STUDY ON THE PROPERTIES OF SOILS IN THE NORTHERN COAST OF TOKYO BAY USING A SELF-BORING PRESSUREMETER

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ABSTRACT

Soil disturbance caused by boring and the stress relief of soil around a borehole left unsupported may influence the parameters obtained from the Menard pressuremeter (abbreviated as MP). The paper presents the results of tests obtained from a self-boring pressuremeter (abbreviated as SBP) devised by the author for a variety of soils such as soft clay, loose sand dense sand distributed in the northern coast of Tokyo Bay. The instrument containing self-boring mechanism is statically penetrated into ground, digging a hole simultaneously. The undrained shear strength of soft clay obtained from the SBP test was well correlated with that obtained from undrained triaxial tests for undisturbed samples reconsolidated under the pressure equivalent to the effective mean stress in situ. The shear modulus during undrained deformation of soft clay obtained from the SBP test was, in general, well correlated to that obtained from triaxial tests. The strength and modulus of cohesive soils were considerably underestimated by unconfined compression tests.

The shear modulus obtained from the SBP was approximately 2 to 7 times greater than that obtained from the MP test. For the soils investigated, aside from loose sand, no significant difference in the limit pressure was conceived between the SBP test and the MP test.

Key words: boring, consolidated undrained shear, deformation, pressuremeter test, stress-strain curve, test equipment

IGC: D6/(C7)

INTRODUCTION

The results obtained from Menard pressuremeter tests used in Japan over 15 years have been applied to various problems in foundation engineering. However, it was pointed out that the modulus of deformation obtained from pressuremeter tests was only one third of the modulus back-calculated from laterally loaded piles (Yoshida and Yoshinaka, 1972). Shields and Bauer (1975) concluded that only the plate loading tests gave moduli comparable to those back-calculated from prototype footings, and that moduli obtained from the pressuremeter tests were 1/2 to 1/3 of the back-calculated moduli.

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Among possible reasons causing such discrepancies, the influence of soil disturbance around a hole pre-bored and left unsupported is considered serious. In order to minimize disturbance of soil, self-boring pressureometers were developed by Hughes (1973) and Baguelin (1972). Although the self-boring pressureometer test is thought to be an improvement of testing techniques, it is reported that the undrained strength obtained from pressureometer tests is considerably larger than those derived from vane shear tests (Amar et al., 1975). Clough and Denby (1980) obtained a little larger undrained strength of soft clay from self-boring pressureometer tests than that derived from conventional triaxial compression tests and vane shear tests. However the undrained strength from the triaxial tests for reconsolidated specimens was equal to or slightly above the results from the pressureometer tests. Windle and Wroth (1977) found that the values of undrained strength of stiff clays from pressuremeter tests were either in good agreement to or exceeding those derived from triaxial tests and cone tests. The values of undrained Young's moduli derived from pressuremeter tests agreed closely with those from large diameter plate tests. Denby and Clough (1980) pointed out that soil disturbance caused by improper insertion of a probe resulted in higher values of undrained strength obtained from the pressuremeter test.

The author built a self-boring pressuremeter improving Menard pressuremeter consisting of the main cell to measure volume change versus applied pressure and the guard cell to ensure plain-strain deformation of soil around the main cell. While Menard used gas pressure to inflate the guard cell, both main and guard cells of the probe built by the author are inflated by pressure of injected water.

This paper presents the results of tests obtained from an instrument devised by the author for a variety of soils such as soft clay, loose sand, and dense sand distributed in the northern coast of Tokyo Bay. The consistency of the parameters obtained from the self-boring pressuremeter was studied comparing the results of tests in two to three boreholes closely spaced each other. The parameters obtained from the self-boring pressuremeter were compared to those from the Menard pressuremeter.

SELF-BORING INSTRUMENT AND TEST PROCEDURE

The outline of two different types of the self-boring pressuremeter (abbreviated as SBP hereafter) is schematically illustrated in Fig.1 and Fig.2. The instrument, type A, shown in Fig.1 is adopted for soft clay and loose sand \((N \leq 10)\). Through the center of G-type probe of Menard pressuremeter (Technique Louis Menard, 1975), a hollow steel shaft is placed. The top of shaft is jointed with a drill rod of 40 mm outside diameter, and a drill bit is attached at its lower end. Thrust bearings between the shaft and the probe make the probe free from rotation of the shaft. The pipes of 65 mm in diameter placed outside of the drill rod are used to transmit a vertical force to push the probe.
The cutting shoe at the lower end of the probe is slightly tapered outwards with an outside clearance ratio of 0.5%. The cutting shoe has saw teeth at its bottom in order to facilitate efficient penetration of the probe and to prevent rotation of it during penetration. The rotation of probe is prevented by thrust bearings, but torsional force may be given to the probe if cuttings clog the space between the bit and shoe.

