis results for the main shock and the largest aftershock were input continuously at location-2. This was because 29 minutes after the main shock the excess pore water pressure ratio was still close to 100%, thus the liquefaction state continued.

Generally, the magnitude of the shear strain that is produced during an earthquake corresponds to the extent of liquefaction or the magnitude of the ground deformation. To check the effect of earthquake duration and the aftershock on the ground deformation, such as settlement, etc., Fig.14 shows the time history of the resultant shear strain $\Gamma$. Focusing first on the main shock, the maximum value of $\Gamma$, $\Gamma_{\text{max}}$, is produced after the maximum value of the input acceleration. As shown in Fig.13, this is due to the excess pore water pressure ratio reaching 1.0 after maximum acceleration.

The amplitude of $\Gamma$ including the aftershock was greater in the aftershock than in the main shock for elements 5, 12, and 16. This is because a small shear force due to the aftershock acts on a stratum where the shear stiffness has become very small in the main shock, and induces a large shear strain; thus in this type of stratum, the liquefaction and settlement due to the aftershock are promoted. This is consistent with the results of the interviews as described above.

In calculating the ground settlement after the earthquake, a method frequently used is to obtain the volumetric strain after the earthquake from the maximum shear strain experienced during the earthquake, which is then integrated with depth. This method has been proposed for sand foundations\textsuperscript{9,10} and clay foundations\textsuperscript{11} but in each of the proposed equations, after undrained cyclic shearing test as element tests, a cock is opened to drain the water, and the volumetric strain after cyclic shearing is obtained based on test results.

In the Earthquake 3.11 a large quantity of sand was ejected, and in some places was deposited 30 to 40 cm deep. If the above empirical equations are applied to evaluate the settlement at locations like these, the settlement will be underestimated. This is because in the element tests, only water is ejected, and the effect of ejection of soil particles is not included.
Therefore, in calculating the settlement, the objective here was to determine the extent of the main shock and the aftershock, rather than to quantitatively evaluate the settlement. The settlement was calculated by obtaining the volumetric strain from the maximum shear strain from the equation of Ishihara and Yoshimine\(^{10}\) for sandy soils, and the equation of Shamoto et al.\(^{11}\) for clay soils.

Figures 15 and 16 show the distribution with depth of the amount of settlement \(S\) calculated from the main shock + aftershock continuous analysis results, and a comparison of the actual measured ground surface settlement and the analysis values. The maximum shear strain during the seismic motion used to calculate \(S\) was obtained using the maximum value of the resultant shear strain \(\gamma_{\text{max}}\), and the settlement \(S\) of the ground surface was obtained for the following four cases.

Case-1: Calculation of the settlement using \(\gamma_{\text{max}}\) produced by the main shock and the aftershock: \(S_{\text{total}}\). (Using \(\gamma\) over the whole time; 0 to 580 seconds)

Case-2: Calculation of the settlement using \(\gamma_{\text{max}}\) produced by the main shock only: \(S_{\text{main}}\). (Using \(\gamma\) over 0 to 350 seconds)

Case-3: Calculation of the settlement using \(\gamma_{\text{max}}\) produced by the aftershock only: \(S_{\text{after}}\). (Using \(\gamma\) over 360 to 570 seconds)

Case-4: Calculation of the sum of the settlement in the main shock (Case-2) and the settlement in the aftershock (Case-3) separately: \(S_{\text{sum}} = S_{\text{main}} + S_{\text{after}}\)

The settlement in the main shock from Case-2 is 16.6 cm, the settlement in the aftershock from Case-3 is 11.1 cm, thus the settlement in the main shock + aftershock from Case-4 is 27.7 cm.
Fig. 14 Resultant shear strain $\gamma$ time histories for the main shock and the aftershock input continuously.
If this final settlement is taken to be 100%, then 60% of the settlement was produced by the main shock, and 40% of the settlement was produced by the aftershock. The fact that a considerable amount of the settlement was produced during the aftershock is consistent with the interview results described earlier. Comparing the final settlement with the actual measured settlement, the calculated results provide values that are smaller compared with the actual settlement. The settlement at the location of the boreholes based on Table 1(b) was 30 to 50 cm, and the settlement from Case-4 was somewhat smaller than these values. As stated previously, this could be because when evaluating the settlement, the quantity of sand and soil ejected was not taken into consideration.

