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SYNOPSIS Two SPTs and two CPTs for ground before and after stabilization,three static loading tests for the stabilized ground with stone columns were performed. It was found that stone columns strengthened the resistance to seismic liquefaction of a saturated clayey silt layer and increased the ground bearing capacity two times.From a water preloading test of an oil tank on the improved ground,it was obtained that stone columns not only decreased the total and differential settlement of the tank but also speeded the consolidation rate of the ground. Stone columns also reduced the initial excess pore water pressures developed in the improved ground in comparison with those in the unimproved one.

INTRODUCTION

An oil tank with the volume of 10000 cubic meters was planned to construct on scft soils in Shanghai. Before it had been built, ground investigation was performed. It was shown that the bearing capacity of the ground didn't meet the need of the design load of the oil tank, and liquefaction of a saturated clayey silt may develop under the basic seismic intensity of seven, which is considered in Shanghai. In order to raise the bearing capacity, to reduce the time of settlement and to strengthen the resistance to the seismic liquefaction of the saturated clayey silt, stone column method was adopted.To testify the effectiveness of the treated soil and to realize the mechanism of stone columns,the in-situ static loading plate tests,SPTs,CPTs and a water preloadiing test of an oil tank were carried out.

SOIL CONDITION AND DESIGN PARAMETER

The oil tank is a steel-sheet floating top structure, with a height of 16m. Its foundatiion is a circular beam, which is shallow one, with a diameter of 30.4m. The designed pressure of the foundatiion base is 190kPa. The allowable relative inclination of the tank is limited to be 0.004.

The soil condition is listed in Table 1, in which the 2nd layer is formed of the clayey silt liable to liquefation under the basic seismic intensity of seven. The (3)a and (3)b layers were newly deposited so that they are very soft . Hence,stone columns were used to treat the above three soil layers.

Stone columns were arranged as a square type,with a spacing of 1.5m. Its stabilization range has a diameter of 39.4m,as shown in Fig.1.In order to reduce the differential settlement of the center and edge of the tank, three types of columns with different length were adopted in three zones,that was the center zone with a length of 11.0m, the middle zone of 9.0m and the outer one of 7.0m. According to the volume of stone aggregates used, the average diameter of stone columns is computed to be about 0.80m. The area replacement ratio of stone columns is 0.22.

INVESTIGATION RESULTS BEFORE AND AFTER TREATMENT

Three static loading plate tests were carried out after installation of stone column for about one month.Stone column No.154 and No.168 with the length of 9.0m were chosen for two loading tests of single column, and No.170 for the loading test of composite ground which is formed of a column

TABLE 1 Soil Parameters

No.	Name	H	w	e0	Ip	a	φ	c	qu	R
(1)	fill	0.65								
(2)	clayey silt	5.70	32.1	0.95	9.8	240	20	6.0	59	100
(3)a	silty clay	8.60	38.8	1.16	11.6	480	19	7.0	38	90
(3)b	silty clay		34.8	1.10	11.0	350	19	7.0	55	100

Note:
H--bottom depth of soil layer,m; w--water content,%;
e0--void ratio; Ip--plastic index; φ--friction angle,° ;
c--cohesion,kPa; a--compressive coefficient,MPa-1;
qu--unconfined resistant strength,kPa;
R--allowable bearing capacity,kPa.

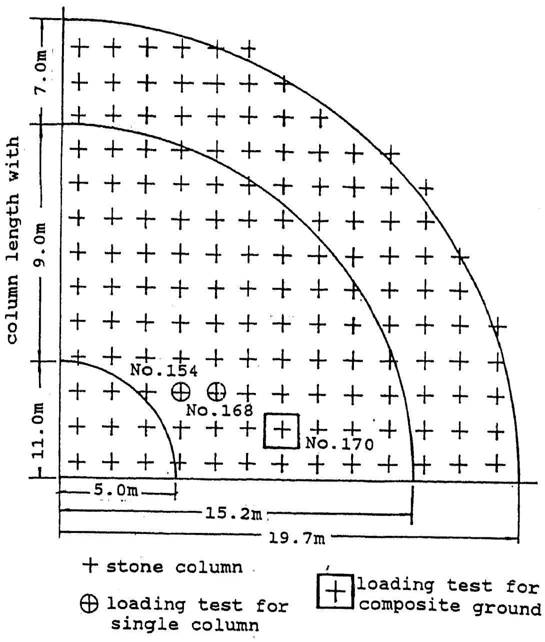

Fig.1 Layout of Stone Columns and Loading Tests

and its surrounding soil.The circular plate with a diameter 0.80m for the former tests and the square one with a size 1.5*1.5 m for the latter test were used. The above tests were adopted with a load-maintain method.