The instrument of type A can be penetrated into soft clay and loose sand without damage to a rubber membrane of the probe. But the membrane is collapsed when the probe is pushed in dense sand. In the type B, the membrane is protected with a steel sheath of 0.7 mm thick (See Fig. 2). The sheath slitted at 7 mm interval in the longitudinal direction allows the probe to expand with a small inertia. The instrument type B can be penetrated into dense sand without damage of the membrane.

Two piezometers are installed between the lower end of the probe and the top of cutting shoe to measure pore-pressure built up during penetration. Pore pressure in sandy soil dissipates normally in 10 to 20 minutes, but it takes a few hours until pore pressure in soft cohesive soil dissipates.

Prior to the installation of the probe, air in both main cell and guard cell is completely evacuated by circulating desired water. Then the outside membrane is inflated to a diameter (85±0.5 mm), while the membrane of the main cell is in contact to the outside membrane by applying pressure of the main cell about 20 kPa higher than that of the guard cell.

A borehole is advanced to a depth about one meter above a depth of testing, then the SBP instrument is penetrated from the bottom of a borehole to a desired depth. Soil inside the shoe is crushed by a bit, carried by drilling fluid, and finally drained through holes at the top of instrument. The SBP instrument can be penetrated into soft cohesive soil or very loose sandy soil by a thrust force of about 10 kN produced by a drill rig. For dense sand or stiff clay, a thrust force up to 100 kN is applied by a hydraulic jack.

When the SBP instrument is penetrated to a desired depth, pore pressure is measured by two piezometers until excess pore pressure becomes less than 5 kPa. The effective horizontal stress is calculated subtracting the hydrostatic pressure from the total stress applied to the probe. The testing procedures thereafter is similar to the Menard pressuremeter test.

TESTS IN SOFT COHESIVE SOIL

Soil Conditions: The site was selected in an area named Komatsugawa in the northern coast of Tokyo Bay. The ground was covered with loose delta sand of 8 m underlain with soft cohesive soil to the depth of 21 m. In Fig. 3 are shown index properties of soil in the test site. The ground water table was about 2.30 m deep from ground surface during testing, and seasonal variation of the water table was expected.

The cohesive soil was classified to CL, MH or CH according to the Unified Classification, but its liquid limit was considerably lower than that of common Tokyo Bay mud whose liquid limit is usually 80 to 100%. The cohesive soils at this site contained 5 to 20 percent of sand (See Fig. 3), which was not uniformly mixed with cohesive soil, but number of thin pipes or laminations developed. Small fragments of shells were also contained. Undisturbed samples were obtained from every meter of depth almost continuously using a 75 mm thin-walled sampler with fixed piston.

The over-consolidation ratio (abbreviated as OCR hereafter) was calculated from the results of oedometer tests conducted in accordance with the Japanese Industrial Standard. The calculated OCR is plotted against depth in Fig. 4. The OCR values are close
Table 1. Parameters obtained from undrained triaxial tests, Komatsugawa, Tokyo

