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Performance of an Embankment Built on a Soft Disturbed Clay

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SYNOPSIS A high embankment was built on a soft clay disturbed due to an inadequate technique. Settlement analysis has been carried out to estimate coefficients of consolidation c_v, final settlements and average degrees of consolidation U. Laboratory and in situ tests have been performed to estimate c_v values as well. Piezometric measurements have also been made to estimate U values corrected with non linear oedometer test data. Times for settlement stabilization have been estimated with the available data.

INTRODUCTION

In mid 1988 an embankment about 24m high was built close to the city of Rio de Janeiro to support an electric substation. The site investigation performed prior to construction showed that a soft clay deposit, 5m thick in average, was present in part of the embankment area. The soft clay was planned to be removed by forced displacements using bulldozers employed in the embankment construction.

Surface marks installed in mid 1989, a few months following completion of construction showed undesirable settlements and settlement rates. Hence additional site investigation (SPT) was planned and a clay layer 2,5m thick in average was, unfortunately, shown to remain under the embankment. For this reason the construction of the concrete structures was halted and COPPE was requested by the client (in late 1989) to perform special studies.

These special studies consisted of the analysis of a large amount of settlement data provided by the client plus installation of deep instrumentation and realization of piezocone and oedometer tests. The main purpose of these studies was to provide information about the most likely time for settlement stabilization. Thus it was important that coefficients of consolidation and final settlements were provided.

SETTLEMENT ANALYSIS

The substation site is shown in Fig. 1. The small hills located at the borders of the site were used as borrow sites for the compacted soil. The most critical area, as far as settlements are concerned, is shown in Fig. 2. This is the area where a thicker deposit of soft clay remained under the embankment. About 90 measurement points from a total of 179 located in the whole area were used for settlement control as also shown in Fig. 2.

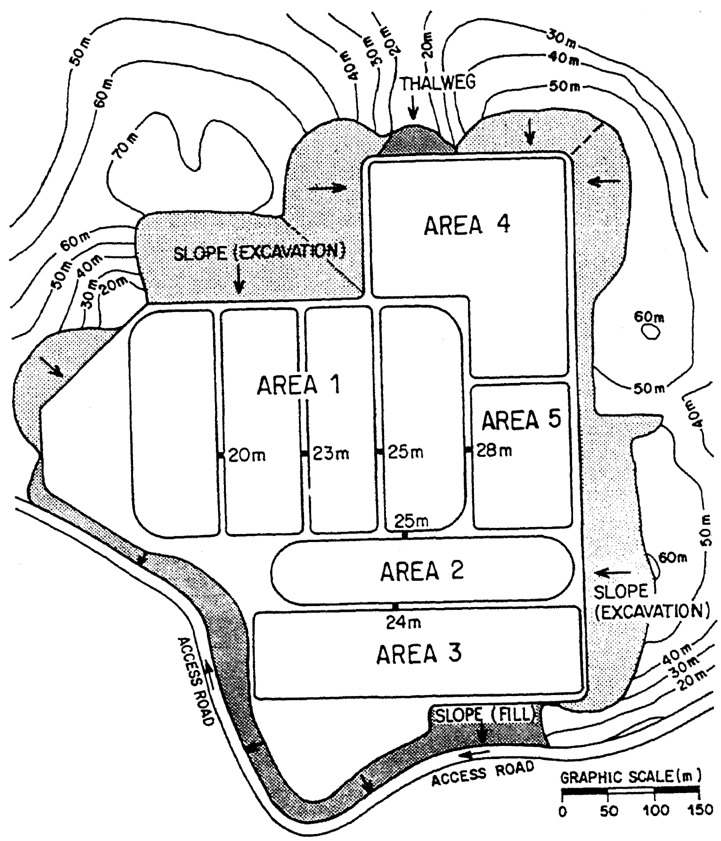

Fig. 1 - Substation site.

