Mechanical Behavior of Soft Marine Silts under Nearshore Structures

Lien-kwei Chien    Tsung-shen Feng    Wen-Chien Tseng    Tsung-Ching Chen

ABSTRACT

Taiwan is restricted by the geographic condition at near shore area and request of national economy development. The site of the coastal structure has been become very difficultly to evaluate and to obtain the suitable land space to build. From the viewpoint of coastal geotechnical engineering practice, the soft ground of seabed has without enough bearing capacity or induced excessive settlement, and inclined to the structures damage etc. Therefore how to deep understand the mechanical behavior of soft marine silts will be one of the important investigations of the nearshore geotechnical engineering topic.

In this study, the related literatures and basic environment data were collected and analyzed to understand the properties of site regional state. From the microstructure analysis, the chemical composition construct of soft marine silts belong to the granite weathering and deposition. And the fabric network is described in terms of inter-aggregate and intra-aggregate porosity and will be obtained lower shear strength. On the other hand, jointed with the analysis of laboratory experiment and comparing with related research results and experience equations, the correctness of the test result is verified. And then, the test result feedback into the development of numerical simulation to make numerical analysis and to assure rationality and quality.

Based on linear-elasticity, Modified Cam-Clay and consolidation theory, the model of settlement and stability was established. Finally, the mode results could be provided to estimate settlement and have deep finding out the real damage reason and mechanism of the “Matsu Harbor of Fu-ao region extend construction for breakwater”. It will conduct disaster prevention, stability under nearshore structure, and could be useful for the reference of the following repair engineering and estimate the settlement stability.

1. INTRODUCTION

In harbor engineering, the construction environment is changeable, and the factors such as the sea, climates, waves, tides, currents and geological structures are complex. Particularly, the cultural, social and economic environments may greatly affect the engineering. However, shore structures have physical appearance, thus easily become the center of focus. The geotechnical site investigation and testing after construction are the intangible parts of engineering, so their importance is often ignored by people. However, the work often affects the success of the engineering. According to previous experiences, the analysis and predicted results for the settlement of soft marine silts are not ideal. Besides the inadequate analysis of data and insufficient factors considered in construction, the main reason is that the selected soil parameters and the analysis model of stabilized settlement are not consistent with the existing circumstances.

During the construction of Baisha Port of Beigan, Matzu in 1995, it was found that the marine silts had been serious and the structure of the rubble mound breakwater had settled gradually. Furthermore, the silt layer at the front section of rubble mound breakwater had upheaved obviously. When the breakwater at Fuao Commercial Port of
Nangan, Matzu was expanded in 2002, the serious settlement and lateral displacement appeared during construction. Meanwhile, marine silts caused foundation settlement of nearshore structures due to inferior nature of works, and the settlement quantity is the variable which changes with the time, storm waves and unpredicted factors, thus is the main reason to bring the disasters to the shore and harbor engineering.

In order to further discuss the disasters caused by marine silts, this study chose the ongoing expansion of breakwaters at Fuao Port of Matzu as the research object, and conducted analysis of relevant indoor static experiment, numerical simulation analysis of settlement stability to further understand factors causing breakage of rubble mound breakwaters and the behavior of such factors. It also provided reference for stabilization of nearshore structure and prevention of disasters.

2. METHODOLOGY

This study planned a series of experiments, including one-dimensional consolidation experiment, and triaxial mechanical experiment. Firstly, the stack mold preparation method was established for soft marine clayey silt specimen according to the saturated clay density depending on water content, in order to understand the soil property through parameters related to compression, consolidation and shear force. This study also verified the previous experimental results, to further understand the characteristics of silt parameters and find better parameter estimation method to provide experience and correct judgment for selection of soil parameters when analyzing stability of settlement in the future.

Geo-slope office (University of Calgary, Canada) was applied in this study to simulate rigid foundations embedded in soft isotropic silts for discussion of settlement of rubble foundations and analysis of breakage mechanism, as well as stress and strain behaviors. The analysis of program architecture and capability, definition of boundary conditions and acquisition of soil parameters can be conducted by collecting data from domestic and foreign literature. Furthermore, the accepted range was evaluated for engineering practice to simulate the program architecture and estimate analysis capability of development.