<table>
<thead>
<tr>
<th>No. of Sample</th>
<th>depth (m)</th>
<th>$av. c_u$ (kPa)</th>
<th>$\mu$</th>
<th>$av. C_v$ (MPa)</th>
<th>$\mu$</th>
</tr>
</thead>
<tbody>
<tr>
<td>S- 1</td>
<td>8.40</td>
<td>65</td>
<td>0.04</td>
<td>3.616</td>
<td>0.10</td>
</tr>
<tr>
<td>S- 2</td>
<td>9.40</td>
<td>95</td>
<td>0.18</td>
<td>4.057</td>
<td>0.12</td>
</tr>
<tr>
<td>S- 3</td>
<td>10.40</td>
<td>92</td>
<td>0.06</td>
<td>5.165</td>
<td>0.19</td>
</tr>
<tr>
<td>S- 4</td>
<td>11.35</td>
<td>76</td>
<td>0.09</td>
<td>5.076</td>
<td>0.09</td>
</tr>
<tr>
<td>S- 5</td>
<td>12.35</td>
<td>74</td>
<td>0.02</td>
<td>5.606</td>
<td>0.02</td>
</tr>
<tr>
<td>S- 6</td>
<td>13.40</td>
<td>71</td>
<td>0.03</td>
<td>4.812</td>
<td>0.05</td>
</tr>
<tr>
<td>S- 7</td>
<td>14.40</td>
<td>73</td>
<td>0.04</td>
<td>5.370</td>
<td>0.14</td>
</tr>
<tr>
<td>S- 8</td>
<td>15.39</td>
<td>78</td>
<td>0.04</td>
<td>5.694</td>
<td>0.04</td>
</tr>
<tr>
<td>S- 9</td>
<td>16.38</td>
<td>93</td>
<td>0.10</td>
<td>5.596</td>
<td>0.12</td>
</tr>
<tr>
<td>S- 10</td>
<td>17.35</td>
<td>92</td>
<td>0.08</td>
<td>4.910</td>
<td>0.06</td>
</tr>
<tr>
<td>S- 11</td>
<td>18.30</td>
<td>93</td>
<td>0.05</td>
<td>5.459</td>
<td>0.05</td>
</tr>
<tr>
<td>S- 12</td>
<td>19.35</td>
<td>109</td>
<td>0</td>
<td>6.243</td>
<td>0.05</td>
</tr>
<tr>
<td>S- 13</td>
<td>20.30</td>
<td>123</td>
<td>0.01</td>
<td>5.419</td>
<td>0.07</td>
</tr>
<tr>
<td>S- 14</td>
<td>21.43</td>
<td>133</td>
<td>0.02</td>
<td>6.488</td>
<td>0.02</td>
</tr>
<tr>
<td>S- 15</td>
<td>22.25</td>
<td>135</td>
<td>0.03</td>
<td>5.184</td>
<td>0.09</td>
</tr>
</tbody>
</table>

$\mu$: coefficient of variation for 3 specimens

to unity in the middle of layer, and higher than 2 near the upper and lower boundaries.

The undrained shear strength of samples was measured by consolidated undrained tests. Specimens were re-consolidated by isotropic pressure equivalent to the mean effective confining pressure in situ assuming the coefficient $K_s = 0.5$. The undrained shear strength, $c_u$, was obtained from the deviatoric stresses at failure in undrained triaxial tests whose results are summarized in Table 1.

The secant modulus, $E_{50}$, corresponding 50 percent of the deviatoric stress at failure
was calculated from stress–strain curves of undrained triaxial tests. The shear modulus, \( G \), was calculated from the equation, \( G = E_0/2(1+\nu) \), where the Poisson’s ratio, \( \nu \), was assumed 0.5 for undrained deformation. The average values of modulus \( E \) and \( G \) for three specimens trimmed out of each sample were obtained. The consistency of the test results appeared satisfactory, since the coefficient of variation with respect to those parameters was less than 0.19 (See Table 1).

The undrained shear strength was normalized with respect to the effective overburden pressure and plotted against depth as shown in Fig. 4. The normalized strength parameter (NSP), \( c_u/\sigma_{v0}' \approx 0.5 \), for OCR = 1 is higher than common values of the NSP for Tokyo Bay alluvium, i.e. \( c_u/\sigma_{v0}' = 0.25 - 0.30 \), obtained from the results of unconfined compression tests.

The cohesive soil along the coast of Tokyo Bay often contain thin sand pipes or laminations with fragments of shells. The undrained shear strength obtained for such cohesive soils is thought to be underestimated by unconfined compression tests. A trend increasing NSP with OCR suggested by Ladd and Foot (1974) is recognized as shown in Fig. 5, where the plotted points are scattered, probably because of the variation of OCR with depth as shown in Fig. 4. Random distribution of thin sand laminations and shells may have also become sources of error.