Methodology for settlement analysis

Settlements were measured by means of surface marks and pins placed in foundation blocks constructed at the embankment surface. These measuring points were installed about three months after the embankment reached the final elevation, as shown in Fig. 3. Therefore

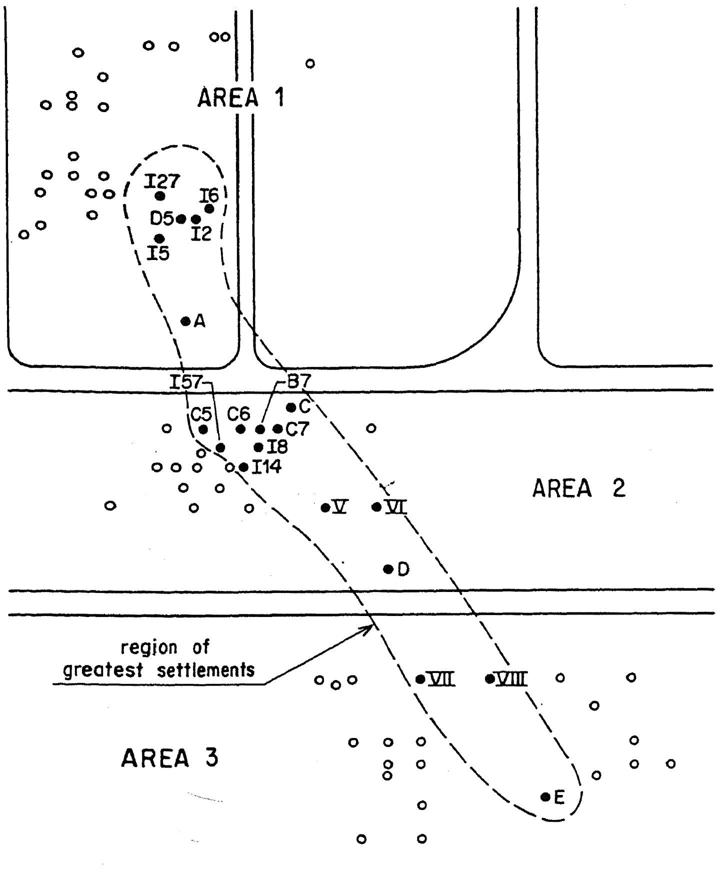

Fig. 2 - Points used for settlement analysis.

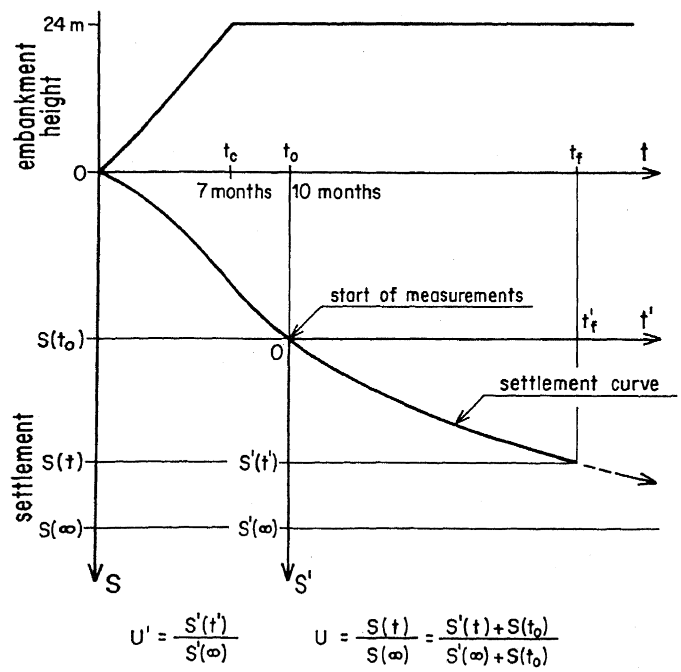

Fig. 3 - Change of settlements with time.

measured settlements $S'(t')$ did not include: a) construction settlements during the seven months construction period (roughly 1st April to 1st November); b) settlements during the three months between end of construction and start of measurements (time t_o). However it is possible to compute fairly accurately the settlement $S(t_o)$ corresponding to time t_o, as shown in the Appendix. Settlements of the compacted embankment soil, assumed to occur before settlement measurements were initiated, were considered to be negligible in comparison with the soft soil settlements.