Besides discussion of settlement quantity of soil foundations for soft marine clay, it was necessary to comprehensively analyze soil bearing capacity and mechanism related to slip and breakage of the foundation. The limit equilibrium method was applied to evaluate the stability of foundation at soft ground. The main hypothesis was that the range of slipped soils has reached the limit state. The critical slip surface and the corresponding safety factor of soil foundation and relevant stress and strain behaviors of the interior slip surface can be found through critical slip surface of minimum safety factors. The analysis results were applied in the calculation of stability to obtain the whole safety factor and predict the possible breakage mechanism.

3. ANALYSIS OF RESEARCH AREA

3.1 Location of research area

Matzu Archipelago is located northwestward by west of the Taiwan Strait, facing the river mouths of Minchiang (Min River), Lienchiang (Lien River), and Luoyuan Bay. This archipelago is situated between 25°56′ to 26°18′ north latitude and 119°51′ to 120°01′ east longitude, extending 54 nautical miles.
Matzu Archipelago is 114 nautical miles (221 km) away from Keelung, Tungying Island, nearest to Taiwan is 90 nautical miles (167 km) away from Keelung, 156 nautical miles (282 km) away from Nangan in the southwest, 180 nautical miles (333 km) away from Penghu in the south, 132 nautical miles (244 km) from Taichung; Gaodeng Island is nearest to Beijiao Peninsula, only 9.25 km, facing Benji Mt, Yantung Mt. and Shamao Mt.

3.2 Analysis of geological and marine meteorological data

Rubble mound breakwaters play an important role in protecting the stability of hinterlands and port basins. Therefore, the selection of marine meteorological conditions is very important, such as tide and waves, are the key factors considered in the design of shore structures.

The waves in this area can be divided into seasonal waves and hurricane waves. The seasonal waves were estimated on the basis of Breschneider (1984). The estimated results related to high sea seasonal waves are shown in Table 1.

### Table 1 Estimated results of monsoon waves in Matzu

<table>
<thead>
<tr>
<th>Wave direction</th>
<th>Maximum wind speed (m/s)</th>
<th>Wave height (m)</th>
<th>Cycle (s)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Six grade</td>
<td>13.8</td>
<td>1.20</td>
<td>4.10</td>
</tr>
<tr>
<td>Seven grade</td>
<td>17.1</td>
<td>1.52</td>
<td>4.60</td>
</tr>
<tr>
<td>Eight grade</td>
<td>20.7</td>
<td>1.88</td>
<td>5.10</td>
</tr>
<tr>
<td>Nine grade</td>
<td>24.4</td>
<td>2.24</td>
<td>5.50</td>
</tr>
</tbody>
</table>

Due to higher height of hurricane waves, the waves have close relation with the stability of shore structures. Hurricane waves were estimated on the basis of the different assumed regression cycles of hurricane models, the estimated results of hurricane waves in high sea in Matzu are shown in Table 2.

### Table 2 Estimated results of hurricane waves in high sea in Matzu

<table>
<thead>
<tr>
<th>Wave direction</th>
<th>NNE</th>
<th>N</th>
<th>NNW</th>
<th>NW</th>
<th>WNW</th>
<th>W</th>
<th>WSW</th>
</tr>
</thead>
<tbody>
<tr>
<td>Wave height (m)</td>
<td>3.13</td>
<td>4.75</td>
<td>4.00</td>
<td>4.25</td>
<td>4.49</td>
<td>4.65</td>
<td>4.89</td>
</tr>
<tr>
<td>Cycle (s)</td>
<td>5.84</td>
<td>7.23</td>
<td>6.33</td>
<td>6.92</td>
<td>7.30</td>
<td>7.64</td>
<td>7.65</td>
</tr>
</tbody>
</table>

Note: recurrence cycle is 50 years, the estimated point is at the depth of 20 m in water.