The SBP Test: The SBP tests were carried out in three boreholes drilled at an interval of 2 m each other. The self-boring instrument penetrated to a desired depth was kept immovable until the pore pressure built up during penetration dissipated. It took about two hours until excess pore pressure became less than 5 kPa. After the measurement of effective horizontal stress in situ, the probe was inflated by pressure applied in steps. Each increment of pressure remained constant for 2 minutes in order to compare the results of the SBP tests to those of Menard pressuremeter tests. From a view point to achieve undrained deformation of soil around the probe, faster increase of pressure is

![Diagram](image_url)

(a) test results at the depth of 9 m  
(b) test results at the depth of 13 m

Fig. 6. Results of the SBP test in comparison with the MP test, Komatsugawa, Tokyo
desirable. The SBP test can be completed within 20 minutes or less by remaining pressure of each step for one minute.

Typical results of pressuremeter tests are illustrated by conventional pressure-volume curves as shown in Fig. 6(a) and (b). In those figures the results obtained from the Menard pressuremeter are compared to the results of SBP tests. The curves of both SBP and MP tests asymptote each other for the pressure close to the limit pressure, but the slope of curves of the SBP test within yield pressure is obviously different from that obtained from the MP tests. From initial tangent to the pressure-volume curves obtained from the SBP test, the pressuremeter modulus, \( E_p \), was calculated by the equation,

\[
E_p = 2(1 + \nu)\left(\frac{dp}{dv}\right)_{p = \text{yield}} = 2(1 + \nu)G
\]

(1)

\( v_0 \) : initial volume of prove (use \( v_m = (v_0 + v_f)/2 \) for Menard pressuremeter, where \( v_f \) : volume corresponding yield pressure)

\( \nu \) : Poisson’s ratio; assumed 0.5 for undrained tests

\( G \) : shear modulus assuming elasticity for small strain

The modulus \( E_p \) was also calculated from the curves of the MP tests using an average slope of a curve within the yield pressure. In Fig. 7 the moduli obtained from the SBP tests or the MP tests are plotted against depth. The open circles represent the mean values of the modulus \( E_p \) obtained from the SBP tests in 3 times at each depth. The maximum and minimum values are also shown in both sides of the open circles. The mean values of the modulus obtained from the MP tests in twice are shown by closed circles. It is noted that the moduli obtained from the SBP tests are 2 to 3 times greater than those from the MP tests. The deviation of moduli from a mean value appears a little larger for the SBP tests than that for the MP test. But they can not be compared directly as the number of the MP tests at the same depth was 2, while the SBP test was done in 3 times.

The limit pressure obtained from the SBP tests was plotted against depth in Fig. 8, and compared to the results of the MP tests. There was no remarkable difference in the limit pressure between the SBP and the MP tests. In Table 2 are summarized the parameters obtained from the SBP tests. It is noted from the coefficients of variation in Table 2 that the parameters such as pressuremeter modulus, \( E_p \), limit pressure, \( p_l \),
Table 2. Parameters obtained from the SBP-tests, Komatsugawa, Tokyo

<table>
<thead>
<tr>
<th>depth (m)</th>
<th>$E_s$ (MPa)</th>
<th>$E_s'$ (MPa)</th>
<th>$\mu$</th>
<th>$\bar{p}_f$ (kPa)</th>
<th>$\bar{p}_f'$ (kPa)</th>
<th>$\mu$</th>
<th>$\bar{p}_f$ (kPa)</th>
<th>$\bar{p}_f'$ (kPa)</th>
<th>$\mu$</th>
</tr>
</thead>
<tbody>
<tr>
<td>9.00</td>
<td>10.80</td>
<td>11.56</td>
<td>0.096</td>
<td>151</td>
<td>151</td>
<td>0.099</td>
<td>346</td>
<td>336</td>
<td>0.023</td>
</tr>
<tr>
<td>12.12</td>
<td>12.75</td>
<td>12.88</td>
<td>0.088</td>
<td>138</td>
<td>139</td>
<td>0.101</td>
<td>325</td>
<td>322</td>
<td>0.101</td>
</tr>
<tr>
<td>10.75</td>
<td>14.36</td>
<td>14.66</td>
<td>0.087</td>
<td>128</td>
<td>128</td>
<td>0.089</td>
<td>314</td>
<td>317</td>
<td>0.101</td>
</tr>
<tr>
<td>11.00</td>
<td>15.37</td>
<td>15.17</td>
<td>0.103</td>
<td>125</td>
<td>125</td>
<td>0.100</td>
<td>302</td>
<td>301</td>
<td>0.101</td>
</tr>
<tr>
<td>12.41</td>
<td>16.52</td>
<td>16.72</td>
<td>0.071</td>
<td>138</td>
<td>146</td>
<td>0.100</td>
<td>312</td>
<td>315</td>
<td>0.028</td>
</tr>
<tr>
<td>13.00</td>
<td>18.21</td>
<td>18.21</td>
<td>0.071</td>
<td>136</td>
<td>146</td>
<td>0.100</td>
<td>312</td>
<td>315</td>
<td>0.028</td>
</tr>
<tr>
<td>15.00</td>
<td>21.06</td>
<td>19.91</td>
<td>0.080</td>
<td>120</td>
<td>146</td>
<td>0.100</td>
<td>312</td>
<td>315</td>
<td>0.028</td>
</tr>
<tr>
<td>17.35</td>
<td>23.24</td>
<td>22.97</td>
<td>0.060</td>
<td>247</td>
<td>243</td>
<td>0.067</td>
<td>590</td>
<td>573</td>
<td>0.022</td>
</tr>
</tbody>
</table>

