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Prediction of settlement of tank foundations

Prédiction de tassement des fondations d'un réservoir

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SYNOPSIS A full-scale preloading test was conducted prior to the design of tank foundations for a large petroleum reserve storage base. The test results showed that the effects of soil improvement and characteristics of settlement conformed to calculated values, thus allowing soil improvement techniques optimized for both safety and economy to be specified for the construction of the actual tank foundations.

INTRODUCTION

The size of oil tanks has become larger and larger in recent years, mainly for economic reasons. For example, some above-ground tanks have a diameter of 80 m or over and, when fully loaded, their load intensity reaches as much as 240 KN/m^2. In Japan, large petroleum reserve storage bases are being constructed at several places in order to ensure a stable supply of petroleum. One of them, the Tomakomai Petroleum Reserve Storage Base has a total capacity of 3 million kiloliters, comprising 28 large above-ground tanks, each 82 m in diameter, 24.5 m in height and exerting 240 KN/m^2 of liquid pressure. The tank foundations for the Tomakomai Base were designed in the following manner:

(1) A full-scale preloading test was conducted with the following objectives:
   a. To ascertain the characteristics of settlement of, and an appropriate method and specification of soil improvement for, the local bearing layer soil, which consists of volcanic ash deposits in shallow layers extending from grade to 15 m in depth.
   b. To ascertain the characteristics of settlement of the diluvial cohesive soil occuring in deep layers at the site, from 30 m to 70 m in depth.

(2) Based on the results of the preloading test, soil improvement was so specified as to secure safety for the tank loadings involved and to effect the utmost economy in construction.

(3) The analysis were made as follows by the use of the FEM newly developed for axi-symmetric, elasto-plastic analysis taking consolidation into account.
   a. Determination of parameters for soil conditions from laboratory tests, and simulation of the characteristics of settlement due to preloading, on the basis of these parameters.
   b. Estimation by the use of the aforesaid parameters of the amount of settlement caused by a water test, and comparison of estimation results with the actually measured settlement to make certain of safety for the tank foundations.

The object of this paper is to present the results of these tests and analysis.

SOIL PROFILE

Fig. 1 shows the soil profile of the project site. The soil profile is divided roughly into local volcanic ash deposits (the Shikotsu Volcanic Ash) in the shallow layers (from grade to 15 m in depth), and diluvial cohesive soil (the Nitapporo Stratum) in the deep layers (from 30 to 50 m and from 55 m to 70 m in depth).

The volcanic ash deposits are (1) very loose (N-value = 1 to 5), (2) so porous as to be liable to fracture (e = 3.00 to 4.00), (3) strati-

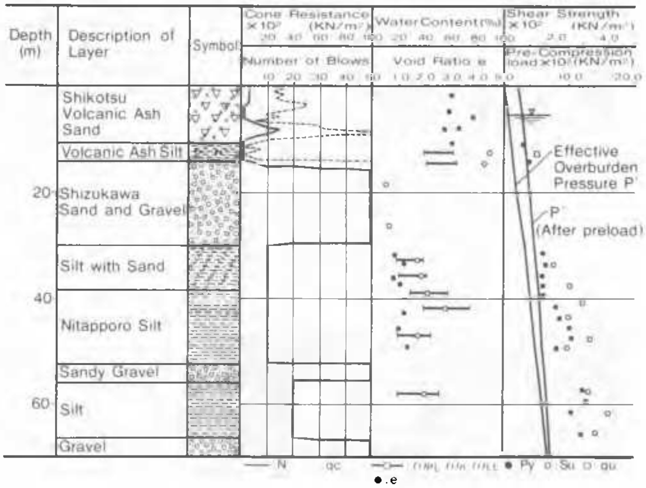

Fig. 1 Soil Profile

graphically complicated because of difference in the age of deposition or volcanism, and (4) very resistant to liquefaction during earthquake even though in the form of very loose sandy soil.

The third property suggests that the sequence of strata and the in-situ strength may be ascertained by static penetration test (e.g., the Dutch cone sounding test) rather than by dynamic penetration tests such as the Standard Penetration Test.

The Nitapporo Stratum is (1) slightly overconsolidated diluvial cohesive soil (O.C.R. = 2 to 3), (2) very hard (q_u = 200 to 400 KN/m^2), and (3) very thick: From consolidation tests, settlement of this stratum when subjected to tank loads was known beforehand to reach as much as 20 to 25 cm.

Thus, engineering problems to be considered when tanks are fully loaded are the effect of soil improvement and the characteristics of settlement for the Shikotsu Volcanic Ash and the characteristics of settlement for the Nitapporo Stratum.

