International Conference on Case Histories in Geotechnical Engineering

(1984) - First International Conference on Case Histories in Geotechnical Engineering

07 May 1984, 11:30 am - 6:00 pm

Embankment on Vertical Drains - Pore Pressures During Construction

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SYNOPSIS Excess pore pressures and consolidation settlements observed during the construction of a trial embankment placed on four different types of vertical drains are examined with the aim of evaluating: undrained pore pressure response, field coefficient of consolidation and drain performance.

1. During the construction of the Porto Tolle Thermal Power Plant on the extreme eastern edge of the delta of the river Po, extensive use of preloading techniques was made for a number of large steel tanks and many secondary structures. Large numbers (≅ 1700 000 m) of vertical drains of various types were installed with the aim of accelerating the consolidation of the 22 to 24 m thick layer of soft silty clay shown in Fig. 1. The geotechnical properties of this layer are shown in Fig. 1 and Table I. Further details may be found in the paper of Jamiolkowski et al. (1980).

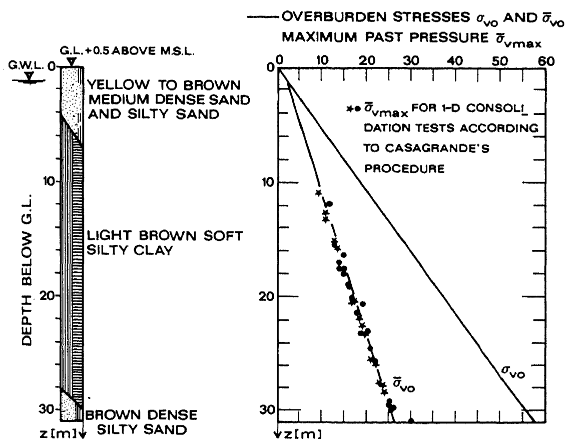


Fig. 1. Soil Profile and Stress History

TABLE I. Properties of the Porto Tolle Silty Clay (Hansbo et al., 1981)

LIQUID LIMIT	=52.6±6.9(%);	PLASTIC LIMIT	=30.9±6.6(%)
w_n	=36.4±4.9;	CLAY CONTENT	=33.9±4.2
c_u (TX-COMPR.)	≅ 0.31 $\bar{\sigma}_{vo}$;	c_u (PS-COMPR.)	≅ 0.34 $\bar{\sigma}_{vo}$
$\bar{\phi}$ (TX-COMPR.)	≅ 29°;	$\bar{\phi}$ (PS-COMPR.)	≅ 32°
K_o (COEFF. OR EARTH PRESS. AT REST)			≅ 0.5
CR (PRIMARY COMPRESSION RATIO)			≅ 0.20
TX = TRIAXIAL TEST;		PS = PLANE STRAIN TEST	

The available geological and geotechnical information led to the conclusion that this soft cohesive stratum is only a few thousand years old and has not yet been subjected to any appreciable secondary compression.
The penetration pore pressures measured by means of piezo-cone tests (Battaglio e Maniscalco, 1983) indicated that the stratum under consideration possesses a well developed macrofabric which consists of thin seems, lenses and pockets of fine sand and silty sand. However, these more permeable inclusions are discontinuous and, as a back-analysis of a trial embankment without vertical drains (Bilotta and Viggiani, 1975) showed, the soft silty clay behaved in spite of this macrofabric as one single consolidating stratum, with drainage occurring at the top and bottom.

2. One of the preloading embankments adopted (cf. Fig. 2) was used as a trial embankment for comparing the efficiency of the four types of drains installed. The characteristics of the latter are given in Table II.

TABLE II. Drain Characteristics

TYPE	SPACING (1) (cm)	DIAMETER (cm)	MANDREL DIAMETER (cm)
JETTED SAND DRAINS	500	30.0	- (2)
GEODRAINS	380	6.3 (2)	16
SOILDRAINS	380	5.5	10
SANDWICKS	420	7.0	9

(1) EQUILATERAL TRIANGULAR ARRAY
(2) EQUIVALENT DIAMETER (DRAINS ARE BAND-SHAPED)