**Remarks:**
- $E_s$: average pressuremeter modulus
- $\bar{p}_f$: average limit pressure
- $\bar{\epsilon}_f$: undrained shear strength calculated by Eq. (5)
- $\mu$: coefficient of variation

and yield pressure, $p_f$, are sufficiently reproducible. The undrained shear strength, $\bar{\epsilon}_f$, shown in the rightmost column of Table 2 will be explained later.

The reason why the pressuremeter modulus obtained from the SBP test was 2 to 3 times higher than that from the MP tests, but the limit pressure from both the SBP and the MP tests are identical can be explained by assuming that disturbance of soil caused by boring is limited to a thin zone neighboring to the wall of a borehole. The deformation modulus was seriously influenced by the thin disturbed zone, but the limit pressure was not affected by it because the soil mass mobilized against the limit pressure was several times greater in diameter than a borehole, and this large soil mass was possibly free from disturbance caused by boring.

**Stress-Strain Curves:** From the results of the SBP tests, stress-strain curves of an element in contact to the pressuremeter probe were produced applying the procedure proposed by Palmer (1972).

The deviatoric stress, $\sigma_r - \sigma_z$, for an element in contact to the probe was obtained by the equation:

$$\sigma_r - \sigma_z = \frac{dp}{d[ln(D/v/v)]}$$  \hspace{1cm} (2)

where $p$: pressure applied by the probe

$D/v/v$: relative increment of the volume of a cylindrical cavity where $v = v_0 + \Delta v$

$v_0$: initial volume of a cylindrical cavity

The circumferential strain, $\epsilon_\theta$, was computed from the equation:

$$\epsilon_\theta_{r=r_0} = (1 - D/v/v)^{-1/3} - 1$$  \hspace{1cm} (3)

where $r_0$: initial radius of cavity

The shear modulus, $G$, was calculated from pressure versus circumferential strain curves by the equation:
\[ G = \frac{d\tau}{d\gamma} = \frac{d\tau}{2d\varepsilon_\theta} = \frac{1}{2} \left( \frac{dp}{d\varepsilon_\theta} \right)_{\varepsilon_\theta=0} \]  

\( \tau \): shear stress
\( \gamma \): shear strain

Eq. (4) is similar to the equation derived by Palmer, but the shear modulus is given as a derivative of shear stress with respect to shear strain which is twice as much as circumferential strain.

Typical stress-strain curves are illustrated in Fig. 9(a) and (b). The stress-strain curves were either plastic or strain hardening beyond the yield point. For the curves showing strain hardening, the undrained shear strength was determined from the deviatoric stress against the circumferential strain of 10 percent. The undrained shear strength, \( \tau_{\text{max}} \).