Settlement analysis was performed using Asaoka's (1978) method. The lack of settlement data for times before t_o (see Fig. 3) have not influenced the back calculation of c_v according to Asaoka's method. As a matter of fact if these earlier data were available most of them would not be used as they usually do not define a straight line as opposed to the remaining points.

Results of the settlement analysis

Computations with Asaoka's method have been made for about 90 points. Out of these 90 points, 20 points with the greatest settlement rates were selected and these are shown in Table 1. Values of H shown in Table 1 are the thicknesses of the clay layer, as defined by the nearest SPT boring available (about 30 SPT verticals were performed). The computation of c_v using Asaoka's method requires the maximum length of drainage H_d which, for the present case, is equal to H/2. The soft clay presented a number of blow counts N equal to zero and N = 16 was typically measured for the compacted embankment. Values of c_v calculated by Asaoka's method vary from 0.8 to 4.3 x $10^{-8}m^2/s$. The average c_v value for all points shown in Table 1 is 1.6 x $10^{-8}m^2/s$.

Final settlements $S'(\infty)$ vary between 49 and 105 cm, the average value being 64 cm, as shown in Table 1. However these do not include initial values $S(t_o)$, but just measured settlements. The final settlement $S(\infty)$ can be computed from $S(\infty) = S'(\infty) + S(t_o)$ (see Fig. 3), provided $S(t_o)$ is known. The equation to compute $S(t_o)$ is developed in the Appendix. The average value of $S(t_o)$ for all points shown in Table 1 was 80 cm, thus giving an average $S(\infty)$ = 80 + 64 = 144 cm.

The average degree of consolidation $U'(t'_f)$ at the time t_f (or t'_f) of interest (May 1990) was 42% in average for all points. The time t_f(or t'_f) corresponds to the date of the last measurement available when settlement analysis was performed. However U' is not the actual degree of consolidation U (see Fig. 3). Although values of U' are of greater interest as far as measured settlements are concerned, values of U are the relevant ones regarding piezometric measurements. Computed values of $U(t_f)$ using the equations shown in Appendix are

Table 1 - Points of maximum settlements

Point	H (m)	c_v ($10^{-8}m^2/s$)	$S'(\infty)$ (m)	$U'(t'_f)$ (%)	$U(t_f)$ (%)	t_{95} (years)
B7	3.5	1.6	91	22	68	5.9
C5	2.0	0.8	54	36	71	3.7
C6	2.0	0.9	56	35	77	2.9
C7	3.0	1.8	68	31	72	3.5
I57	2.0	1.0	58	44	77	2.7
I8	2.0	1.2	58	35	76	2.1
I14	2.3	1.4	58	47	79	2.4
D5	2.0	0.9	55	46	77	2.9
I5	2.0	1.0	49	48	79	2.6
I27	2.0	0.9	53	44	76	3.0
I2	2.0	0.9	59	45	76	3.1
I6	2.5	1.3	64	42	73	3.3
V	2.5	1.8	49	48	83	2.2
VI	2.0	0.8	58	36	72	3.5
VII	4.0	4.1	50	45	80	2.5
VIII	4.0	2.2	105	27	61	5.5
A	2.0	0.9	63	54	76	3.0
C	3.0	2.0	77	54	76	3.0
D	4.5	4.3	76	52	74	3.2
E	3.5	2.2	84	46	69	4.1
Average Values	2.6	1.6	64	42	74	4.3

also given in Table 1. The average value of $U(t_f)$ for all points shown in Table 1 is 74%. However as far as measured settlements S' are concerned, the average value of $U'(t'_f)$ is 42%. Thus the magnitude of expected settlements is greater than the settlements measured up to May 1990.

The last column of Table 1 also shows values of t_{95}, but these will be discussed further below.