OUTLINE AND RESULTS OF THE PRELOADING TEST

The layer in which to introduce soil improvement was limited to the Shikotsu Volcanic Ash.
Soil improvement was tried at nine points, with varying specifications so as to verify the effects of the soil improvement on a comparative basis. The areas of the improved soil were preloaded with an earth-mound of fill of the full diameter (80 m) and the actual load (240KN/m^2) of a tank. Fig. 2 shows the nine kinds of specifications for the soil improvement and the geometry of the preloading.

The main method of soil improvement was the sand compaction pile (S.C.P) method in which crushed stone was used instead of sand.

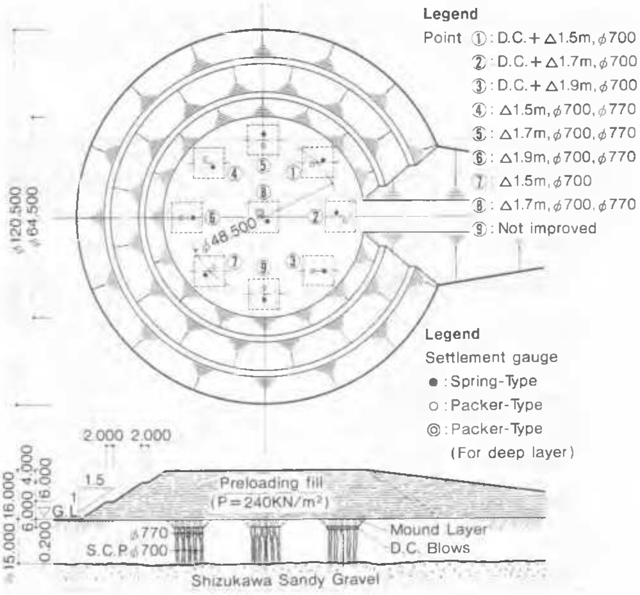

Fig. 2 Specifications for the Soil Improvement and Geometry of the Preloading

At points 1, 2 and 3, the dynamic consolidation (D.C.) method was additionally applied.
These soil improvement methods were expected to give a compaction effect to the Shikotsu Volcanic Ash. In particular, the S.C.P. method can increase the strength by replacing the existing soil with crushed stone. For the D.C. method, preliminary tamping showed that it was most appropriate to drop a 12 ton weight in a grid pattern of 3.25 m x 3.25 m from a height of 20 m.

A volume of fill of 150,000 m^3 was required for the preloading. Two kinds of settling element type multi-layer settlement gauges were installed at each point to determine the characteristics of settlement of the layers under study. Measurement by these gauges was conducted from the start of preloading until two years after its completion.

Ascertaining the effect of soil improvement

Fig. 3 shows to what degree the soil had been improved by the respective methods.

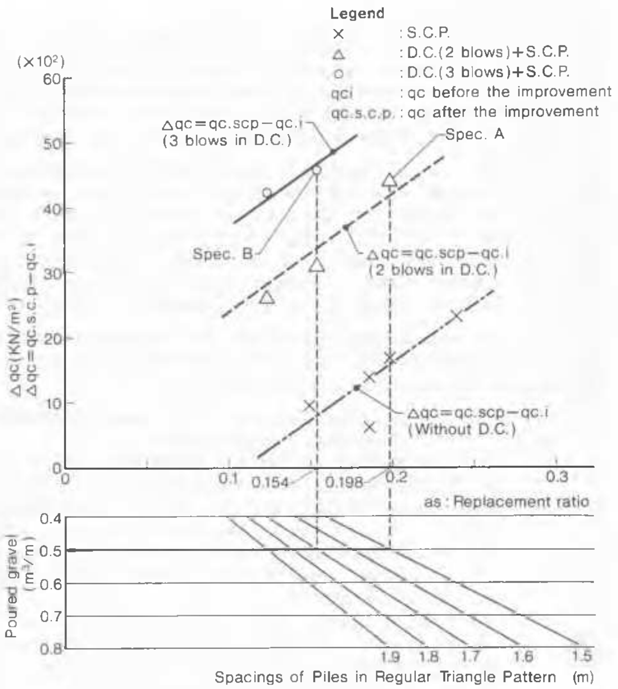

Fig. 3 qc Increase vs. Replacement Ratio

If the conventional design method of soil improvement is applied by regarding the Shikotsu Volcanic Ash as an equivalent sandy soil, the most suitable specification for improving the soil under the project conditions is a combination of the S.C.P. method, where piles are spaced 1.5 m apart in a regular triangle pattern and the replacement ration a_s is 0.198, and the D.C. method where two blows of a weight are applied with a total energy E of 454 (KJ/m^2) (Spec. A). The comparative study of the soil improvement tests, however, showed that greater strength than that achieved by Spec. A could be obtained by adopting a combination of a 1.7 m-spaced regular triangle pattern and $a_s = 0.154$ in the S.C.P. method, and three blows with a total $E = 682 (KJ/m^2)$ in the D.C. method (Spec. B). Additionally, it was found that Spec. B was much more economical than Spec. A due to the reduced number of piles required. Also, from Fig. 4 which shows the effect of soil improvement where the D.C. method only was used, it is most effective to apply three blows.

