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Improvement of a Quick Sand

L'Amélioration de Sable Boulants

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SYNOPSIS Quick sand, a special sand deposit which possesses a metastable structure, is characterized by potential liquefaction under static shear conditions. This paper describes the shear characteristics of a quick sand found at the proposed site for a breakwater in the Arabian Gulf. The paper further reports the change in shear characteristics of the quick sand as well as mechanical properties through soil improvement by the Dynamic Consolidation method.

INTRODUCTION

Quick sand, a very special sand deposit which possesses a metastable structure, is characterized by potential liquefaction failure under static shear conditions (Bjerrum et al. 1961). A very loose silty sand was found at the proposed site for a breakwater in the Arabian Gulf. Samples from this layer could seldom be obtained from the standard penetration tests. A sand sampler, a modified version of the Bishop Sampler was tried and succeeded in obtaining high quality undisturbed samples from this silty sand (Hanzawa et al. 1979). Ko-consolidated undrained triaxial tests were conducted on undisturbed samples and the test results indicated that the silty sand is classifiable as one kind of quick sand. Consequently, soil improvement by the Dynamic Consolidation method was carried out. In order to understand the efficiency of improvement, standard penetration tests, pressure meter tests and undisturbed sampling were carried out at a trial area. In addition, static and cyclic triaxial tests were also performed on undisturbed samples from the quick sand before and after improvement.

This paper first describes the undrained strength of the quick sand for practical use and then reports the change in shear characteristics of the quick sand through soil improvement by the Dynamic Consolidation method.

DETERMINATION OF UNDRAINED STRENGTH OF THE QUICK SAND FOR PRACTICAL USE

Typical soil conditions at the site are as indicated in Fig. 1. N blows showed low values (0-1) for the S2-2 layer. Undisturbed sampling by the Modified Bishop Sampler was tried and succeeded. Undisturbed samples from the S2-2 layer were put into the triaxial apparatus, anisotropically consolidated under the stress ratio, $\bar{\sigma}_{hc}/\bar{\sigma}_{vc}=0.5$ ($\bar{\sigma}_{hc}$ and $\bar{\sigma}_{vc}$ are the horizontal and vertical consolidation stresses in the triaxial test) and then subjected to shear by compression under constant volume conditions. Stress-strain-excess pore pressure curves obtained from these tests are indicated in Fig. 2 to-

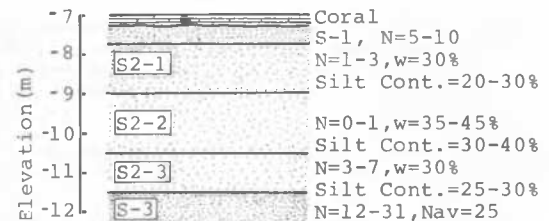


Fig. 1 Typical soil conditions at the site

gether with pore pressure parameter, A values with strain. As shown in the figure, the deviator stress reached a peak value at a very small axial strain ($\epsilon_1=0.2\%$) and then decreased with more strain value ($\epsilon_1=1\%$ to 1.5%). After this strain value was reached, it again increased. This is one of the undrained strength characteristics of quick sand classified as having limited liquefaction behaviour (Castro 1969). When the shear strength at the peak point (point P in the figure) is plotted versus $\bar{\sigma}_{vc}$, the following strength parameters were obtained:-

$$Su(c) = 10 + 0.31\bar{\sigma}_{vc} \quad \text{KN/m}^2 \quad (1)$$

$Su(c)$ = undrained compression strength

When the strength of the silty sand is expressed in the form as shown in eq.(1), the following two parameters should be investigated before strength values are applied for practical problems such as stability analyses

- (1) Anisotropy in undrained strength
- (2) Whether the strength at the P point can be used or not used because the deviator stress again increased at a relatively small axial strain value as shown in Fig. 2. If the same phenomenon takes place in practical problem, it would be too conservative to use the strength at the P point.