After the embankment was placed, its behaviour was monitored by means of 45 piezometers of various types, surface settlement plates, deep settlement sensors of the below-hose type and vertical and horizontal inclinometer tubes. This instrumentation allowed the measurement of surface settlements, vertical strains, pore pressures and horizontal displacements. Some details of the measurements may be found in Hansbo et al. (1981). Unfortunately, the long-term performance of the major part of the installed piezometers was unsatisfactory in this environment because of the presence of isolated pockets of organic gas. It must be assumed that the drains installed contributed to the diffusion of the gas in the soil mass by connecting the isolated pockets.
A typical example of the unsatisfactory performance of the piezometers is shown in Fig. 3 which compares measured pore pressures with measured surface settlements and maximum lateral displacements. It appears that in spite of the progress in consolidation indicated by the surface settlements (which are combined with small lateral di-

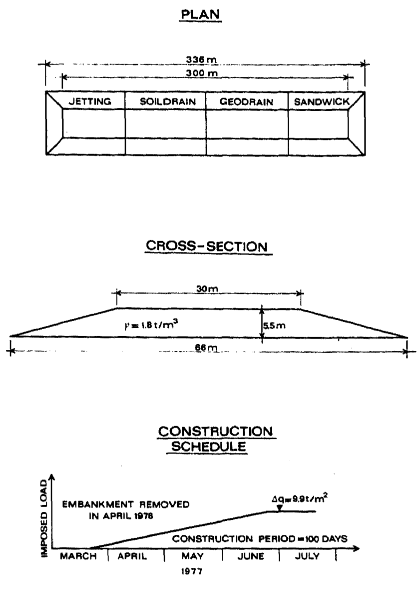


Fig. 2. Geometry of Trial Embankment

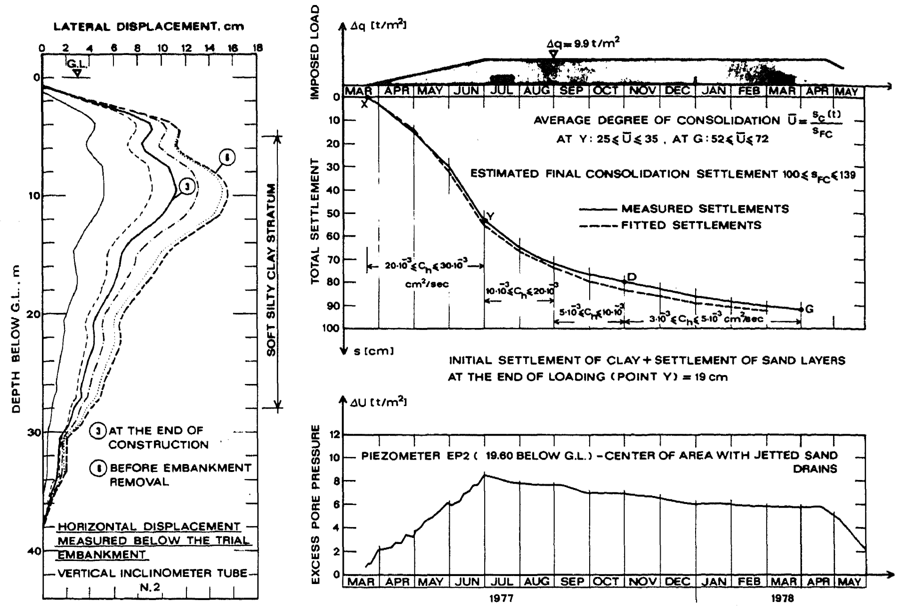


Fig. 3. Settlement and Pore Pressure Records

splacements of the soft layer), the piezometers do not indicate any appreciable dissipation of the excess pore pressures caused by the applied loads.

3. In the present paper an attempt is made of a nalysing the piezometer data measured during the construction of the trial embankment with the aim:

a. To compare the undrained excess pore pressure which may be predicted on the basis of diffe- rent theories, Δu_o^P, with the maximum excess pore pressure measured in the piezometers at the end of the loading ramp, Δu_{max}.

b. To evaluate from the piezometer readings the horizontal coefficient of consolidation, c_h, assuming that the influence of gas on the pie zometer performance is negligible during em- bankment construction when the soil is subjec ted to a simultaneous increase of both total stresses and pore pressures.

