Table 1. Physical characters of soils

| Soil          | Fc (%) | $e_{max}$ | $e_{min}$ | $\rho$(g/cm$^3$) | Uc  | $D_{50}$(mm) |
|---------------|--------|-----------|-----------|------------------|-----|-------------|
| Aira shirasu  | 0      | 1.869     | 1.011     | 2.453            | 4.2 | 0.34        |
|               | 10     | 1.767     | 0.888     | 2.454            | 5.8 | 0.30        |
|               | 20     | 1.720     | 0.879     | 2.455            | 22.0| 0.24        |
|               | 30     | 1.757     | 0.870     | 2.456            | 36.0| 0.19        |
| Toyoura sand  | 0      | 0.968     | 0.628     | 2.636            | 1.2 | 0.20        |

Experimental study on the dynamic deformation characteristics of volcanic soil "Shirasu" on equivalent granular void ratio

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ABSTRACT

Shirasu is a volcanic soil which is widely distributed in South Kyushu in Japan. It contains non-plastic fines which was created by its fragment of coarse particles. In this study, a series of bender element tests on Shirasu was performed with varying its fines content. As a result, it was observed that the shear modulus of Shirasu was decreased with increasing the fines content although void ratio decreased as fines content increased. However, it was found that there is a unique relationship between shear moduli of Shirasu and the equivalent granular void ratio, in which the proper contribution factor $b$ for fines was assumed. Based on the findings in the study, an empirical formula to express the shear moduls of Shirasu with various fines content was developed as the function of the fines content, the equivalent granular void ratio and the effective confining stress.

Keywords: volcanic soil, shirasu, bender element test, shear modulus, fines content

1. INTRODUCTION

Studies of liquefaction since the 1964 Niigata Earthquake have been mainly carried out on clean river sands. Liquefaction failures mainly occur in reclaimed land and artificial islands which in Japan often consist of residual soils or volcanic ash. In the south of Kyushu, Japan’s southernmost large island, widely distributed deposits of Shirasu resulting from pyroclastic flows are frequently used for soil structures and fills. Shirasu fills are often deposited as wet slurries and therefore there are major concerns about their future stability. Very Little research work has been carried out on such soils. The Ebino Earthquake in 1968 (M=6.1) and the Kagoshimaken Hokuseibu Earthquake which occurred twice in 1997 (M=6.3) caused liquefaction of Shirasu deposits.

Shirasu consists of rapidly cooled very rough glassy particles whose morphology is not clear. These particles are highly crushable leading to a high fines content which affects both monotonic and cyclic strength. In order to carry out seismic response analyses it is necessary to understand the effect of the fines on the small strain shear characteristics and in particular small strain shear modulus values. Asonuma et al. (2002) performed cyclic triaxial tests on non-plastic volcanic soil to investigate its cyclic deformation properties taking into account soil crushability and its effect on dynamic shear deformations. In their paper they related the generation of fines, which causes a decrease in void ratio results in a decrease in monotonic and cyclic shear strength and associated stiffness. Iwasaki and Tatsuoka(1977), Suhapol and Miura(2005), and Salgado et al.(2000), reported that the dynamic shear modulus for non-plastic soils decreased with increasing...
void ratio $e_{ge}$. The granular void ratio is calculated assuming the fines to be part of the voids. On the other hand the equivalent void ratio was calculated assuming only some proportion of the fines contributed to the voids.

The granular void ratio was therefore defined as

$$ e_{ge} = \frac{V_r + V_{sf}}{V_{ss}} $$

(1)

where; $V_r$ is the volume of voids; $V_{sf}$ is the volume of fines and $V_{ss}$ is the volume of coarse grained solids.

The equivalent granular void ratio was introduced by Thevenayagam (2000) to extend the concept of granular void ratio. In this case the solids include a proportion of the fines. By using this concept it was possible to construct a unique $e$ vs $p'$ steady state line for various fines contents. If the specific gravity for the fine and coarse materials are assumed to be identical then the fines content by volume is defined as:

$$ F_c = \frac{V_{sf}}{V_{ss}} $$

(2)

and the effective granular void ratio is:

$$ e_{ge} = \frac{e + (1 - b) F_c}{1 - (1 - b) F_c} $$

(3)

Where: $e$ is the void ratio and $b$ is the fines contribution factor.

In the case of $b=1$, $e_{ge}=e$ and in the case of $b=0$, $e_{ge}=e_{gg}$. $b$ is assumed to vary between 0 and 1 for the fines.

The fines content was varied between 0% and 30% in order to evaluate an appropriate $b$ value using the line for 0% fines as a benchmark.

3 BENDER ELEMENT TESTS

3.1 Testing equipment

The bender elements were arranged such that the transmitter was in the base pedestal and receiver in the top cap of the triaxial sample. The bender elements were located in the same plane and the transmitter was wired in parallel while the receiver was in series. (Lee and Santamarina 2005). The receiver was 4.4mm high, 12mm wide and 1.4mm thick, while the transmitter was 4.1 mm high, 12mm wide and 1.2 mm thick. These dimensions include the thickness of the coating. The height of the element represented the length intruded into the sample. The bender element system was self monitoring and the voltage generated by a sensor on the transmitter was recorded as the transmitter signal.

3.2 Sample preparation

Specimens 50mm diameter and 100mm high were compacted at 10% moisture content with 3 different compaction energies $Ec = 22, 113$ and 504 kJ/m$^3$. A latex membrane was placed on the mould and the samples were then compacted in five layers with a steel rammer controlling drop height and number of blows to
achieve a given compaction energy calculated as:

$$E_c = \frac{W_R \times H \times N_t \times N_b}{V}$$  \hspace{1cm} (4)

where: $W_R$ is the weight of the rammer (0.00116kN); $H$ is the drop height; $N_t$ is the number of layers; $N_b$ is the number of blows per layer and $V$ is the volume of the specimen.