In order to investigate these two problems, 1) strain and stress controlled triaxial compression and extension tests, and 2) stress controlled simple shear tests were carried out on disturbed samples from the S2-2 layer. The test results

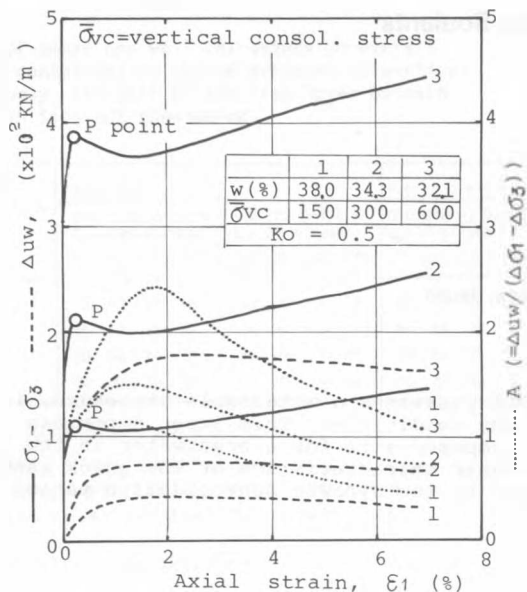


Fig. 2 Stress-strain-excess pore pressure curves obtained from K_0 -consolidated undrained triaxial tests conducted on undisturbed samples from the S2-2 layer

obtained from these tests are summarized in Fig. 3. As shown in the figure, the following two facts can be pointed out:-

- (1) From the stress controlled tests, it can be clearly observed that the strain rate after reaching point P suddenly increases reaching 6% to 10% for an instant, which strongly demonstrates that the undrained strength at point P should be used for practical problem.
- (2) Anisotropy in undrained strength is relatively greater as follows:-

$$\frac{Su(e)}{Su(c)} = 0.18 - 0.24 (\approx 0.2) \quad (2)$$

$$\frac{Su(s)}{Su(c)} = 0.62 - 0.64 (\approx 0.6)$$

$Su(e)$ = undrained extension strength

$Su(s)$ = undrained simple shear strength

When the strength ratio expressed by eq. (2) is multiplied by the compression strength under undisturbed conditions expressed by eq. (1), undrained extension and simple shear strengths, $Su(e)$ and $Su(s)$ can be obtained as follows:-

$$Su(e) = 0.2 \times (10 + 0.31\bar{\sigma}_{vc}) \text{ KN/m}^2 \quad (3)$$

$$Su(s) = 0.6 \times (10 + 0.31\bar{\sigma}_{vc}) \text{ KN/m}^2$$

IMPROVEMENT OF THE SILTY SAND BY DYNAMIC CONSOLIDATION METHOD

Because of the low undrained strength of the quick sand discussed in the previous section, there is a high possibility for a catastrophic flow failure when a structure is constructed on the silty sand at the site. It was determined, therefore, to carry out soil improvement for this silty sand. For the soil improvement, it was proposed to adopt the Dynamic Consolidation method (hereafter called D.C. method). The following field evaluation procedure was adopted.

- (1) Carry out trial treatment with D.C. at a

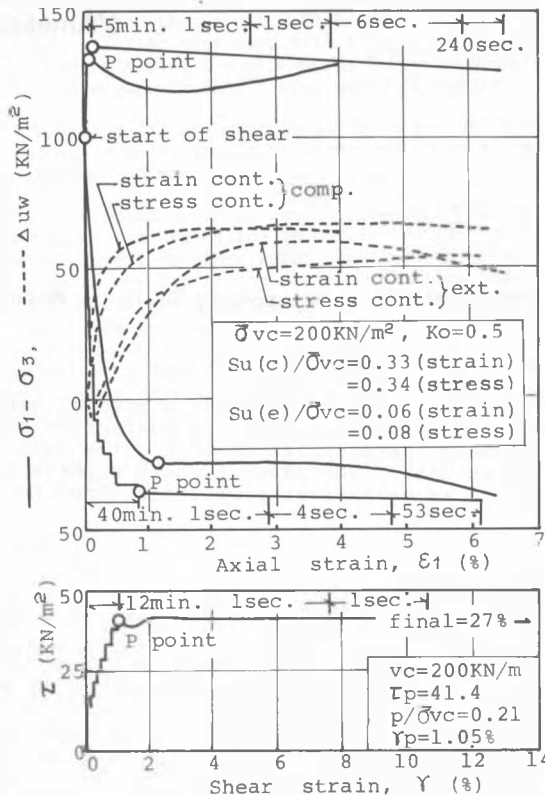


Fig. 3 Stress-strain curves obtained from 1) stress and strain controlled triaxial compression and extension tests, and 2) stress controlled simple shear tests on disturbed samples from the S2-2 layer