As shown in Fig.2, the allowable bearing capacity of the composite ground equals to 200kPa and those of single columns to more than 400kPa.But that of the native ground is presented to be 100kPa in Table 1. Thus,the capacity of the treated ground is twice that of the untreated one.

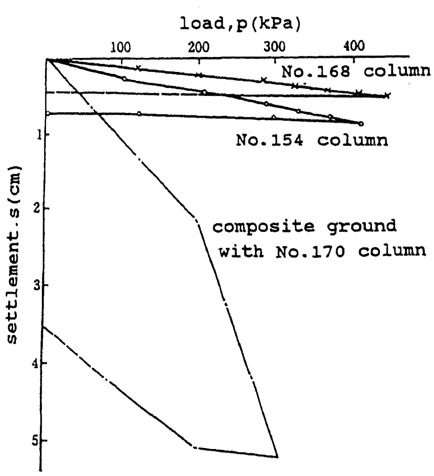

Fig.2 Curves of Loading Tests of Single Column Composite Ground

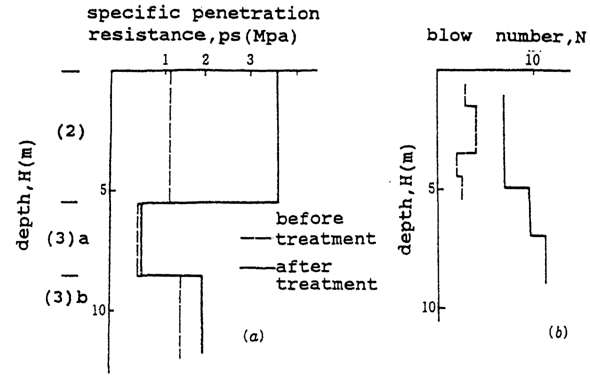

Fig.3 Comparison of Blow Number(SPT),Specific Penetration Resistance(CPT) for Treated and Untreated Grounds

As shown in Fig.3,the average specific penetration resistance ps of CPT and the blow number of SPT for some treated soil layers had obviously changed in comparison with those for the untreated ones, especially for the 2nd soil layer, the ps had increased by 180 %. The follow formulas is a standard in China to judge whether liquefaction will occur or not for saturated clayey silt.

$$N'=N(1+0.125(ds-3)-0.05(dw-2)-0.10(Pc-3)) \quad (1)$$

where, N'— critical blow number;
N — basic blow number;
ds— depth of blow point;
dw— depth of water level;
Pc— clay content.

Let Pc equal to 3 and basic seismic intensity to be seven in Shanghai,the critial blow numbers of SPT for different depth in the 2nd clayey silt layer are computed as listed in Table 2. The

TABLE 2 Comparision of Critical and Measured Blow Number

Depth(m)	1.0	2.0	3.0	4.0	5.0	6.0	8.0
N1	4.1	4.8	5.5	6.3	7.1	7.8	9.3
N2	3.0	4.0	4.0	2.0	2.5		
N3	6.5	6.5	6.5	6.5	9.5	9.5	11.0

Note:
N1--critical blow number;
N2--measured blow number before treatment;
N3--measured blow number after treatment.

measured blow numbers for the 2nd untreated soil layer at different depth are less than the critial value,therefore,the possiblity of liquefaction exists. However,the blow numbers for the treated one are more than the critical value.This proves the effectiveness of stone columns on preventing the liquefaction of the 2th soil layer under basic seismic intensity of seven.

PRELOADING TEST OF OIL TANK

In order to control the rate of settlement of the oil tank and keep the ground stable during water preloading, serval observation points were established in advance.