4. The prediction of the undrained pore pressure response of saturated or nearly saturated soft cohesive soils under an imposed surface load is possible by one of the following four approaches (Foot and Ladd, 1973):

a. Empirical approach which assumes that $\Delta u_o^P=\Delta\sigma_v$ (=increment of total vertical stress at the level of the considered piezometer).

b. Approach in which it is assumed that the soil behaves as an elastic perfectly plastic mate- rial governed by the Tresca failure criterion. In this case, as long as the soil around the piezometer responds "elastically", i.e. $f=\Delta\sigma_1-\Delta\sigma_3/2c_u < 1$, $\Delta u_o^P=\Delta\sigma_o$. When a condition of con tained plastic flow is reached (f=1), then $\Delta u_o^P=\Delta\sigma_v$.

$\Delta\sigma_o$ = increment of mean total stress around the piezometer

c_u = undrained shear strength of soil

$\Delta\sigma_1$, $\Delta\sigma_3$ = increments of major and minor total prin cipal stresses around the piezometer, respectively

c. Semi-empirical approach which assumes that as long as f < 1, the value of Δu_o^P is governed by the following formula:

$$\Delta u_o^P = \Delta\sigma_o + a\ \Delta\tau_o \qquad (1)$$

where:

$\Delta\tau_o$ = increment of octahedral shear stress around the piezometer

a = Henkel's pore pressure coefficient.

d. Approach in which it is assumed that the soil behaviour is in agreement with the modified Cam Clay model (MCCM), see Burland (1967) and Wroth and Parry (1977).

The evaluation of Δu_o^P by means of approaches a. to c. is quite obvious, but the use of the MCCM merits some further comments. Since the Porto To le soft silty clay is virtually NC, it may be as sumed that its current initial effective stress is at or very close to the yield surface (cf.Fig.4) and its behaviour may be approximated by that of a compl tely plastic "wet" clay under plane strain cond tions (Burland, 1967).

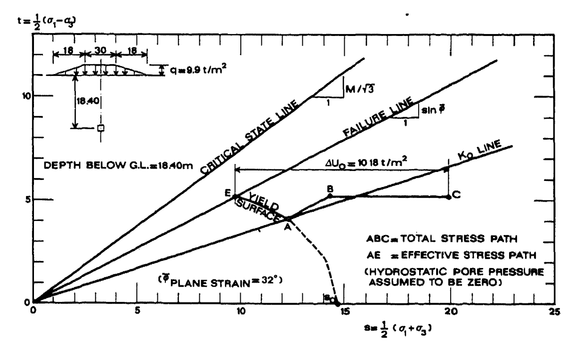


Fig. 4. Pore Pressure Prediction according to Mo dified Cam Clay Model

In such circumstances the undrained effective stress path equation coincides with that of the yield surface (Burland, 1967) (see Fig. 4):

$$\left(\frac{t}{s}\right)^2 = \left(\frac{M^2}{3}\right)\left(\frac{s_o}{s} - 1\right) \quad (2)$$

where:

$$s_o = s_i \left[1 + \frac{\left(\frac{t}{s}\right)_i}{\frac{M^2}{3}}\right] \quad (3)$$

$$s_i = \frac{\bar{\sigma}_{vo}}{2}(1 + K_o); \quad t_i = \frac{\bar{\sigma}_{vo}}{2}(1 - K_o) \quad (4)$$

$s = 1/2\ (\bar{\sigma}_1 + \bar{\sigma}_3)$; $t = 1/2\ (\sigma_1 - \sigma_3)$

M = slope of the critical-state line (CSL) as determined from a triaxial compression test

$\bar{\sigma}_{vo}$ = effective overburden stress

On the basis of experimental evidence (Burland, 1967), it was assumed that the Coulomb-Terzaghi failure criterion holds for the ratio t/s at failure:

$$\left(\frac{t}{s}\right)_f = \frac{3M}{6 + M} = \sin\bar{\phi} \quad (5)$$

where $\bar{\phi}$ is intended as the effective angle of friction for plane strain conditions. It should be remembered that $(t/s)_f$ defined by eqn. (5) is lower than the $(t/s)_{cs}$ ratio at the critical state, the latter being equal to $M/\sqrt{3}$ (see Fig. 4). Moreover, the total (plastic) stress path (AB on Fig. 4) has been computed assuming a ratio $\Delta\sigma_3/\Delta\sigma_1$ corresponding to elastic conditions.