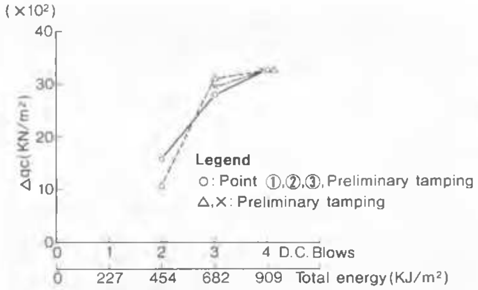

Fig. 4 No. of D.C. Blows vs. Δqc
(for Volcanic Ash)

Ascertaining the characteristics of settlement

Fig. 5 shows how total settlements of the improved layers and deeper layers including the Nitapporo Stratum changed at four of the improved points as the preloading proceeded. From Fig. 5, it can be seen that the improvement of the Shikotsu Volcanic Ash layer is remarkably effective and that the resulting settlements vary little between Spec. A and Spec. B. In addition, the deep layers including the Nitapporo Stratum do not show consolidation settlement but rather instantaneous settlement due to the preloading.

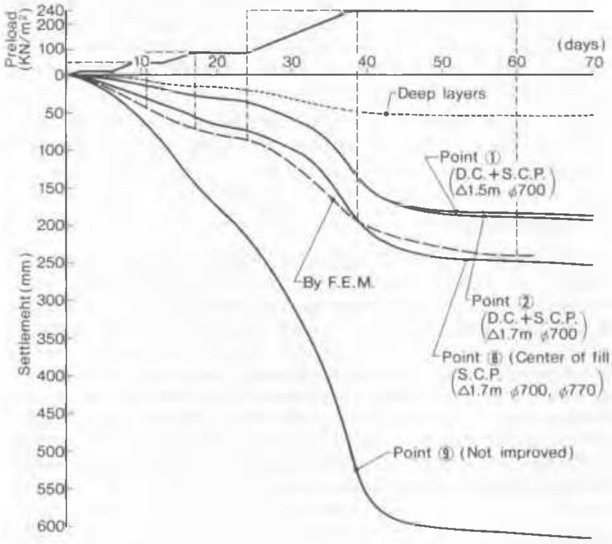

Fig. 5 Observed Settlements due to the Preloading Test and Calculated Settlements

ANALYSIS OF THE SETTLEMENT

The characteristics of settlement ascertained through the preloading test were analyzed by the FEM for axi-symmetric taking consolidation into account. The relation between stress and strain to be used for the analysis was the hyperbolic stress-strain equation proposed by Duncan and Chang (1980) for Volcanic ash and sand and the stress-strain equation obtained by revising the modified Cam clay model by Roscoe and Burland (Duncan et al. 1981). These equations are briefly explained as follows:

Stress-strain relation for sand

As shown in Fig. 6, the stress-strain relation for sand is approximated by a hyperbola.
In this hyperbola a coefficient E_i of initial elasticity is expressed as follows:

$$E_i = K \cdot P_a \left(\frac{\sigma_3}{P_a}\right)^n \qquad (1)$$

where,
P_a : Atmospheric pressure
K : Modulus number
n : Hyperbolic parameter

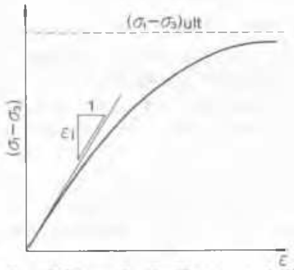

Fig. 6 Hyperbolic Representation of a Stress-strain Curve

A coefficient E_t of tangential elasticity in a certain stress is expressed as follows using the Mohr-Coulomb's failure conditions:

$$E_t = \left[1 - \frac{R_f(1-\sin\phi)(\sigma_1-\sigma_3)}{2C\cdot\cos\phi + 2\sigma_3\sin\phi}\right]^2 \cdot K \cdot P_a \left(\frac{\sigma_3}{P_a}\right)^n \qquad (2)$$

where,
C : Cohesion
ϕ : Angle of internal friction
R_f : Failure ratio

Stress-strain relation for clay

The stress-strain relation for clay is analyzed by the use of the model obtained by revision of