test area (100m long and 40m wide) where the thickness of quick sand is maximum, and to carry out various in-situ and laboratory tests on the silty sand before and after trial treatment with D.C.

- (2) If the tests results after D.C. satisfy the stability requirements of the foundation of the breakwater, the D.C. method would be adopted for the proposed site.

For the trial treatment, a square shaped rammer (2.4m x 2.4m), weighing 32t, hollow (to decrease the resistance in the water), was dropped from a 10m to 12m height. Rock fill material 1.5m thick was filled over the original ground surface to keep the rammer sinking into ground during tamping. Then tamping was carried out in two steps as follows:

- (1) First path: The trial area was divided into square grids (4m x 4m) and 7 to 10 tamps were applied to the center of each grid.
- (2) Second path: 3 to 7 tamps were applied to the square grid (2.5m x 2.5m) and numbers of tamps were controlled to obtain a flat ground surface.

The layout of D.C. method and the tamping method used at the trial area was as shown in Fig. 4

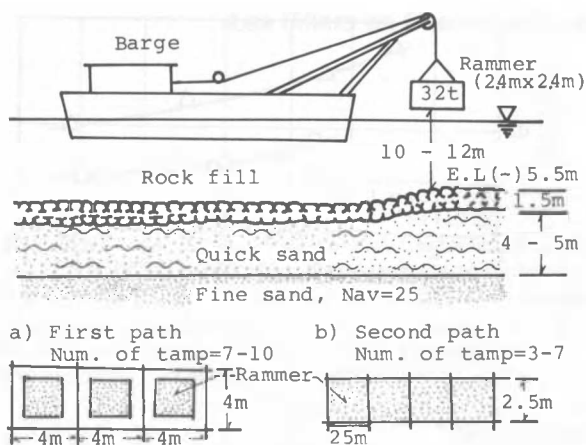


Fig. 4 Layout of D.C. method and tamping method used at the trial area

COMPARISON OF IN-SITU TESTS RESULTS BEFORE AND AFTER SOIL IMPROVEMENT BY THE D.C. METHOD

Following in-situ tests, 1) standard penetration tests, 2) pressure meter tests and 3) undisturbed sampling were carried out at the trial area before and after soil improvement by the D.C. Comparison of water content values measured on undisturbed samples, N values, deformation modulus (E) and limit pressure (pl) values (from pressure meter tests) before and after improvement are summarized in Fig. 5. It can be clearly observed that the water content after D.C. decreased 5% to 7% and that $E/\bar{\sigma}_{vo}$ and $pl/\bar{\sigma}_{vo}$ values considerably increased after D.C. ($\bar{\sigma}_{vo}$ = effective vertical stress). The in-situ tests results before and after improvement did not show any significant difference below E.L.(-)12m, which suggests that the effect of D.C. would be limited to a depth of 5m to 6m from the under-water ground surface.