![Graphs showing stress-strain curves](image)

(a) at a depth of 13m  
(b) at a depth of 19m

Fig. 9. Stress-strain curves obtained from the SBP tests

<table>
<thead>
<tr>
<th>Depth (m)</th>
<th>( G ) (MPa)</th>
<th>( \bar{G} ) (MPa)</th>
<th>( \mu )</th>
<th>( \tau_{\text{max}} ) (kPa)</th>
<th>( \bar{\tau}_{\text{max}} ) (kPa)</th>
<th>( \mu )</th>
</tr>
</thead>
<tbody>
<tr>
<td>9.00</td>
<td>4.55</td>
<td>2.70</td>
<td>3.78</td>
<td>3.68</td>
<td>69</td>
<td>70</td>
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<tr>
<td></td>
<td>4.15</td>
<td>4.20</td>
<td>4.68</td>
<td>4.01</td>
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<td>71</td>
</tr>
<tr>
<td>11.00</td>
<td>4.95</td>
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<td>6.05</td>
<td>4.97</td>
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<td>61</td>
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<td></td>
<td>3.55</td>
<td>5.08</td>
<td>5.70</td>
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<td>71</td>
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<tr>
<td>15.00</td>
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<td>6.30</td>
<td>5.70</td>
<td>6.55</td>
<td>70</td>
<td>80</td>
</tr>
<tr>
<td>17.00</td>
<td>8.95</td>
<td>8.75</td>
<td>7.48</td>
<td>8.06</td>
<td>105</td>
<td>114</td>
</tr>
</tbody>
</table>

\( \mu \): coefficient of variation  
\( \bar{G} \): average modulus \( G \)  
\( \tau_{\text{max}} \): average shear strength

Table 3. Parameters computed from stress-strain curves, Komatsugawa, Tokyo
and shear modulus are summarized in Table 3. Comparing the average undrained shear strength, \( \tau_{\text{max}} \), in Table 3 to the average \( c_u \) in Table 1, it is noted that the shear strength obtained from the SBP tests is well correlated with that obtained from CU triaxial tests.

Gibson and Anderson (1961) derived the following equation assuming that soil is ideally plastic beyond the yield point:

\[
p_t = p_0 + c_u [1 + l_n(E/2c_u(1+\nu))]
\]

(5)

where

- \( p_t \): limit pressure
- \( p_0 \): horizontal stress in situ
- \( c_u \): undrained shear strength
- \( E \): Young's modulus
- \( \nu \): Poisson's ratio

Undrained shear strength calculated by Eq. (5) is listed in the rightest column of Table 2. Comparing those values to the values of the average \( \tau_{\text{max}} \) obtained from the stress-strain curves shown in Table 3, it is noted that the shear strength obtained from stress-strain curves is 1.2 to 1.4 times of that calculated by Eq. (5).

The shear moduli obtained from either the SBP tests or triaxial tests are summarizingly illustrated in Fig. 10. The shear moduli calculated by Eq. (1) agreed with those obtained from stress-strain curves (Eq. (4)). The moduli obtained from the SBP test were a little smaller than those obtained from the CU-test for depths less than 13 m, and a little larger for depths more than 15 m. The moduli obtained from unconfined compression tests are much lower than the moduli obtained from the CU-triaxial tests or the SBP tests.

The undrained shear strength obtained from different methods of calculation or testing is compared in Fig. 11. The undrained shear strength calculated by Eq. (5) using the \( p-\nu \) curves is about 70 to 80 percent of the strength obtained from stress-strain curves, which was in good agreement with the strength obtained from the CU-test. Large difference of undrained shear strength between CU-tests and unconfined compression tests can be explained by the fact that the strength obtained by unconfined compression tests is under-estimated because of irregularly developed sand laminations.
In-Situ $K_0$ Coefficient: The in-situ $K_0$ coefficient calculated from the results of the SBP test was plotted against depth in Fig. 12. The $K_0$-coefficient was also estimated by the empirical formula proposed by Massarsch (1979) using the plastic limit of cohesive soil. Massarsch's formula can be applied to normally consolidated clay. The cohesive soil at the site of Komatsugawa was lightly overconsolidated, but its influence on the $K_0$-coefficient was negligible, since the OCR was less than 2. The $K_0$-coefficient measured by the SBP test was well correlated with the value estimated by the empirical rule, although the value at 17 m was low. For the measurement of the $K_0$-coefficient, it is very important to remove air bubble completely before penetrating the pressuremeter probe.