3.3 Date related to offshore drilling on site

This foundation ground is located between seabed surface and the seabed below at the depth of 20.45 m, and can be divided into four layers from upper to bottom, including silty clay layer (located between seabed surface and the seabed below at the depth of 4.5~17.2 m, N-value in standard penetration test was 1~2), silty clay layer with fine-sand silt or medium fine sands (seabed below at the depth of 12.3~16.8 m, N-value was 1~2), silt containing stone layer or stones containing silt layer (from this layer to pore bottom (depth was 20.45 m), this part belonged to this layer (clast), N-value was over 50).
4. EXPERIMENTAL METHODS AND RESULT ANALYSIS

4.1 Basic physical properties of soil

The particle size distribution curve (as shown in Fig 1) was plotted after testing and analysis of seabed soils at five different locations in the same area. According to the results, the content of gravels and sand soils was approximately 2%, silt content was approximately 45%, and clay content was approximately 53%.

The Atterberg limit test showed that, the liquid limit (LL) of seabed soils at five different locations was approximately 42~47%, plastic limit, (PL) was approximately 26~31%, and plasticity index (PI) was 12~20%.

According to XRD diffraction results of soft marine silts in Fuao, Matzu and the possible elements judged by EDS, the chemical compositions of soft marine silts were mainly $\text{SiO}_2$ and $\text{Al}_2\text{O}_3$ which content was 78.1% and 9.1% respectively, and the minor compositions were $K_2\text{O}$, $\text{MgO}$, $\text{Fe}_2\text{O}_3$, $\text{FeO}$, $\text{Na}_2\text{O}$, and $\text{CaO}$, which content was 4.3%, 2.6%, 2.2%, 2.1%, 1.2% and 0.4% respectively. The results related to X ray diffraction test of soft silts are shown in Fig 2.

4.2 Shear strength experiment and result analysis

C.K.C. Automatic Triaxial Test System was applied in this study, as shown in Fig 3. The characteristic of this system is the way of stress controlled, strain controlled and any stress path controlled. According to stress controlled or strain controlled, air pressure and oil pressure can be adopted for the pressure system in the experiment. All the experimental procedures were performed through human-computer dialogues, providing an easy and convenient window operating environment.

As for the normal consolidation clay, the failure envelopes based on the strength parameters calculated without conducting consolidation and drainage experiments were horizontal. Therefore, the non-drainage shear strength according to half of maximum axial differential stress $(\sigma_1 - \sigma_3)_{\max}$ could be determined. When axial strain exceeded 15%, but not reached the maximum axial differential stress, the non-drainage shear strength can be defined according to 15% of axial differential stress.

Since Terzaghi (1925) proposed the concept of effective stress, it is universally considered that soil shear strength is controlled by effective stress (without exception of clay), so that Su can derive the effective stress parameters. Leonards (1962) proposed that Su of shear strength can be calculated through effective stress parameters when conducting compaction and non-drainage experiments. The equations are written as follows:

$$S_w = \frac{c' \cos \phi' + \sin \phi' \left[ K_u + A_f \left( 1 - K_0 \right) \right]}{1 + \left( 2A_f - 1 \right) \sin \phi'}$$

(1)

Normal consolidation clay $c' = 0$, so (1) can be reduced to

$$\sigma'_w = \frac{\sin \phi' \left[ K_u + (1 - K_0) \right]}{1 + \left( 2A_f - 1 \right) \sin \phi'}$$

(2)

where, $c'$, $\phi'$: effective shear strength parameters $A_f$: pore water pressure during breakage $\sigma'_w$: effective vertical compaction stress $K_0$: coefficient of earth pressure at rest

For $K_0$ soil compaction theory, Liao and Su (1994) derived a formula on the shear strength relation between CIU and CK$_0$U:
\[
\left(\frac{S_c}{\sigma_{cv}}\right)_\text{CIU} = \frac{K_s + 2(1-K_s)M}{K_s + 2(1-K_s)M_s} \left[A_f (1-K_s) + K_s\right]
\]

Where, \(A_f\): CIU pore water pressure during breakage of tested soils (Skempton, 1954)

\(A_f\): CK\(U\) pore water pressure during breakage of tested soils (Skempton, 1954)

4.3 Isotropic consolidation experiment results

The results of soft marine silts experiment showed the total stress and effective stress path in Fig 6 and 7. Under the different isotropic consolidation stresses, final stress path and the failure lines approached to consistency. Furthermore, all the failure lines passed origin.