Six piezometers were placed under the oil tank foundation so that the process of increment and dissipation of the excess pore water pressures in the ground could be monitored during the preloading (Fig.4). During this process, two piezometers U1-1 and U2-1 went wrong one after another.

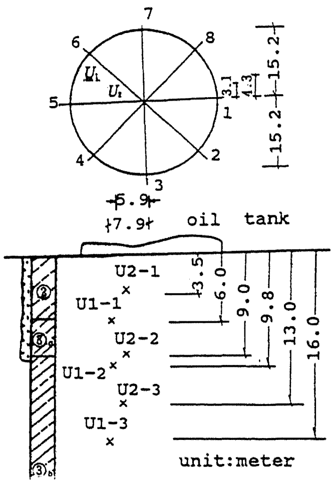

Fig.4 Layout of Settlement Observation Points and Piezometers

Also as shown in Fig.4,eight settlement observation points were arranged at the same distance around the tank edge.A nut welded and connected at the bottom of the tank by one end of a steel wire, penetrated through a pre-hole at the center on the top of the tank. A plummet tied with the other end of the wire was hung outside the tank. Thus,the settlements at the center were obtained through measurement of the height change of the plummet.
According to the requirement of the design,the total height of the water level within the tank is 16.03m.Thirteen stages of water preloading were carried out,as shown in Fig.5. The rate of water preloading was controlled according to the rate of settlement of the tank not more than 15mm to 20mm per day and the excess pore water pressure increment less than sixty percent of the applied load in order to keep the foundation of the tank stable.

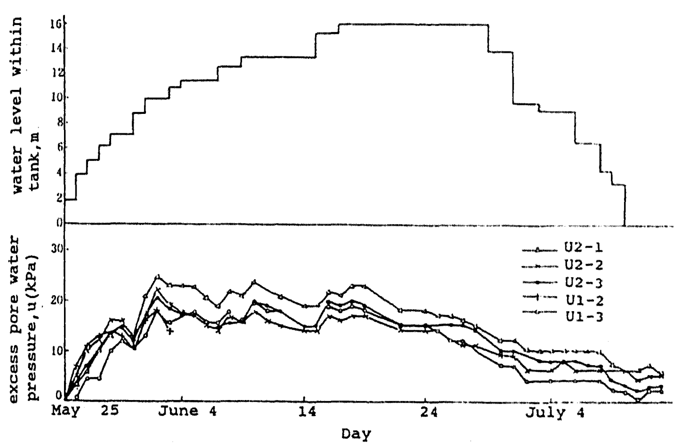

Fig.5 Relationship between Measured Excess Pore Water Pressures and Water Level with Tank

MEASURED EXCESS PORE WATER PRESSURES

The excess pore water pressures during the water preloding were measured as shown in Fig.5, which increased and dissipated regularly with the load applied and intermission.
As plotted in Fig.6,the accumulative excess pore water pressure,$\Sigma\Delta u$ has a liner relationship with the applied load,$\Sigma\Delta p$. This shows that the ground was kept in the state of elastic deformation and not in that of plastic one during all the water preloading.

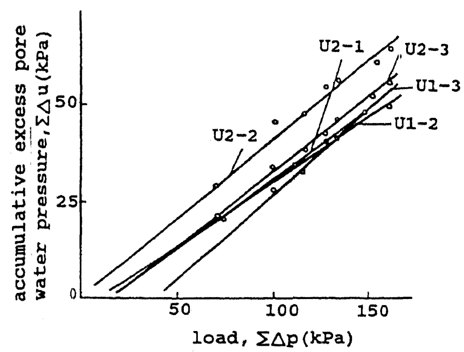

Fig.6 Relationship between Accumulative Excess Pore Water Pressure and Applied Load

COMPUTED COEFFICIENT OF PORE WATER PRESSURE

Excess pore water pressure, u0,caused by immediate load applied on saturated soft clay can be expressed by

$$u0 = \sigma_m + \alpha_1 \tau_m \tag{2}$$

where, u0—excess pore water pressure caused by immediate load;
σ_m—octahedral normal stress;
τ_m—octahedral shear stress;
α_1 —pore water pressure coefficient.