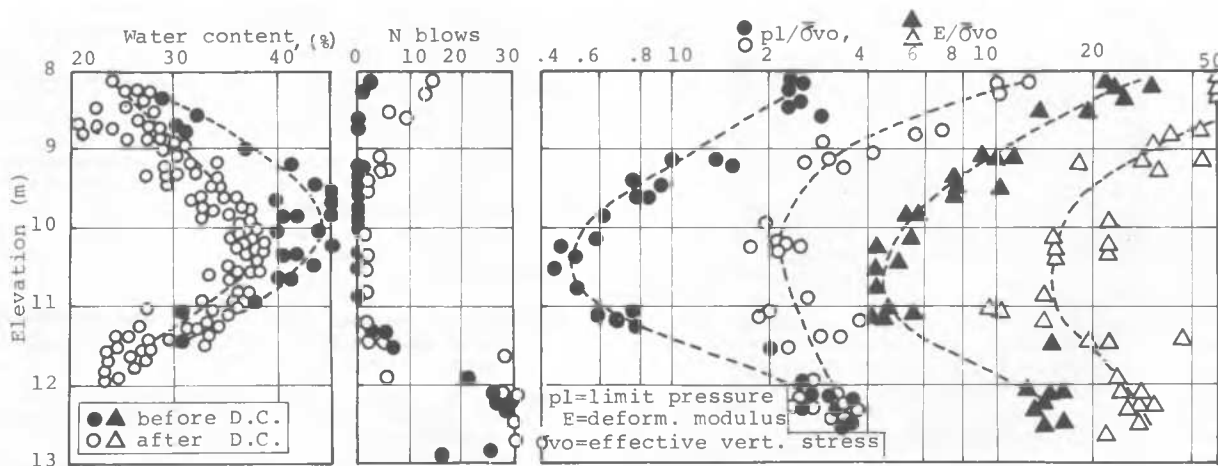


Fig. 5 Change in water content, blow counts (N), deformation modulus (E), and limit pressure (pl) values for the silty sand after treatment with Dynamic Consolidation

COMPARISON OF SHEAR CHARACTERISTICS OF THE SILTY SAND BEFORE AND AFTER IMPROVEMENT BY D.C.

Various kinds of triaxial tests on undisturbed samples from the silty sand were conducted to determine the efficiency of the D.C. method. Test results obtained from isotropically consolidated drained tests on undisturbed samples from the S2-2 layer before and after D.C. are indicated in Fig. 6 in which volumetric strain values $\Delta V/V_0$ are plotted versus normalized stress, $(\sigma_1 - \sigma_3)/\bar{\sigma}_c$ ($\bar{\sigma}_c$ = isotropic consolidation stress). As observed in the figure, there is considerable difference in $\Delta V/V_0$ values before and after improvement, particularly when $\bar{\sigma}_c$ is less than 200 kN/m^2 .

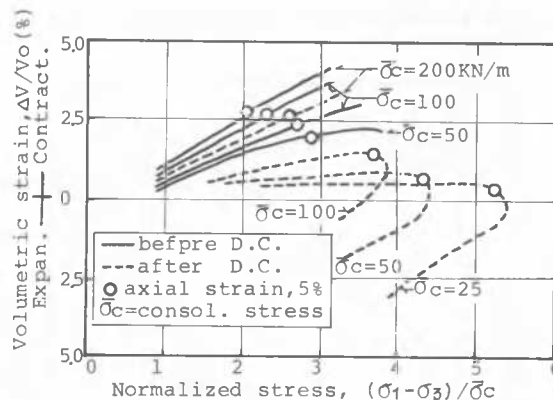


Fig. 6 Normalized stress-volumetric strain curves obtained from isotropically consolidated drained tests on undisturbed samples from the S2-2 layer before and after improvement by D.C.

On the other hand, as indicated in Fig. 2, the undrained shear behaviour of the silty sand at the site is strongly characterized by limited liquefaction behaviour at very small axial strain. It was most important to see whether this characteristic was changed or not after D.C. Anisotropically consolidated ($\bar{\sigma}_{hc}/\bar{\sigma}_{vc} = 0.5$) un-

drained triaxial tests were carried out on undisturbed samples from the S2-2 layer. The normalized stress-strain curves obtained from the tests are indicated in Fig. 7 together with pore pressure parameter, A values with strain. As shown in the figure, the limited liquefaction phenomenon observed on specimens before improvement was not observed on specimens after improvement. That is, the metastable structure of the quick sand was destroyed and changed to a normal sand structure through soil improvement by D.C. The effect of improvement was noted when $\bar{\sigma}_{vc}$ is equal to or less than 150KN/m^2 . The same trends were also observed in drained shear tests as previously pointed out. Since the stress level under dead load induced by the breakwater is less than 100KN/m^2 , this characteristic is important for evaluating the stability of the breakwater.