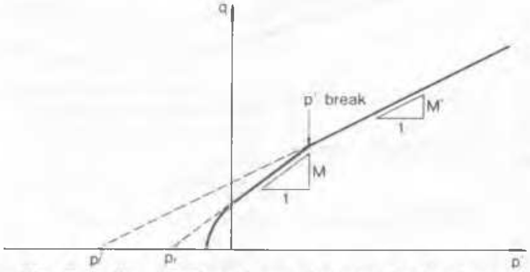

Fig. 7 Failure Surface in q-p' Space for Revised Cam Clay Model

Parameters of Sand and Clay

Layer No.	γ_t (tf/m³)	ϕ	C	K	n	R_f	K_b	m
S1	1.5	38	0.0	600	0.4	0.7		0.2
S1'	1.8	30	0.0	400	0.4	0.7		0.2
S2	2.0	46	0.0	1,750	0.4	0.7	1,200	0.2
S3	1.9	42	0.0	1,500	0.4	0.7	800	0.2
S4	2.0	56	0.0	2,000	0.3	0.7	1,250	0.2

Layer No.	γ_t (tf/m³)	κ	e_0	P_r, P_r' (KN/m²)	M, M'	K (cm/min)	ν
C1	1.8	0.0057	0.85	0.0	2.2	2.8×10^{-7}	0.45
C2	1.8	0.0037	1.20	0.0	2.2	1.7×10^{-7}	0.47
C3	1.9	0.0060	1.13	0.0	2.0	9.6×10^{-8}	0.43
C4	1.6	0.0281	2.33	0.0 144.0	1.85 1.11	1.62×10^{-6}	0.30

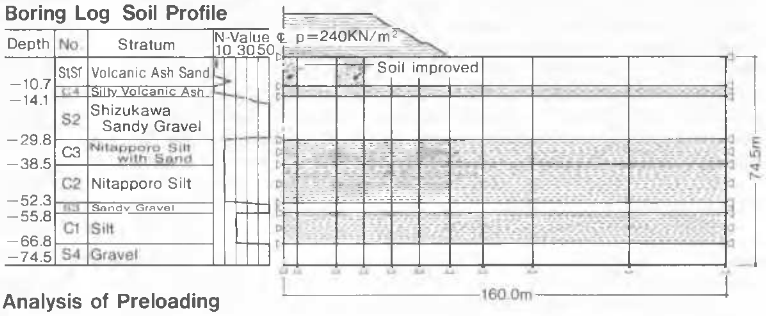

Fig. 8 Mesh for the FEM Analysis of Preloading

the modified Cam clay model. This model was revised as follows:

(1) As shown in Fig. 7, the failure surface was bilinearly expressed so as to delineate the test result that M decreases as P' increases.

(2) Cohesion was taken into account.

(3) For other states than triaxial compression, M was reduced according to b-values which show the degree of intermediate principal stress, by making use of the Lade's failure surface.

(4) For the parameter λ, the value obtained through the consolidation test was used according to the consolidation pressure P_o'.

(5) The Poission's ratio was used according to P' so as to express the change in bulk strain.

Fig. 8 shows the mesh used for the analysis of the preloading fill. The parameters for the stress-strain equations were determined from the triaxial drained shear test for volcanic ash and sand and from the consolidation test and the triaxial compression test for clay. Values of the parameters are also given in Fig. 8. The analysis results which are indicated by a broken line in Fig. 5 agree well with the measured values.

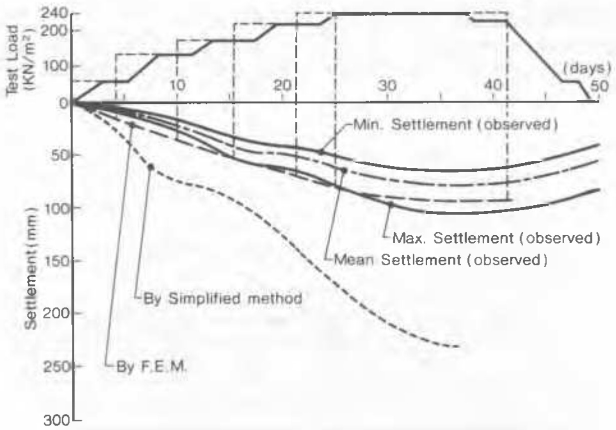

Fig. 9 Observed and Calculated Settlements during the Water Test

ESTIMATION OF SETTLEMENT DUE TO THE WATER TEST

The amount of settlement to be caused by the water test before completion of the tanks was estimated by the use of the parameters employed for the analysis of settlement due to the preloading test. In Fig. 9 the estimated settlements of the representative tank are compared with the measurement results. Fig. 9 shows in combination the settlements calculated by the simplified method, that is the De'Beer's formula for sand and the e-log p method for clay. The settlements obtained by such a simplified method were excessive as shown in Fig. 9, and so it is found that the FEM is very effective in estimating the settlements of the tanks.

CONCLUSION

(1) The preloading test has validity for optimizing tank foundation design for a large petroleum base, such as that described.

(2) The applied soil improvement (Spec. B) led to remarkably economical construction in comparison with the soil improvement (Spec.A) applied to adjoining tank foundations having the same soil profile.

(3) The FEM used for the analysis is very effective in estimation of the tank settlements.

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