In the state of elastic deformation,the octahedral stress at some point in the ground under circular normall load applied can be computed by

$$\sigma_m = K_\sigma \cdot p \tag{3}$$
and $$\tau_m = K_\tau \cdot p \tag{4}$$
where, p— applied load;
K_σ— coefficient of octahedral normal stress;
K_τ— coefficient of octahedral shear stress.

Therefore,Eq.(2) can be rewritten as follows

$$u0 = (K_\sigma + \alpha_1 K_\tau)\cdot p = Ku\cdot p \tag{5}$$

where,Ku—coefficient of pore water pressure and load.

The coefficient,Ku, is an important index commonly used to control the rate of load applied in practise. From the Fig.6, the Ku value can be calculated as listed in Table 3.
The coefficient of pore water pressure can be computed by

$$\alpha_1 = \frac{Ku-K_\tau}{K_\tau} \tag{6}$$

As presented by Skeptom,the coefficient of pore water pressure,A,can also be calculated by

$$A=\frac{\sqrt{2}\alpha_1+1}{3} \tag{7}$$

If the ground treated by stone column is considered as a composite ground which is homogeneous,the above formulus can also be used here.As shown in Fig.7,the relationship of pore water pressure coefficient, α_1 or A with depth,H is established.With the increase of depth,the values,α_1 and A change from the negative to the positive. It is interesting that the values α_1 and A are both minus within the depth of 9.0m, which is just the length of stone column. At the depth of 9.0m or so, which is the intersection of the treated zone with untreated one,the values, α_1 and A,become near zero. In Shanghai area, for slight excess consolidation clay the value A equals to about 0.3, and for normal consolidation clay, A=0.60-0.75. Thus,under the depth of 9.0m the value,A, shows the positive as the untreated soil . It can be concluded that stone columns have the role in reducing the coefficient of pore water pressure of the ground and restraining the occurrence of initial pore water pressure,so that it can keep the ground stable.

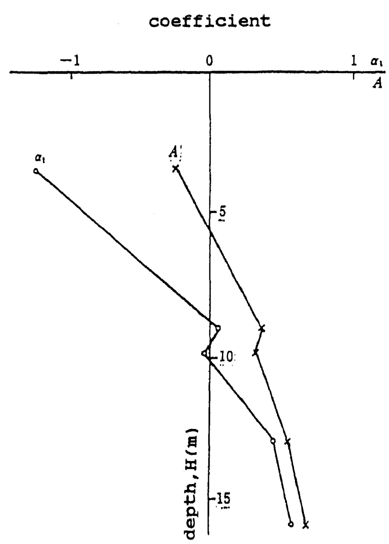

Fig.7 Computed Coefficients of Pore Water Pressures with Depth

It's worth pointing out that these values are two computed ones,which are different from those of the surrounding soil obtained from laboratory test. But they are both composite values and can still be used to calculate the initial excess pore water pressure developed in treated ground,which can be compared with those in untreated one.Of course,it is under study to establish the relationship of the computed values with the laboratory ones.

TABLE 3 Coefficent of Pore Water Pressure-Load

No.	U1-2	U1-3	U2-1	U2-2	U2-3
$Ku=\frac{\Sigma\Delta u}{\Sigma\Delta p}$	0.322	0.399	0.358	0.410	0.408

SETTLEMENT CALCULATION OF TANK ON UNTREATED GROUND

The settlement of the tank on the untreated ground can be calculated by

$$s_\infty = sd+sc=ms\cdot sc \tag{8}$$

where,sd—immediate settlement;
sc—consolidation settlement;
ms—an empirical coefficient,which depends on the factr of lateral deformation and others,generally in the order of 1.2-1.4 in Shanghai.

For this case, when the height of water level within the tank reaches 16.03m, the calculated total settlement at the center and the average one around the edge of the tank are 84.44cm and 53.61cm,respectively.Their differential settlement is 30.83cm,which is 2.03 percent of the radiu of the tank.