The amount of settlement obtained in Case-1 was 17.0 cm, which was considerably underestimated. The reason for this is that amplitudes close to the maximum value of $\gamma$ were produced several times during the main shock and the aftershock, as can be seen from the time history of $\gamma$. There was only an interval of 29 minutes between the main shock and the aftershock, but they were separate earthquakes, thus rather than calculating the settlement considering the main shock and the aftershock to be a single earthquake as in Case-1, the settlement should be calculated separately as in Case-4. This would be the same even if the interval between the main shock and the aftershock were, for example, 1 minute. Taking this idea to its logical conclusion (assuming there is no time interval between the main shock and the aftershock), they can be considered to be a single earthquake. Therefore, in earthquakes for which the duration time is long, as in multi-segment earthquakes, the method of calculating the settlement should be as follows: divide the total duration of the earthquake into several time periods, calculate the settlement in each of the time periods using $\gamma_{\text{max}}$, and add the calculated settlements. This concept is similar to the cumulative damage theory.
Therefore, in earthquakes with long duration, there is a possibility that calculating the settlement by obtaining a single $I_{\text{max}}$ will underestimate the settlement. It is not possible to say that the shear strains that are smaller than $I_{\text{max}}$ all contribute to the settlement, but if the concept of "accumulation" of strain is not introduced, it is possible to underestimate the settlement. The concepts of accumulation include, for example, the cumulative shear strain $G^*$ as shown in Equation (3).

### 4. DISCUSSION OF THE APPLICABILITY OF THE METHOD OF DETERMINING LIQUEFACTION

The results of applying the method of determining liquefaction ($F_L$ method) based on the Specifications for Highway Bridges Part V Seismic Design by the Japan Road Association for location-2 in Table 1 are shown as the blue line in Fig.17. Here the value of $F_L$ is given by the following equation, where $R$ is the liquefaction resistance value (liquefaction strength), and $L$ is the acting shear stress ratio.

$$F_L = \frac{R}{L} \tag{4}$$

The maximum acceleration amplitude on the ground surface was set to 120 Gal, which is slightly larger than the result in Fig.13 (100 Gal). The determination results showed weak liquefaction for silty sand ($F_L = 0.82$ to 1.0) and borderline liquefaction for sand ($F_L = 0.96$ to 1.06), thus even though the acceleration was set on the high side, the extent of liquefaction was underestimated compared with the analysis results. To obtain consistency between the two, it is necessary to review the liquefaction strength $R$ and the acting shear stress $L$ in the $F_L$ method, and a proposal for this is shown below.

#### (1) Review of liquefaction strength $R$

In earthquakes with long duration, the value of the liquefaction strength $R$ should be reduced to take into consideration the effect of the number of cycles. Instead of the liquefaction strength $R_{20}$ at 20 cycles, it is possible to consider, for example, the liquefaction strength $R_{100}$ at 100 cycles ($= 0.9$ to $0.8 \times R_{20}$), or to use the lower limit value $X_l$ of the liquefaction strength (see the following equation).

$$R \rightarrow \left\{ \begin{array}{ll} \alpha \times R_{20} & (\alpha = 0.9 \sim 0.8) \\ X_l & \end{array} \right. \tag{5}$$

The value of $\alpha$ varies with the density and the fine fraction content $F_c$. The higher the density and the higher the fine fraction content $F_c$, the smaller the value of $\alpha$. Figure 18 shows the liquefaction strength curve for Sengenyama sand ($F_c=2.4\%$) for various relative densities $D_r$, and the coefficient $\alpha$ in the above equation. The results for Toyoura sand according to Toki and others are: $\alpha=0.89$ ($D_r=80\%$), 0.94 ($D_r=50\%$). Neglecting the influence of $F_c$, $R_{100}$ and $X_l$ are given approximately as follows:

$$R_{100} \approx X_l = \left(1.0 - 0.3 \frac{D_r}{100}\right) R_{20} \tag{6}$$

Based on these results, the liquefaction determination results using $0.85 \times R_{20}$ as $R$ are shown in Fig.17, corresponding to the analysis results for the excess pore water pressure ratio. Therefore, when there is a large number of cycles at small acceleration (in the case of ground distant from a multi-segment type earthquake), it is necessary to correct $R$ as in the equation above.