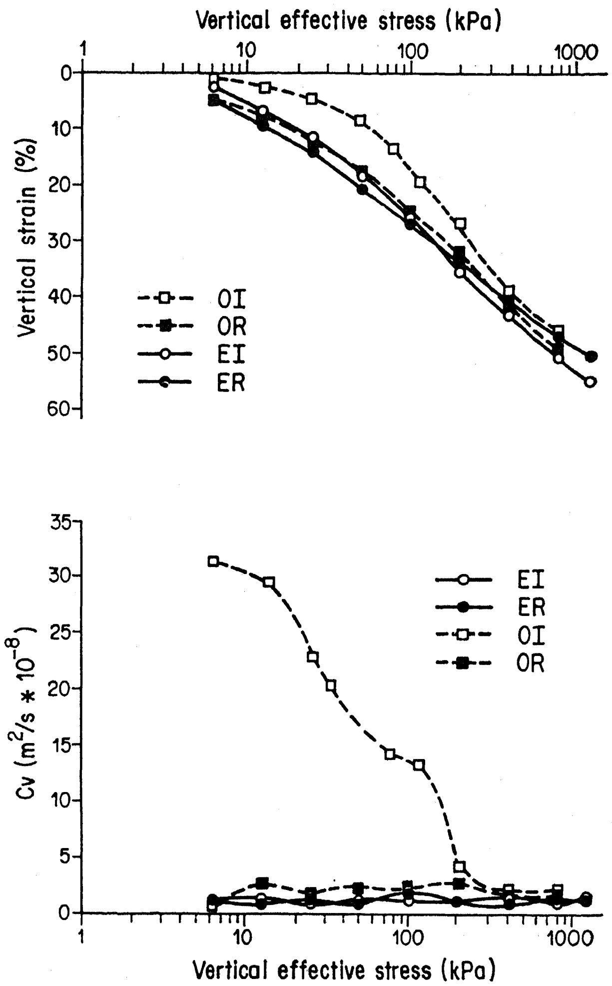

Fig. 4 - Oedometer test data.

SITE INVESTIGATION

The area chosen for additional site investigation (oedometer and piezocone testing) and piezometric studies was the region in which a thicker clay layer remained under the embankment (clay thickness varying from 3 to 5 m). Index tests performed at the design stage indicated the following average values: water content of 130%, liquid limit of 70% and platic limit of 35%.

Oedometer tests

A limited amount of oedometer tests was performed at the design stage. The test data available showed that specimens were fairly disturbed, apparently due to both the small sampler diameter and the unsatisfactory technique used for sampling.

Therefore a new programme of soil sampling was undertaken, this time not only under the embankment but also outside the embankment area. A 127 mm shelby stationary piston· sampler and well stablished sampling techniques used at COPPE were adopted. Fig. 4 shows two sets of test data: one for "E" specimens under the embankment and the other for "O" specimens outside the embankment area. Tests EI and OI are for intact specimens and tests ER and OR are for specimens fully remoulded (on purpose) in laboratory. Test data for the EI, ER and OR specimens are in the same range. However c_v data for the OI specimen are much higher than for the remaining specimens. All values of c_v shown were computed using Taylor's method.

The above results clearly suggest that the soft clay under the embankment was fully disturbed due to the unsuccessfull technique applied in removing it.

Piezocone tests

Piezocone dissipation tests were performed under the embankment using the COPPE piezocone equipment (Soares et al, 1987; Danziger, 1990) with the purpose of providing coefficient of consolidation data. As there was no interest in driving the piezocone through the embankment, a pre-borehole was made to allow access by the piezocone to the top of the soft clay layer. Dissipation test depths measured from the top of the soft clay are given in Table 2. Dissipation data were interpreted with Houlsby and Teh (1988)'s method using a rigidity index I_r = 55 (Almeida, 1982; Danziger, 1990) suitable to the local soft clay. Piezocone c_h values are given in Table 2. It is well known (Levadoux, 1980) that these values correspond to the recompression state and therefore to the overconsolidation behaviour. Values of c_h corresponding to the normally consolidated condition have been computed using the equation Levadoux (1980)

$$c_h(NC) = \frac{C_s}{C_c} c_h \text{ (piezocone)} \quad (1)$$

where C_s/C_c is the ratio between the swelling index and the compression index.