ANALYSIS OF MEASURED SETTLEMENT-TIME CURVE

The total settlements of the tank on the treated ground with stone columns may be deduced by means of the measured settlement-time curve. The exponent equation method is adopted here.Generally the consolidation degree of stone column ground is also expressed as

$$Ut=1-\alpha\exp(-\beta t) \tag{9}$$

On the curve,three points (t,s) are chosen,let t=t2-t1=t3-t2. The parameters, α, β, s_∞ and sd, can be solved in the following equation

$$\exp(-\beta\Delta t)=\frac{s3-s2}{s2-s1} \tag{10}$$

$$s_\infty = \frac{s3(s2-s1)-s2(s3-s2)}{(s2-s1)-(s3-s2)} \tag{11}$$

$$\alpha\exp(-\beta t1)=\frac{1}{1+\frac{s1-sd}{(s2-s1)2}(2s2-s1-s3)} \tag{12}$$

$$sd=\frac{st-s_\infty(1-\alpha\exp(-\beta t))}{\alpha\exp(-\beta t)} \tag{13}$$

where,s1,s2,s3,st—the settlements at the time of t1,t2,t3 and t,respectively.

From Fig.8, the following results can be deduced and are listed in Table 4.

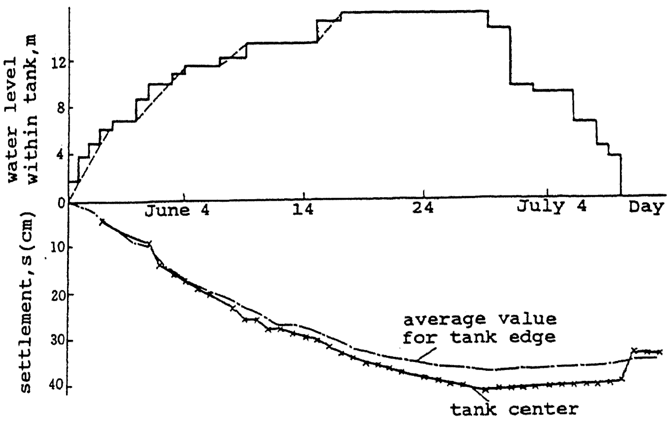

Fig.8 Measured Settlements with Time

TABLE 4 Deduced Results from Settlement-Time Curve

observation point	at the center	around the edge
measured settlement for 35 days, s35d(cm)	41.6	37.4
consolidation coefficient, β (per day)	0.0756	0.1171
total settlement s_∞ (cm)	47.8	39.2
consolidation coefficient, α	0.95	1.00
consolidation degree for 35 days, U(%)	86.9	96.5
immediate settlement sd(cm)	0.7	0.0
consolidation settlement sc(cm)	47.1	39.2
deduced value ms	1.01	1.00
s35d/s	0.871	0.954
rebound after dewatering(cm)	8.0	2.5

Some conclusions can be made from the Table 4:

a) When the water preload is applied for 35 days, the consolidation degree at the center of tank has reached 87 percent, the average one at the edge approximately 96 percent, which don't include the secondary consolidation. The rate of consolidation is very fast. It is because that stone columns are good drainage and the stress concentrated on the columns reduces that on the soil and the excess pore water pressures.

b) The computed values, α equals to 0.95-1.00, β to 0.0756-0.1171 per day. The latter is as many as 4-7 times of that estimated from the similar untreated ground introduced in ref.(1). The value, β is an important index expressing the rate of consolidation. It is valuable to establish the regional empirical data of this value.

c) The value, ms is calculated to be equal to 1.00 for this site. However, that is adopted in the range 1.2-1.4 for normal consolidation or slight excess consolidation clay, which is untreated. Hence, the settlement caused by lateral deformation is reduced because of the stabilization of stone columns.

d) The total settlements of the tank on the treated ground are 47.76cm at the center and 39.22cm at the edge. They are reduced by 43.3 percent and 26.8 percent, respectively by comparison with the untreated one. The differential settlement at the center and at the edge is 8.54cm, having 0.56 percent of the radiu of the tank. It is greatly reduced after the treatment. This shows it capable of stone columns with different lengths in different zones to reduce the differential settlement of the tank.

e) The rebound at the center and at the edge are 8.0cm and 2.5cm, respectively after the preload unloaded.