The total energy was controlled by the blow count, all other parameters being kept constant except for the case of the lowest energy of 22kJ/m$^2$ when the drop height was only 0.050m. In the case of 113 and 504kJ/m$^2$ the drop height was 0.184m.

### 3.3 Testing method

Initially air was displaced from specimens by sparging with carbon dioxide. De-aired distilled water was then permeated through the soil at a back pressure of 100kPa and an effective stress of 20kPa to achieve a pore pressure $b$ value larger than 0.95 for all specimens.

Bender element tests were carried out on samples isotropically consolidated to 50, 100, 200 and 400kPa using a sinusoidal 20v signal over a range of frequencies $f=2.5, 5, 10, 15, 20, 25$ and 30Hz. The transmission length used for shear wave velocity calculation was taken as the tip to tip distance between the bender elements. The velocity $V_s$ and shear modulus $G$ were calculated from:

$$V_s = \frac{L}{\Delta t}$$  \hspace{1cm} (5)

$$G = \rho_i \cdot V_s^2$$  \hspace{1cm} (6)

where $\rho_i$ is the dry density of the specimen.

### 4 TEST RESULTS

Empirical equations of the following form have been proposed for the shear modulus by many researchers as a function of void ratio and effective stress:

$$G = C P_h^{1-n} f(e) \sigma_m^{-n}$$  \hspace{1cm} (7)

Where $C$ and $M$ are empirical constants, $f(e)$ is a function of void ratio, $P_h$ (=100kPa) is a normalising parameter for stress. In this study $\sigma_m$ is the same as $\sigma_c$. Hardin and Richart (1963) and Jamtjikovski (1991) proposed Equations (8) and (9) for $f(e)$

$$f(e) = \frac{\left(B-e^{-1}\right)^2}{1+e}$$  \hspace{1cm} (8)

$$f(e) = e^a$$  \hspace{1cm} (9)

The parameter $n$ corresponds to the slope of the log $G$ vs log $\sigma_m$ relation. Salgado et al. performed bender element tests on Ottawa sand mixed with non-plastic silt and showed that $n$ increased with increasing fines. For $F_e = 0, 5, 10, 15\%$ $n$ was found to be 0.443, 0.458, 0.557, 0.715 respectively. If an average value of 0.5 is assumed for $F_e = 0$ Equation 10 reduces to:

$$\frac{G}{\left(\sigma_m/P_h\right)^{1.5}} = C P_A e^a$$  \hspace{1cm} (10)

This equation shows the shear modulus normalised by $\sigma_m$ as a function of void ratio. The equivalent granular void ratio $e_{ge}$ was introduced into this equation instead of $e$.

Figures 3 show the normalised shear modulus variation with equivalent void ratio for various values of $b$. In Figure 3a the average line for the Hardin and Richart (1963) data is shown for coarse Ottawa sand using:

$$\frac{G}{\left(\sigma_m/P_h\right)^{1.5}} = 327 \cdot P_A \cdot \frac{(2.97 - e)^2}{1 + e} \text{ (in kPa)}$$  \hspace{1cm} (11)

In the case of Shirasu with no fines the results fell on the Hardin and Richart line. For the case of $b=1$ ($e=e_{ge}$), the normalised shear modulus decreased with fines content and no single unique relationship could be found. On the other hand for $b=0$ ($e_{ge}=e_{ge}$) the normalised shear modulus increased with increasing fines. The relationship was therefore investigated for a range of $b$ values. There appears to be a unique relationship for normalised shear modulus and equivalent void ratio for $b=0.5$. means that 50% of the fines contribute to the behaviour of the coarse grained skeleton. In the case of $F_c=0$ then:

$$\frac{G}{\left(\sigma_m/P_h\right)^{1.5}} = 640 \cdot P_A \cdot e^{-0.5} \text{ (in kPa)}$$  \hspace{1cm} (12)

Using $b=0.5$ a unique relation is obtained for the normalised shear modulus for a range of $F_c$ values from 0% to 30% such that:

$$\frac{G}{\left(\sigma_m/P_A\right)^{1.5}} = 540 \cdot P_A \cdot e^{-1.5} \text{ (in kPa)}$$  \hspace{1cm} (13)

which reduces to:

$$G = 640 \cdot P_A^{0.5} \cdot e^{-1.5} \cdot \sigma_m^{0.5} \text{ (in kPa)}$$  \hspace{1cm} (14)

and then substituting Equation 3 for the equivalent void ratio we obtain:

$$G = 640 \cdot P_A^{0.5} \cdot \left(\frac{\sigma + 0.5 \cdot F_c}{1 - 0.5 \cdot F_c}\right)^{-1.5} \cdot \sigma_m^{0.5} \text{ (in kPa)}$$  \hspace{1cm} (15)

### 5 CONCLUSIONS

Bender element tests on triaxial samples have been carried out to determine the influence of fines on the shear modulus of Shirasu, a volcanic soil. It was concluded that:

1) The shear modulus of Shirasu decreases with increasing fines and decreasing void ratio.
2) The shear modulus was shown to be a function of effective stress raised to the power 0.5.
3) The contribution factor $b=0.5$ was applicable to Shirasu with a fines content from 0% to 30%.
4) Using this factor a unique relation was found between shear modulus, void ratio and mean effective confining stress.
Figure 3. Relation between shear modulus and equivalence granular void ratio with normalized by mean effective principal pressure.

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