#### (2) Review of the acting shear stress $L$

When the magnitude $M$ is large, it seems to be necessary to take into consideration the effect of the earthquake duration (number of cycles) in another way.
In the Recommendations for Design of Building Foundations\(^{18}\), the effective number of cycles of the envisaged seismic wave form is corrected with a coefficient \(r_n\) to take into consideration the ground density. However, doubts remain whether the relationship\(^{14}\) between the magnitude \(M\), the number of cycles, and the correction coefficient \(r_n\) can be used up to \(M9.0\).

In a method proposed by the National Center for Earthquake Engineering Research (NCEER)\(^{15}\), the \(F_L\) value is obtained with a \(M7.5\) earthquake as criterion, thus the \(F_L\) value is corrected using a correction factor MSF. The relationship between the correction factor MSF for the liquefaction safety factor and the magnitude M shown in that method goes up to \(M8.5\), but is not proposed up to \(M9.0\).

If instead of the amplitude of the acceleration, the Arias intensity is used, which is defined as an energy based on the two horizontal components of the seismic motions and has the units of velocity, both the amplitude of the acceleration and the duration are evaluated over each frequency range, thus in the opinion of some this is more logical.\(^{16}\) In any case, in the current method of determining liquefaction, the duration and the number of cycles are determined from \(M\) only, and this is not appropriate for giant earthquakes or multi-segment earthquakes. The failure mechanism at the epicenter, the transmission path, and the ground structure, etc., should be taken into consideration.

5. SUMMARY

In this paper, liquefaction of the ground mainly in the Urayasu area in the Earthquake 3.11 was studied. Effective stress analysis was also carried out for a location where there was liquefaction and a location where there was no liquefaction, inputting the main shock and the aftershock continuously. From the results, the following can be summarized.

1) The amount of settlement in the Urayasu, Shin Kiba, and Tatsumi areas was measured a few days after the occurrence of the earthquake, and a map of the distribution of the settlement was produced. The results showed that the amount of settlement varied greatly from spot to spot, even within the same area. The causes were due to: (1) differences in soil and differences in the surface thickness of the reclaimed stratum, and (2) whether the ground had been improved or not. In addition, according to trial calculations using the constitutive equation (bowl model) used here, another cause was the small differences in the acting shear stress due to the large number of cycles causing extreme phenomena (after liquefying once, the subsequent large number of cycles resulted in severe liquefaction).

2) The accelerations were not very high, but the number of cycles was large, thus the rise in excess pore water pressure was slow, and slowly increased after the main seismic motion had passed, resulting in liquefaction. Thereafter, when the oscillations continued, severe liquefaction occurred.

3) When the total settlement due to the main shock and the aftershock was analyzed, based on the assumption that the settlement was the sum of the settlements due to the main shock and the aftershock separately, it was found that the settlement was 27.7 cm, which was smaller than the values (30 to 50 cm) found in the site survey. This was because when evaluating the settlement, the ejection of sand was not taken into consideration.

4) The aftershock also caused a considerable amount of the settlement, 40% of the total settlement being due to the aftershock. This was consistent with the testimony obtained from site.

5) In earthquakes with extremely long durations and large number of cycles, by reducing the liquefaction resistance value to about 0.85, it is possible to obtain a value of \(F_L\) that is consistent between the actual liquefaction phenomena and the analysis results.
In the Tokyo Bay area including Urayasu, detailed soil investigations and level survey for long-term settlement of clay layer are in progress[17]. In the future, we intend to confirm the validity of the analysis parameters, etc., using this type of soil test data, and carry out estimates of damage in multi-segment earthquakes that are postulated to occur in the future.

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