As shown in Fig.9, the largest settlement occured in the point 3 at the edge, which was 40.2cm when the preloading completed and the smaller one in the point 7, which was 34.6cm. The differential one equals to 5.6cm with the relative inclination of 0.0019, which meets the design requirement of less than 0.004 for floating top oil tank. Also as shown in Fig.9, the tank inclines toward the south-west direction and the settlement curve only has a peak. This shows the total plane inclination occuring for horizontal circular foundation after the settlement of the tank. There is no harmful twisting deformation.

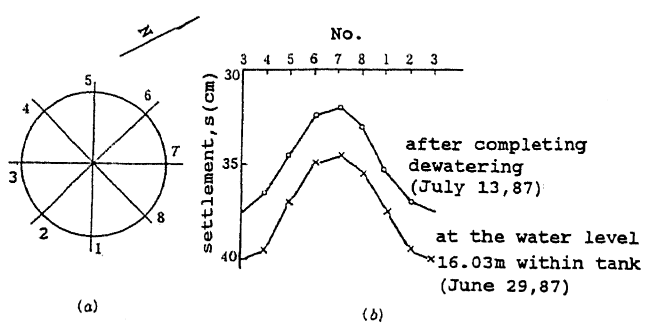

Fig.9 Distribution of Measured Settlements along Tank Edge

COMPUTED CONSOLIDATION DEGREE

As shown in Fig.8, the loads are considered to be applied in four stages within 23 days. The improved method for sand well ground is adopted to compute the average consolidation degree at the center of the tank

$$U_t=\sum_{i=1}^{n}\frac{\dot{q}_n}{p}\left((t_n-t_{n-1})-\frac{\alpha}{\beta}e^{-\beta t}\left(e^{\beta t_n}-e^{\beta t_{n-1}}\right)\right) \qquad (14)$$

where, U_t — the average consolidation degree applied by serval load stages at time, t;

$\dot{q}_n$ — the average rate of load applied for the nth load;

p — the total load at time, t;

t_{n-1}, t_n—the time at the start point and the terminal one for each load applied;
α, β —two consolidation parameters, computed from the measured settlement-time curve as discussed above.

From the Eq.(14), the average consolidation degree at the center of the tank is computed to be 87.5 percent after the tank is preloaded for 35 days. It is shown that their results from settlement-time curve and theoretical equation are in agreement. This also shows the improved method can be used to compute the consolidation degree of stone column foundation. However, it must be pointed out that the values, α , β may be adopted from the measured settlement-time curve before they can be determined rationally. It is because stone columns have their special effects, some of which are different from ones sand wells have.

CONCLUSIONS

After serval in-situ tests have been studied, the following conclusions can be made:
1.The allowable bearing capacity of treated ground by stone columns was twice that of the untreated ground.
2.After the installation of stone columns, the clayey silt was compacted obviously, so that it has an ability to prevent the possible liquefaction under the basic seimsmic intensity of seven.
3.The treated ground has the excess pore water pressure coefficients, α_1 and A less than the untreated one. Stone columns have a role in reducing the initial excess pore water pressure developed in the treated ground.
4.The parameters of consolidation of the treated ground, α and β are in the range of 0.95-1.00 and 0.0756-0.1171 per day, respectively. The consolidation degree at the center of the tank reached 87 percent after the tank was preloaded for 35 days. The improved method for sand well ground can be used to compute the above average consolidation degree of stone column treated ground.
5.Stone columns which are arranged to have different lengths in different zones can be effectively used to reduce the differential settlement of the oil tank. The empirical coefficient of settlement, ms, is computed to be equal to 1.00 for this site.

REFRENCES

1.Sun,G.S. & Zheng,D.T. (1984), Soft Soil Foundation and Underground Engineering(in Chinese), China Building Industry Press.
2.Jia,Q.S. & Jing,B.Y. (1985), Field Study of Large Scale Oil Tanks Constructed on Shanghai Soft Soil With Horizontal Laminated Sand(in Chinese).
3.Han,J. & Ye,S.L. (1989),Field Tests and Analyses of Soft Clays Treated with Stone Columns in Shanghai Lianggong Valve Plant (in Chinese), Journal of Zhejiang Construction,No.2.
4.Han,J. & Ye,S.L. (1991),Field Tests of Soft Clay Stablized by Stone Columns in Coastal Areas in China,4th Int. DFI Conf.,Italy.