Table 2 - Piezocone dissipation tests

Test	Depth[1] (m)	c_h (piezocone) ($\times 10^{-8} m^2/s$)	$c_h(NC) = c_v(NC)$ ($\times 10^{-8} m^2/s$)
1 - 1	0.9	16	2.1 - 3.0 [2]
1 - 2	1.9	8	1.0 - 1.5
1 - 3	2.8	10	1.3 - 1.9
2 - 1	2.1	7	0.9 - 1.3
2 - 2	3.1	14	1.8 - 2.7

Note: (1) from the top of the soft clay.
(2) the range of values correspond to the range of C_s/C_c ratio.

As the soft clay under the embankment was remoulded, it is reasonable to assume a c_h/c_v ratio equal to unity, thus $c_v(NC) = c_h(NC)$ as shown in Table 2.

PORE PRESSURE ANALYSIS

Piezometric data

The following instruments were installed to provide piezometric data: 1 water level indicator, 4 Casagrande piezometers and 3 pneumatic piezometers. The analysis to be presented below is restricted to the data of the pneumatic piezometers. The clay thickness where the pneumatic piezometers were installed was 3 m and the location of the instruments with depth is shown in Fig. 5.

Excess of pore pressure measurements (Δu) at the three piezometers installed are shown in Table 3. Measured values show, as expected, a slow decrease with time.

Table 3 - Excess pore pressure data

Date of Reading	Excess Pore Pressure (kPa) PP-206	PP-207	PP-209
08/06/90	171	181	180
15/06/90	168	180	179
18/06/90	164	179	177

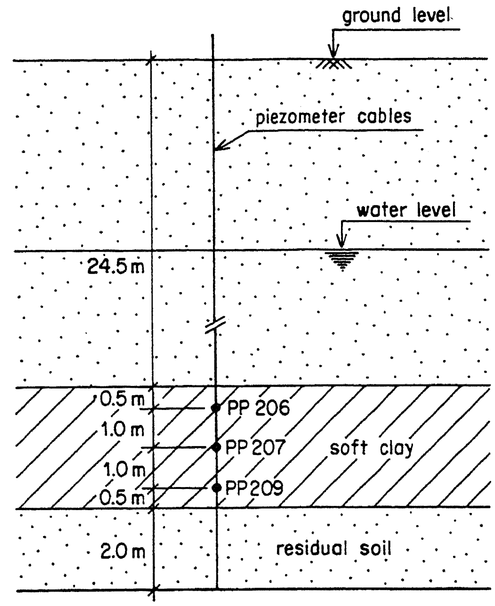

Fig. 5 - Location of the pneumatic piezometers with depth.

Average degree of consolidation

The interest in the pore pressure data was to give indications about the average degree of consolidation as a way to check the information provided by the settlement data.

The definition of the degree of consolidation U_z at a depth z assuming a linear relationship between voids ratio (or ϵ_v) and vertical effective stress (σ'_v), is given (Taylor, 1948) in terms of pore pressure is given by

$$U_z = 1 - \frac{u}{u_o} \quad (2)$$

where u is the excess pore pressure at time t and depth z, and u_o is the excess pore pressure equivalent to the vertical stress applied to the soft clay. The average degree of consolidation U can be determined by computing the area of the isocrone at a time t and the area of the rectangle corresponding to u_o, as shown in most textbooks.

The ϵ_v versus σ'_v relationship is, in the present case, strongly non linear, specially for the range of stresses (10 kPa to 300 kPa) of interest here, as shown in the example of Fig. 6. Therefore at each depth of piezometer installation, in which oedometer test data were also available, degrees of consolidation U_z have been computed assuming linear and non-linear ϵ_v versus σ'_v relationships, as shown in Fig. 6. U_z and U data are shown in Table 4 and in Fig. 7.

It is observed that the average value U for a non-linear relationship is almost twice that of the value of U for the linear relationship. The value U = 70% (June 1990) for pore pressure data compares well with U = 74% obtained from settlement analysis (May 1990).