5. As an example, Table III gives a comparison between Δu_o^P predicted by means of the four above mentioned approaches and Δu_{max} measured by the electropneumatic piezometer EP-2 located at a depth of 19.60 m below ground level under the centerline of the embankment, in a sector where jetted sand drains where installed.

TABLE III. Predicted vs Measured Excess Pore Pressure of Piezometer EP-2 (Δu_{max} = 7.85 t/m²)

	APPROACH			
	a	b	c	d
$\Delta u_{max}/\Delta u_o^P$	0.91	1.11	0.89	0.78
$\Delta u_o^P / \Delta\sigma_v$	1.00	0.82	1.03	1.17

On the basis of the data shown in Table III, the following comments can be made:

a. The elasto-plastic approach, b., leads to an unreliable prediction of Δu_o for it is obvious that Δu_{max} should be less than Δu_o^P, particularly in the presence of vertical drains.

b. As far as approaches a. and c. are concerned, they satisfy the above condition; however, they lead to a degree of consolidation $U_h = 1 - u_{max}/u_o^P$ of 0.1, approximately, at the end of the loading ramp which is too low if compared with the consolidation settlements of the soft cohesive stratum measured in the same period of time (cf. Fig. 3).

c. The evaluation of Δu_o^P by means of the MCCM approach leads to $\Delta u_{max}/\Delta u_o^P$ ratios which are qualitatively in good agreement with the values of the consolidation settlements observed at the end of the loading ramp.

d. The latter approach indicates $\Delta u_o^P/\Delta\sigma_v$ ratios which are consistently higher than 1. This may be attributed at least partly to the occurrence of contained plastic flow and even a slight strain softening of the in situ clay of Porto Tolle. Additional experimental evidence supporting the above observations may be found in Table IV in which Δu_o^P, obtained on the basis of the MCCM approach, is compared with the values of Δu_{max} and $\Delta\sigma_v$ of other three piezometers.
The same table also shows the time from the beginning of the loading ramp after which the soil around the piezometers reached a state of contained failure (f=1).

TABLE IV. Examples of Excess Pore Pressures Predicted by the MCCM Approach

SECTOR	PIEZO-METER Nr.	DEPTH BELOW G.L. m	$\Delta\sigma_v$ t/m²	Δu_o^P t/m²	Δu_{max} t/m²	$\frac{\Delta^P_{uo}}{\Delta\sigma_v}$	$\frac{\Delta u_{max}}{\Delta u_o^P}$	t_f days
GEODRAINS	BAT-2	15.27	9.05	10.27	7.32	1.13	0.71	32
GEODRAINS	BAT-3	18.40	8.70	10.18	7.73	1.17	0.76	36
JETTING	EP-2	19.60	8.60	10.10	7.85	1.17	0.78	38
JETTING	BP-2	24.10	7.90	9.72	7.32	1.23	0.75	46