As for the total stress strength (short-term), the parameters of effective stress strength (long-term) were smaller, \(\phi\) was approximately half of \(\phi'\), and this is consistent with the defined result of normal consolidation clay mentioned by Holtz and Kovacs (1981).

4.4 K0 consolidation experiment result

Due to anisotropy, only the intrinsic anisotropy produced by particle arrangement can be considered in the traditional CIU experiment, and the anisotropy produced by K\(0\) consolidation (\(K_0 \neq 1\)) cannot be responded. Furthermore, anisotropic consolidation may affect the evaluation of shear parameters related to soft marine silts (Mayne, 1985). The basic mechanical behavior under K\(0\) consolidation is discussed below.

According to the experimental results of soft marine silt specimen, the total stress and effective stress path are shown in Fig 8 and 9. Under the different K\(0\) consolidation stresses, the initial value of the stress path was in the stress state after K\(0\) consolidation. Under the different initial densities, the final stress and the failure lines approached to consistency. Furthermore, all the failure lines passed origin.

4.5 Comparison of results of isotropic consolidation and K\(0\) consolidation

The \(c\) and \(c'\) values of the soft marine silts is 0 kPa because during the settlement of soft marine silts the silts cannot bear shear just like water, and therefore \(c = 0\). When overlapping with other silts, the effective shear strength was not equal to zero, in the discussion of cohesiveness, all the other items except cementation had relation with effective stress, when the effective stress was equal to zero the other items cannot provide strength. When the particles closed to each other, the cement can fill the contact points and the effective stress was not equal to zero. Therefore, the soft marine silts may belong to normal consolidation soils.

In the experiment \(\phi\) and \(\phi'\) of soft marine silts lowered when the density of specimen lowered and its water content increased. However, when the initial dry density was 1.00 t/m\(^3\), \(\phi\) and \(\phi'\) was higher. It is possibly because the specimen with initial density of 1.00 t/m\(^3\) was very soft, during experiment, it may be disturbed due to its minuteness. According to Compression Shell proposed by Bishop and Henkel (1962), when considering the rubber mold and specimen were the different materials and they
borne the axial load together, the soft marine silts may cause the specimen to suffer the slight exterior stress due to expansion of rubber mold and modulus of compression. Therefore, the structure of loosen particles changed, and the angle of friction increased.

4.6 Settlement stability numerical simulation analysis.

The software SIGMA/W was applied to convert the research area into two dimension plane strain, and simulate stress-strain produced by stratification of rubble breakwaters. The stability of soil settlement through the change of finite element girds are discussed in the following.

In order to understand the influence of long time effect and excessive pore water pressure on soft marine silts, the settlement of soft marine silts caused by the breakwater at Fuao Port of Matzu was analyzed through consolidation analysis results and SEEP/W seepage analysis application program. The real time consolidation and settlement of seabed soils caused by rubble mound breakwaters were predicted.

4.7 Simple analysis of section and selection of parameters

In order to cooperate with numerical grids setting and for the convenience of numerical mode calculation, the stratigraphic section was rationally simplified before analysis. The thickness of soil layer in shore was 12m. With the rubble mound breakwater extending to high sea, the thickness of soft marine silts increased, at 0k+190m, the thickness of the whole soft marine silts become gentle, extending to high sea, and the thickness of the gentle sea bed was 18m. The original and simplified stratigraphic sections are shown in Fig 10.