TESTS IN DENSE SAND

Soil Conditions: The site was selected in an area located in the outskirts of Chiba City. The site was covered with plastic clay of volcanic origin called Kanto Loam, underlain with medium to dense sand of the Pleistocene epoch. The soil profile near the test site is shown in Fig. 13. The sand contained 8 to 10 percent fine grained soil passing through No. 200 sieve, which consisted of ferrous compounds binding granular soil. The ground water table measured by a piezometer were about 11 m below the ground surface.
According to triaxial shear tests, the shear strength parameters, $\phi'$, and $c'$, from depth 9 to 18 m was as follows:

$$\phi' = 35' - 37'$$
$$c' = 0 - 11 \text{ kPa}$$

**The Pressuremeter Test:** The SBP tests were carried out in three boreholes shown as No.1, No.2 and No.3 in Fig.14. Three other boreholes, No.1', No.2' and No.3' were drilled for the MP tests. The SBP test was first attempted in the borehole No.1 using the instrument of A-type. The membrane of the A-type instrument was collapsed while it was pushed to the depth of 12 m. Moreover the instrument could not be penetrated by the force generated by a hydraulic power of a drill rig. The testing in the hole No.1 was abandoned, and the tests were continued in the hole No.2 using the instrument of B-type. The instrument was pushed by a hydraulic jack having the capacity of 100 kN, transmitting its power through the 65 mm outer pipe. The instrument was penetrated smoothly by the vertical forces 20 kN or less without collapsing the membrane. The hole No.3 was used for the SBP tests below 12 m deep which could not be achieved in the hole No.1.

The modulus of deformation obtained from the SBP tests is plotted against depth in Fig.15, in which the parameters obtained from the MP tests are also illustrated comparatively. An open circle indicates the average value of the parameters obtained from the SBP tests in twice at each depth. Similarly a closed circle represents the average value obtained from the MP tests in three times at each depth. It appears that the moduli of deformation obtained from the SBP tests are, in general, 1.7 to 2.2 times greater than those of the MP tests. No clear difference in the reproducibility of test data is perceived between these two methods. The range of the maximum and minimum values appears to be governed not by the reproducibility of the modulus but by non-uniform distribution of the stiffness in situ.

The distribution of yield pressure and limit pressure with depth is illustrated in Fig.16 and Fig.17 respectively. The scatter of data in these figures are much less than the
data of the E-modulus. The values of yield or limit pressure for the SBP tests are practically same as those for the MP tests. The disturbance of cemented sand by boring is supposed to be limited to a thin zone in the vicinity of a borehole.

**TESTS IN LOOSE SAND**

*Soil Conditions:* The test site was selected in a reclaimed area, named Saginuma, in the northern coast of Tokyo Bay. The site was covered with fill of silty sand to the depth more than 8 m as illustrated in Fig. 18. Sand contained less than 20 percent of fine grained soil, but laminations or lenses of cohesive soil were irregularly inserted as shown at the depth of 5 to 6 m in Fig. 18. The ground water table was less than one meter deep. The soil at this site was being improved by the dynamic consolidation method in order to prevent liquefaction of loose sand during earthquake.

*The Pressuremeter Test:* The SBP test was carried out in two boreholes in the original ground, and two other boreholes in the ground improved by the dynamic consolidation method. The MP test was also carried out in four boreholes: two in the original ground and two in the improved ground.

The pressuremeter moduli obtained from the SBP test and the MP test are plotted against depth as shown in Fig. 19(a) and (b). In Fig. 19(a) are shown the moduli for the original ground, and in Fig. 19(b) for the improved ground. The moduli obtained from the SBP test are more widely scattered than those obtained from the MP test. The SBP modulus appears more sensitive to variation in deformability of soil than the MP modulus. The SBP modulus is 2 to 7 times greater than the MP modulus for the improved ground. It is noted that the modulus of deformation of loose sand may be very much under-estimated by the MP test.

In Fig. 20 (a) and (b) is shown the limit pressure obtained from the SBP test and MP test; (a) for the original ground and (b) for the improved ground. It is noted that the limit pressure obtained from the SBP test is considerably higher than that obtained from the MP test for the depth 1 to 4 m in the improved ground. It is probably due to the fact that the sand densified by improvement was loosened by opening a hole in it and leaving the hole unsupported. Soil disturbance caused by boring in sand seems not limited to a thin disturbed zone around a borehole, but a fairly large mass of sand...
around a borehole is supposed to be loosened.
In Fig. 21, the limit pressure obtained from the SBP test is compared between the original ground and the improved ground. Remarkable improvement was evidenced to the depth of 4 m, and soil was improved to 7 m deep aside from a depth of 5 m where soil was cohesive.