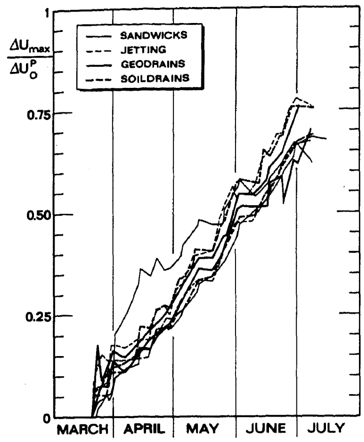
BAT = ELECTRIC PIEZOMETER

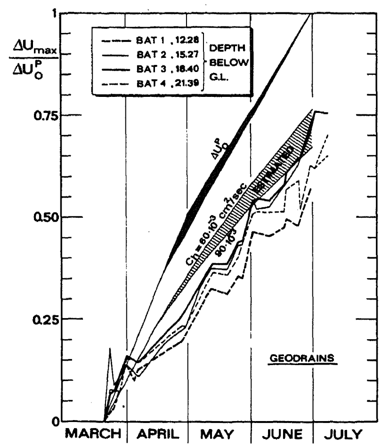
EP = ELECTROPNEUMATIC PIEZOMETER

6. Fig. 5 shows the range of measured values of Δu_{max} (normalized with respect to Δu_o^P) evaluated by the MCCM approach, as recorded under all four sections of the trial embankment. From this figure it may be argued that, although different types of drains are involved, the response of the normalized pore pressure is substantially similar for all piezometers at the end of the loading ramp, with $\Delta u_{max}/\Delta u_o^P$ ranging from 0.75 to 0.65. These values, corresponding to a local degree of consolidation for radial flow of $U_h \cong 0.25$ to 0.35, compare well with the average degree of consolidation $\bar{U}$ estimated on the basis of the observed consolidation settlements shown in Fig. 3. The $\bar{U}$ values were estimated from a range of CR comprised between 0.175 (lower bound of laboratory determined values) and 0.24 (average CR corrected for sample disturbance according to Schmertmann (1955).
The substantially uniform pore pressure response of all piezometers (Fig. 5) led to the conclusion that, at least during construction of the trial embankment, the performance of the prefabricated drains was the same as that of the jetted sand drains.
This evidence is of practical interest in that it shows that:

a. The well resistance of the drains does not affect the consolidation process. This statement is also supported by the fact that the ratio $\Delta u_{max}/\Delta u_o^P$ for the piezometers installed at different depths between the prefabricated drains is substantially independent of depth; see for example Fig. 6 for band-shaped Geodrains.

b. Fig. 6 confirms indirectly that the high discharge capacities recently measured (Hansbo, 1983; Jamiolkowski et al. 1983-b) for these drains in the laboratory may be considered to be reliable, at least for a short period after drain installation.





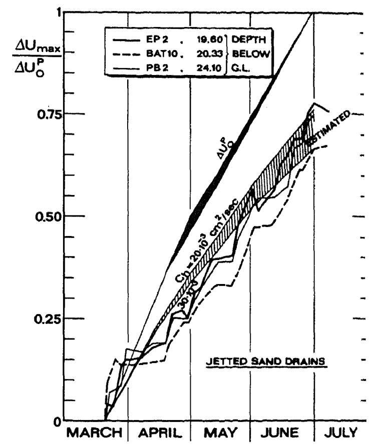


Fig. 5. Normalized Pore Pressures Measured During Construction Loading

Fig. 6. c_h Values from Back-Analysis for Area of Geodrains

Fig. 7. c_h Values from Back-Analysis for Area of Jetted Sand Drains

The above statements may not be entirely true for a longer period of time when the prefabricated drains suffer some deterioration.

7. Assuming that the MCCM approach predicts reliable values of Δu_o^P and considering radial pore water flow only, it is possible to evaluate c_h controlling consolidation during the construction period. This has been done using the simple uncoupled radial consolidation theory which allows to take into consideration a linear increase of the surface load with time (Schiffman, 1958). The resulting c_h values for the jetted sand drains are shown in Fig. 7. If for these drains, instead of using Δu_o^P according to the MCCM, Δu_o^P is assumed to be equal to $\Delta\sigma_v$, one obtains for the same piezometers $c_h \cong$ (7 to 15) 10^{-3} cm^2/sec; c_h which allows to fit the rate of the consolidation settlement from point X to point Y in Fig. 3 is equal to $\cong$ 20-30 . 10^{-3} cm^2/sec.

As far as the prefabricated drains are concerned, one obtains the same range of c_h values if the soil disturbance caused by drain installation is neglected. If, however, for these drains the existence of a less permeable smear zone around the drains is assumed, one obtains the very large c_h values shown in Fig.6.

The above clearly shows how difficult and uncertain the evaluation of c_h may be in the presence of vertical drains, even in the case of a well instrumented trial embankment as the one considered here. The problem becomes particularly complex in the case of prefabricated drains where, in addition to the effects of possible well resistance and drain deterioration, various other factors must be considered, such as the problem of accurately estimating Δu_o^P, smear around the drains and its influence on soil permeability, and also errors in estimating the distance between the considered piezometer and the nearest drain. In the case considered here, the estimate of c_h was also difficult because of the presence of gas and its effect on the piezometers.

In any case, the following conclusions are possible:

a. The observed settlement rates and the measured pore pressure responses during embankment construction both indicate a substantially similar performance of all four types of installed drains.

b. The pore pressures measured during construct indicate that prefabricated drains are not su jected to the phenomenon of well resistance.

c. The evaluation of c_h on the basis of Δu_{max} leads to "reasonable" values of the consolida tion coefficient for jetted drains. The same plies also to prefabricated drains if the effect of smear is neglected.

d. The estimate of c_h on the basis of pore pres re response strongly depends on the assumed v lue of Δu_o^P.

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