After analysis of the breakwaters design standard and understanding of basic design, On the basis of stability of the slope gradient of breakwaters and economy, the slope gradient is 1: 2 for the side of sea and 1: 1.5 for the side of port. 0k+730m was the dividing line for the breakwater surface, the crest width (B) of nearshore and offshore is 18m and 21m respectively. For the convenience of numerical simulation analysis, the section of the rubble mound breakwater were divided into 7 parts (AA~GG). The section of the rubble mound breakwater is shown in Fig 11 (B is width of the breakwater, h is the depth of water and d is the thickness of soil layer).

After selecting the index according to the parameters of preceding mode, the soils were assumed as anisotropic materials. However, the soils kept linear elastic during simulation and the soils were simplified as the consolidation parameters of single soil layer. The relevant experimental parameters were obtained from experimental results. According to the mode requirement, the stratum condition and soils property parameters are shown in Table 3.
Table 3  Comprehensive generalization of selected parameters for mode and study the use of parameters

<table>
<thead>
<tr>
<th></th>
<th>Linear Elastic</th>
<th>Modified Cam-Clay</th>
<th>Consolidation</th>
</tr>
</thead>
<tbody>
<tr>
<td>$A$</td>
<td>(pore pressure parameter)</td>
<td>0.78&lt;sup&gt;#&lt;/sup&gt;</td>
<td>$C_v$ (compression index)</td>
</tr>
<tr>
<td>$B$</td>
<td>(pore pressure parameter)</td>
<td>1.00&lt;sup&gt;#&lt;/sup&gt;</td>
<td>$C_s$ (swelling index)</td>
</tr>
<tr>
<td>$E$</td>
<td>(elastic modulus)</td>
<td>5 MPa, 8 MPa* (Soft silt)</td>
<td>$\lambda$ (slope of normal consolidation line)</td>
</tr>
<tr>
<td>$\nu$</td>
<td>(poisson’s Ratio)</td>
<td>0.4 (Soft silt)</td>
<td>$\kappa$ (slope of swelling line)</td>
</tr>
<tr>
<td>$K_0$</td>
<td>(coefficient of earth pressure at rest)</td>
<td>0.60, 0.60*</td>
<td>$\phi'$ (effective friction angle)</td>
</tr>
<tr>
<td>$\gamma_i$</td>
<td>(bulk unit weight)</td>
<td>16.4, 16.7 kN/m&lt;sup&gt;3&lt;/sup&gt; (Soft silt)</td>
<td>$M$ (slope of critical state line)</td>
</tr>
<tr>
<td>$k$</td>
<td>(hydraulic conductivity)</td>
<td>$2.07 \times 10^{-4}$ m/day</td>
<td>$\Gamma$ (specific volume)</td>
</tr>
<tr>
<td>$m_v$</td>
<td>(coefficient of volume compressibility)</td>
<td>$3.70 \times 10^{-3}$ 1/kPa</td>
<td>OCR (over consolidation ratio)</td>
</tr>
<tr>
<td></td>
<td></td>
<td>$1.42 \times 10^{-4}$ m/day*</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>$3.65 \times 10^{-3}$ 1/kPa*</td>
<td></td>
</tr>
</tbody>
</table>

Note: 1. The condition of filling rubbles for breakwater meets port structures design standard issued by Ministry of Communications and parameters selected from numerical mode Jeng et al. (2001).
2.  "#" : parameters when initially forming analysis mode.
3.  "*" : mechanical parameters of soft silts which unit weight is 16.7 kN/m3.
Fig1. Particle size distribution curve of soils

Fig2. X-ray diffraction of marine soft silts (S-3).