Table 4 - U_z and U data for linear and non-linear relationships

Piezometer	U_z(%) $\epsilon_v - \sigma'_v$ linear	U_z(%) $\epsilon_v - \sigma'_v$ non-linear
PP-206	37	74
PP-207	33	63
PP-209	33	60
U(%)	39	70

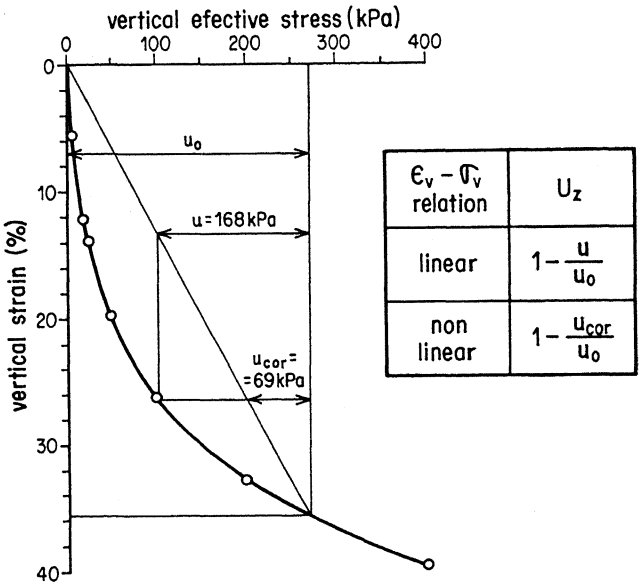

Fig. 6 Oedometer test data for specimen at the depth of piezometer PP-206.

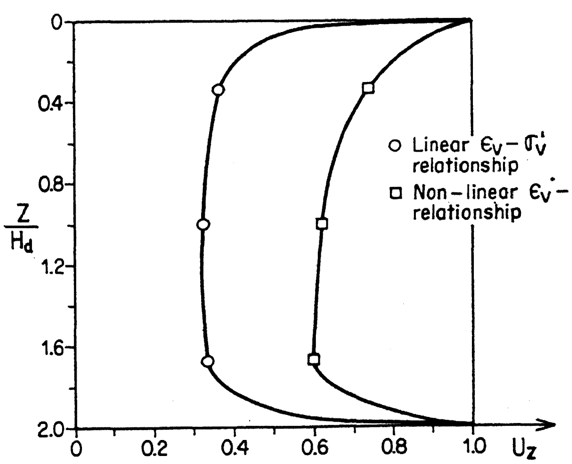

Fig. 7 Variation of the degree of consolidation U_z with depth as given by pore pressure data.

TIMES FOR SETTLEMENT STABILIZATION

The coefficient of consolidation c_v is a key parameter for the computation of times for settlement stabilization. Values of c_v have been computed here using three procedures: oedometer tests, piezocone tests and settlement analysis with Asaoka's method. The range of the values obtained with these procedures are given in Table 5.

Table 5 - Comparison of c_v values

Procedure	Interpretation Method	Range of c_v (x $10^{-8}m^2/s$)
oedometer tests	Taylor's $\sqrt{t}$	0.8 - 2.0
piezocone tests	Houlsby and Teh (1988)	0.9 - 3.0
settlement analysis	Asaoka (1978)	1.0 - 2.0

Values of c_v presented in Table 4 are all in the same range, confirming the overall consistency of the analyses presented here.

Values of the time t_{95} to reach 95% of the final settlement are given in the last column of Table 1. The date corresponding to t = 0 is July 1989, the halfway date between start and end of construction. It is observed that t_{95} varies roughly between 2 to 6 years, the average value of t_{95} being 4,3 years. Therefore settlements should be stabilized in most of the embankment area by late 1993. A number of methods of deep soil improvement were evaluated but all of them proved to be of prohibitive costs, thus the client decided to wait for settlement stabilization.

COPPE was requested by the client to provide predictions of settlement development with time for some points. Settlements S' versus time t'