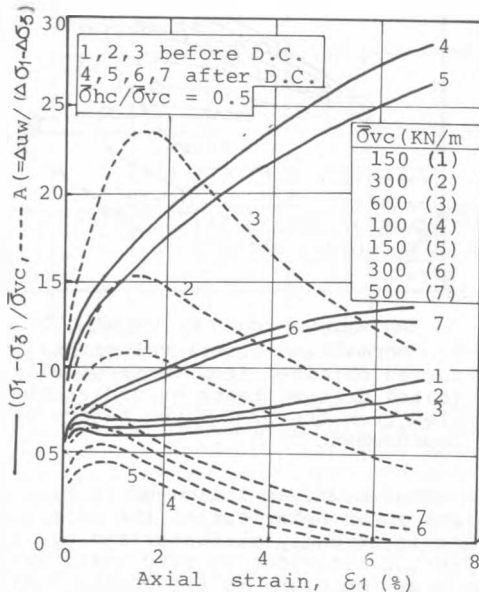


Fig. 7 Normalized stress-strain-pore pressure parameter (A) curves obtained from anisotropically consolidated undrained triaxial tests on undisturbed samples from the S2-2 layer before and after improvement by Dynamic Consolidation

LIQUEFACTION STRENGTH OF THE SILTY SAND

When the breakwater is subjected to wave forces under storm conditions, cyclic shear stress will be induced in the foundation. It is important, therefore, to determine the liquefaction strength of the silty sand for evaluating the stability of the breakwater under storm conditions. Liquefaction strength values of the silty sand (S2-2 layer) before and after improvement obtained from cyclic triaxial tests performed on undisturbed samples are compared in Fig. 8. As shown in the figure, liquefaction strength after improvement by D.C. increased about 1.4 times over that before improvement. In addition, it should be noted that liquefaction strength of the silty sand before improvement is very low when compared with that of some typical sands. This low liquefaction strength would be closely related to the limited liquefaction phenomenon observed in static undrained tests.

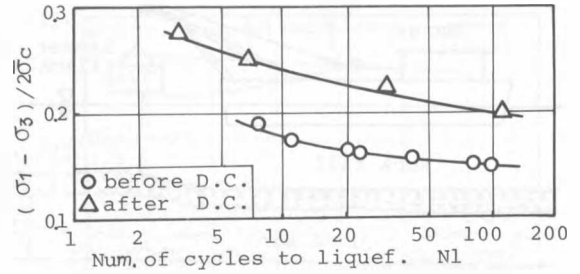


Fig. 8 Comparison of liquefaction strength of the silty sand (S2-2 layer) before and after improvement by Dynamic Consolidation

CONCLUSIONS

The following conclusions were obtained from the study:-

- (1) Undrained strength values of the silty sand at the site for practical use are as follows:-
 $S_u(c) = 10 + 0.31\bar{\sigma}_{vc} \quad \text{KN/m}^2$
 $S_u(e) = 0.2 \times (10 + 0.31\bar{\sigma}_{vc})$
 $S_u(s) = 0.6 \times (10 + 0.31\bar{\sigma}_{vc})$
- (2) Decrease in water content after improvement reached about 5% and considerable increase in deformation modulus and limit pressure (from the pressure meter tests) were observed after improvement by Dynamic Consolidation.
- (3) The efficiency of the improvement was clearly observed from measured changes in volume change characteristics during drained shear tests.
- (4) Ko-consolidated undrained triaxial test results strongly demonstrated that characteristic property of quick sand (liquefaction failure under static shear conditions) was changed by improvement.
- (5) The efficiency of the improvement by Dynamic Consolidation was clearly observed particularly when stress levels were equal to or less than 150KN/m^2 .
- (6) The efficient depth of Dynamic Consolidation under sea water was limited to a depth of 5m to 6m below the ground surface.
- (7) Liquefaction strength after improvement was increased about 1.4 times the strength before improvement.

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