Fig3. Schematic of automatic triaxial test system combination
Fig 4 $\Delta u-\varepsilon$ curve of isotropic consolidation without drainage

Fig 5 $\Delta u-\varepsilon$ curve of $K_0$ consolidation experiment without drainage

Fig 6 p–q relation of isotropic consolidation of different initial densities without drainage

Fig 7 p'–q' relation of isotropic consolidation of different initial densities without drainage

Fig 8 p–q relation of $K_0$ consolidation of different initial densities without drainage

Fig 9 p'–q' relation of $K_0$ consolidation of different initial densities without drainage

Fig 10 Profile of soil foundation at rubble mound breakwaters

Fig 11 Profile of Simplified Rubble Mound Breakwater
5. MODE PREDICTION AND RESULT DISCUSSION

5.1 Simulation results at different phases of stratified structure.

Seven sections of the rubble mound breakwater (AA–GG sections, as shown in Figure 11), with changed with the maximum settlement quantity under the construction. The settlement quantity of soft marine silts increased by degree with the increase of construction height (as shown in Fig 12); the seabed slightly upheaved during the construction of the toe of sea side (GG). After analysis, the largest settlement was located at not the core of the breakwater but near the toe of sea side (BB); in Fig 13 As seen, the whole soft silts were stable before construction height reached 6m, but when the construction height exceeded 9m, large-area soil layer displaced vertically and the seabed upheaved. The thickness of different seabed matched with the sections of breakwater. The settlement quantity increased with the increase of thickness (h) of soil layer and width (B) of the breakwater (see Fig 14). It can be concluded that the thinker soil layer may have larger settlement.

The soft soil layers of different sections displaced in the side direction due to extrusion caused by construction of the breakwater. The displacement at the side of port was more obvious than that at the sea side (see Fig 15). The behaviors of seabed under the different loads when filling rubbles was analyzed. With the increase of loads when filling rubbles, the soft marine silts at the breakwater displaced greatly, and then front toe of the breakwater may upheave due to extraction, the affected range of stress and strain was about 40 m, equivalent to 4 times of thickness of the soil layer. It can be predicted that when the side slope of soft marine silts suffered asymmetry loads, the bearing capacity of the soft silts cannot resist and the possible partial breakage mechanism and slip may occur.

5.2 Simulation result related to dissipation of pore water pressure

Fig 16 shows the relation between pore water pressure and time after completion of breakwater construction. As shown, when the breakwater construction was completed, larger excessive induced pore water pressure existed deeper seabed. The bottom structure was waterproof granite plate, so the drainage path of bottom silts was longer and the dissipation rate was affected; the upper part of breakwater was the materials with better water permeability, and the drainage path of surface silts was shorter, thus, the dissipation of pore water pressure can be finished at short time. Due to the smaller permeability coefficient of marine soft silts ( m/day estimated from experiment), the delay of low dissipation rate may be longer, and then the settlement rate appeared. Therefore, the dissipation of excessive pore water pressure may reach the stable state after long time.

With the increase of time, the excessive pore water pressure dissipated and the settlement quantity of stratum increased gradually. The main consolidation settlement only affected the breakwater bottom. After the initial completion of construction, it was found that the maximum excessive pore water pressure was induced near at the side of port near the toe of breakwater. According to the result related to the large settlement quantity of toe of breakwater at the side of port due to different construction phases, with the slow dissipation of excessive pore water pressure, the settlement of the slope near the toe of breakwater increased continuously until the settlement was stable and balanced. Therefore, the slope at the side of port may have maximum settlement quantity after initial analysis.
5.3 Situation results under different analysis mode configuration

Based on the same analysis, 3m thick linear elastomer at front section of extended layer of the breakwater was modified to replace the whole soft silts with modified dam-clay mode. Then the two configurations can be discussed according to the comparison of settlement stability results above.

Firstly, the soil layer was 18m before completion of the breakwater construction. When the layer was the modified cam-clay mode, the values cannot be calculated. Due to the soft marine silts failing to suffer load of rubble mound breakwater, the overlarge deformation produced and caused the numerical analysis divergence and grids in disorder (see Figure 17). When the soil layer was 12m thick, the differences of initial settlement quantity was larger for the whole real time consolidation settlement of soft silts at breakwater (difference was about 2.6m), the difference of seabed upheaval was about 10.6m, the difference of later consolidation settlement quantity was only 0.03m. It was observed that the horizontal side displacement at the side of sea and at the side of port, the difference of horizontal displacement was 10.5m (see Fig 18 and 19).