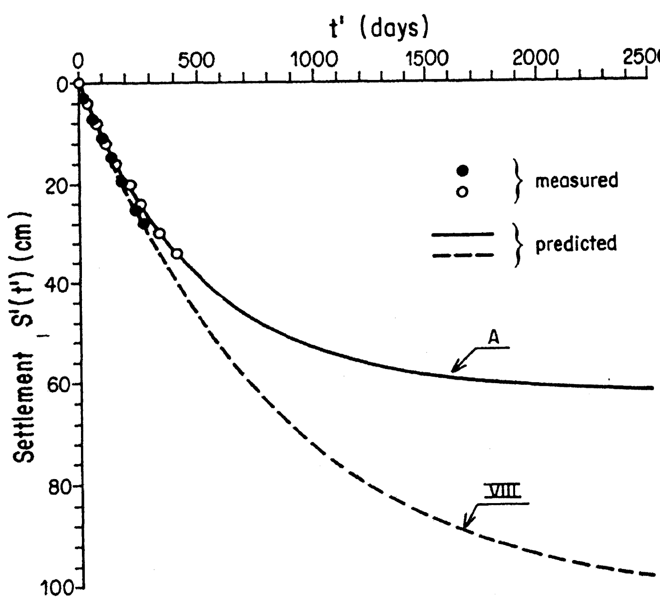

Fig. 8 Predicted and measured time-settlement curves.

(see Fig. 3) for two of these points are given in Fig. 8. Only settlements due to primary consolidation are considered here.

Settlements due to secondary consolidation were estimated to be about 4 cm for a time of 50 years and a clay thickness of 2.5 m. Therefore this is a negligible figure as it is less than 6% of $S'(\infty)$. Small secondary settlements compared with primary settlements are to be expected when the ratio $\Delta\sigma_v/\sigma_v$ of the applied to the existing load is very high as in the present case ($\Delta\sigma_v/\sigma_v$ of about 30).

CONCLUSIONS

The following conclusion can be obtained from the studies presented herein:

a) The procedure adopted to remove the soft clay from the site was thoroughly inadequate; Consequently about 2.5 m disturbed soft clay material remained under the embankment;

b) The disturbance induced to the clay caused a reduction of the coefficient of consolidation;

c) Coefficients of consolidation estimated by oedometer tests, piezocone dissipation tests and backanalysis of settlement data were in very good agreement;

d) Average degrees of consolidation estimated from pore pressure data (U = 70%) and from settlement data (U = 74%) were also in excellent agreement;

e) The average estimated time for settlement stabilization was 4.3 years, thus the embankment should stop settling by late 1993; the methods assessed for deep soil improvement were very costly, thus it was decided to wait for settlement stabilization.

ACKNOWLEDGEMENTS

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APPENDIX - Computation of U(t)

The average degree of consolidation U(t) at time t (see Fig. 3) is given by

$$U(t) = \frac{S(t)}{S(\infty)} = \frac{S(t_o) + S'(t)}{S(t_o) + S'(\infty)} \quad (A.1)$$

where $S'(t)$ is the measured settlement at time t and $S'(\infty)$ is the final settlement as computed by Asaoka's method.

The settlement $S(t_o)$ at time t_o (when measurements were initiated) is unknown but can be computed (see Fig. 3) from

$$S(t_o) = S'(\infty) \cdot \frac{U(t_o)}{1 - U(t_o)} \quad (A.2)$$

where $U(t_o)$ is the average degree of consolidation defined by the empirical equation

$$U(t) = \left(\frac{4T}{\pi}\right)^{1/2} \quad (A.3)$$

valid for U < 60%, and T is the time factor

$$T = \frac{c_v \cdot t}{H_d^2} = \frac{4c_v \cdot t}{H^2} \quad (A.4)$$

$$\text{and } t = t_o - \frac{t_c}{2} \quad (A.5)$$

for non instantaneous loading.

Substituting (A.4) and (A.5) in (A.3) one gets

$$U(t_o) = \frac{4}{H}\left[\frac{c_v}{\pi}\left(t_o - \frac{t_c}{2}\right)\right]^{1/2} \quad (A.6)$$

where c_v is the coefficient of consolidation given by Asaoka's method.

The equation to compute U(t) is equation (A.1) with $S(t_o)$ given by equation (A.2) or

$$U(t) = \frac{U(t_o)\left[S'(\infty) - S'(t)\right] + S'(t)}{S'(\infty)} \quad (A.7)$$

where $U(t_o)$ is given by equation (A.6). Values of $U(t_o)$ computed with equation (A.6) were virtually all below 60%. The average value of $U(t_o)$ for all points shown in Table 1 was 56%, i.e., 56% of the settlements had already taken place when measurements were initiated.