CONCLUSION
The conclusive results of the studies presented in this paper are summarized as follows:
(1) The self-boring pressuremeter has advantage that it can be used for a variety of
soil as soft clay, loose sand, stiff clay and dense sand, and it can reduce the influence of soil disturbance caused by boring.

(2) The moduli of deformation of soft cohesive soil obtained by the SBP were 2 to 3 times greater than those obtained from the MP test.

(3) There was no significant difference in the limit pressure of soft cohesive soil between that obtained from the SBP tests and the MP tests. Disturbance of soil caused by boring can be assumed limited to a thin zone along the wall of a borehole. The modulus of deformation was affected by the thin disturbed zone, but the limit pressure was not affected, because the soil mass mobilized against the limit pressure was several times greater in diameter than a borehole. This large soil mass was possibly free from disturbance caused by boring.

(4) The shear moduli calculated from the $p-u$ curves of the SBP test in alluvial cohesive soil were coincident to the moduli obtained from CU-triaxial tests. The shear moduli obtained from stress-strain curves of an element in contact to the probe were also well correlated to the moduli calculated from the $p-u$ curves. The moduli obtained from unconfined compression tests were much lower than the moduli obtained from the CU triaxial tests or the SBP tests.

(5) The undrained shear strength calculated by the equation of Gibson and Anderson was about 70 to 80 percent of the strength obtained from stress-strain curves. Therefore those two methods of calculation yielded practically similar results. The undrained shear strength obtained from the SBP test was well correlated to that obtained from CU triaxial tests. The undrained shear strength obtained by unconfined compression tests were considerably lower than that obtained from the SBP tests.

(6) The in-situ $K_s$ coefficient obtained from the SBP test in soft cohesive soil was well correlated to the empirical rule of Massarsch.

(7) The moduli of deformation obtained from the SBP test in dense sand of the Pleistocene epoch were, in general, 1.7 to 2.2 times greater than those obtained from the MP test.

(8) The values of yield or limit pressure of dense cemented sand were practically same for the SBP test and the MP tests. Disturbance of cemented sand by boring was supposed to be limited to a thin zone in the vicinity of a borehole.

(9) The moduli of deformation obtained from the SBP tests in loose sand were 2 to 7 times greater than those obtained from the MP test.

(10) Not only the modulus of deformation but the limit pressure in loose sand was considerably different in the results between the SBP test and the MP test.

(11) Further study in the future is needed with respect to the following subjects:

(i) The in-situ $K_s$ coefficient for granular soils measured by the SBP test was scattered even at the same depth. The source of variation might be the heterogeneity of ground, but research on this subject is needed.

(ii) The consolidation of cohesive soil under a constant pressure is not included.
in the studies presented in this paper. The study on the consolidation or creep phenomena of soil around a self-boring pressuremeter is needed in the future.

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NOTATION

c' = effective cohesion intercept
CH = highly plastic clay
CL = lean clay
CU = consolidated undrained
\( c_u \) = undrained shear strength
\( E \) = Young's modulus
\( E_p \) = pressuremeter modulus
\( E_{50} \) = secant modulus
\( G \) = shear modulus
\( K_0 \) = coefficient of earth pressure at rest
MP = Menard pressuremeter
\( N \) = blow count of the SPT
NSP = normalized strength parameter
OCR = overconsolidation ratio
\( \beta \) = pressure
\( p_f \) = yield pressure by pressuremeter
\( p_l \) = limit pressure by pressuremeter
\( p_s \) = static pressure by pressuremeter
\( r \) = radius
\( r_0 \) = radius of a borehole
SBP = self-boring pressuremeter
\( U \) = undrained (unconfined)
\( v \) = volume
\( v_i \) = initial volume
\( \tau \) = shear strain
\( \varepsilon_0 \) = circumferential strain
\( \mu \) = coefficient of variation
\( \nu \) = Poisson's ratio
\( \sigma_h' \) = effective horizontal stress
\( \sigma_n \) = standard deviation for population
\( \sigma_r \) = radial stress
\( \sigma_{ob} \) = effective overburden pressure
\( \sigma_\theta \) = tangential stress
\( \tau \) = shear stress
\( \tau_{max} \) = maximum shear stress
\( \phi' \) = effective angle of internal friction

REFERENCES


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