According to the comprehensive analysis, seabed upheaval and initial real-time settlement increased when changing observation mode configuration. Due to the larger horizontal side displacement, the upheaval of seabed before breakwater exceeded the sea surface. When comparing with the disaster occurred during construction, there were some errors. It explained the extrusion of surface soils under the loads of filling rubbles at the soft marine silts. The ideal result could be obtained through linear elastic mode, so that the rationality and integrality of numerical mode can be proved.

5.4 Simulation results of seabed upheaval in rubble filling schedule

When the mileage of filling rubbles on site reached 0k+250m, unreasonable settlement occurred. In order to evaluate the upheaval height caused by side displacement before the breakwater, the changes of seabed was calculated at an interval of 120 m when filling rubbles. The relevant stratum, mode configuration and landform before the breakwater are shown in Fig 20 and 21.

The results of the settlement of the soil layer at breakwater bottom were quite different from the previous results. It was because the two dimension plane strain was applied in the analysis. The section of rubble mound breakwater was assumed as half of infinite space, so the upheaval at the two sides cannot be simulated. This cause the results calculated from analysis mode inconsistent with in-situ situations. The mode for analysis of the progress of filling rubbles only provided the information related to landform changes and was not suitable for prediction of settlement of the whole breakwater.
Fig 14. Vertical settlement quantity of bottom soils at rubble mound breakwater under construction

Fig 15. Side displacement of the different sections of breakwater under construction

Fig 16. Dissipation of pore water pressure after long time
6. CONCLUSIONS

This study employed X diffraction to analyze the minerals and content ratio of marine silts. According to the possible element determine by EDS, this study inferred the chemical compositions of soft marine silts may be consistent with the compositions of granite in igneous rock. Furthermore, the higher content of $SiO_2$ was consistent with results (ML) related to soil classification. The SEM results showed that the distance was smaller in the black shadow pore and the pore between soil particles, thus proving that the silts had quite higher water content (Locat, 1996). The understanding of the relationship between microstructure of soils and engineering is a great innovation for engineering and research in the future.

The soft marine silts after isotopic consolidation which was controlled by confining pressure of specimen can bear deformation in the side direction, causing the pore water pressure was higher than that after K0 consolidation. The density after isotopic consolidation was tighter that that after K0 consolidation. According to the loading capacity rate of clay indicated by Holtz and Kovacs (1981), the shear strength may tend to be higher when the loading capacity rate became quicker. In this study, the
intrinsic anisotropy caused by soil arrangement was considered and anisotropic property 
\( K_0 \neq 1 \) and the original and actual stress and fabric structure of clay can be responded.

The experimental results showed the change of dry density after isotropic and \( K_0 \) consolidation was 1.70\%~16.0\% and 1.70\%~18.0\% respectively. The relation between water content of specimen and effective internal friction angle \( \phi' \) (Skempton and Sowa, 1963) was analyzed. The consolidation changed the fabric structure of the previous loosen soils to cause the increase of friction angle. This can explain the higher value of internal friction angle in soft marine silts.

\[
\phi' = -1.08 \times \omega_{\text{initial}} + 96.17 \\
\phi' = -0.65 \times \omega_{\text{consolidated}} + 64.59
\]

where, \( \phi' \), \( \omega_{\text{initial}} \) and \( \omega_{\text{consolidated}} \) are effective internal friction angle (\(^\circ\)), initial water content (\(\%\)) and the water content after consolidation (\(\%\)), respectively.

The rubble mound breakwater was not in the asymmetric shape. The center of gravity of the whole section inclined to the side of the sea, causing the side displacement of soft marine silts at the side of the port larger than that at the side of the sea. Furthermore, the fabric structure of soils at seabed became loosen. The excessive pore water pressure was induced at the side of port near the breakwater toe. The maximum settlement quantity may occur at the side of port (the calculated width angle variable of the breakwater top is between 0.011 and 0.0523).

REFERENCES


