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SYNOPSIS: The result obtained in a case history of differential settlement recovery are reported. The effectiveness of sand drains in peaty soil is questioned. The importance and the mutual influence of the primary and secondary settlement are discussed. Remarks are made on the value of secondary settlement coefficient.

INTRODUCTION

The authors presented to the First International Conference on Case Histories in Geotechnical Engineering a paper concerning the recovering of differential settlements in a large six storey building by means of vertical drains and partial overloading of the foundation mat.
The details of the design together with general soil characteristics, settlement measurements up to November 1983 and observational analysis of settlements are reported in the mentioned paper. But the limited available space did not allow an accurate analysis of the settlements behaviour with respect to the soil characteristics: this will be done in this paper. Furthermore an other measurement of the settlements has been made in July 87, so that more precise forecast for the future behaviour is now possible.

SOIL CHARACTERISTICS

The general soil characteristics have been reported in the previous paper. It has to be stressed that the area was a marshland until a few years before the building construction and that propably the building was located on the border of the said area. In fig. 1 the lay-out of the foundation mat is reported with the boreholes and CPTs performed at the time of the intervention (1974). Borehole 1 and CPTs A and B showed a soil with characteristics substantially different from those of borehole 2 and CPTs C and D (see previous paper). In fact the side of the building interested by these latter tests was close to the water stream flowing at about the center of the marshy area.
In 1978 a new investigation was performed, consisting of one borehole, one electric CPT and one DMT (Marchetti Dilatometer Test). These tests were made in the vicinity of the existing borehole 2 (fig. 1) but at a sufficient distance from the building not to be affected by its load.

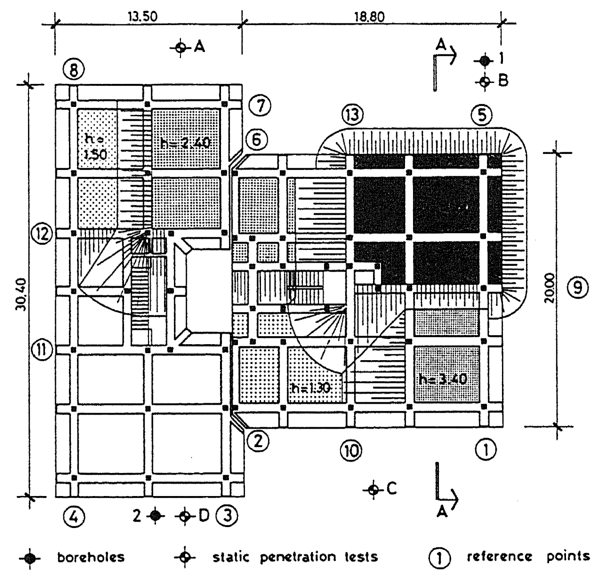

Fig. 1 Lay-out of the Foundation Mat with the Height (h) of Sand Loading and Reference Points

In fig. 2 are reported the profiles of CPT and DMT. In the borehole samples some characteristics resulted as follows:

sample depth (m)	peat content (%)	γ_0 (t/m^3)	w_0 (%)	w_L (%)	w_P (%)	c_u (t/m^2)	C_R
2.15 - 2.55	-	1.83	39	55	18	2.6	0.153
4.00 - 4.35	-	1.38	103	-	-	-	-
4.35 - 4.55	-	1.79	43	57	17	2.3	0.196
6.00 - 6.60	17.4	1.60	65	103	24	2.9	0.306
10.50 - 11.05	13.5	1.77	43	67	18	3.4	0.246
14.70 - 15.25	-	1.70	50	74	21	4.2	0.272
20.00 - 20.55	11.6	1.66	54	83	22	6.0	0.323
23.10 - 23.60	-	1.80	38	70	19	5.3	0.230
29.20 - 29.40	-	1.88	31	39	17	3.4	0.166
32.30 - 32.80	-	1.88	29	38	18	5.5	0.172

The values of cu were determined by laboratory vane tests: the trend is linear from 2 t/m2 at 2m depth to 4 t/m2 at 15 m. The samples resulted slightly overconsolidated down to about 10 m. C_R is the compression ratio ($\frac{\Delta H}{H}$ /log t) on the virgin curve in the compression range ($p > p_c$). As far as the peat content is concerned, and hence the compressibility, the layer between 5 and 10 m approximately resulted critical; this is in accordance with what had been obtained for borehole 2.

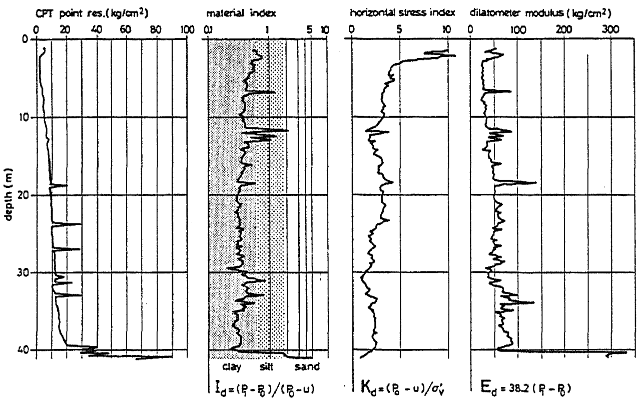

Fig. 2 CPT and DMT Tests in the Vicinity of Borehole 2

SETTLEMENT BEHAVIOUR

The drains were installed from 1st August to mid October 1974; in fig. 3 are sketched the diagrams of settlement for two opposite corners of the building, together with the drains execution period and the four loading steps. These steps with reference to the maximum height of sand loading on the foundation mat (fig. 1) were:

1st step Hmax = 1.10 m
2nd step = 2.20 m
3rd step = 3.50 m
4th step = 4.50 m

The partial unloading refers to a reduction of the maximum height of sand to 2.20 m all over the area; the partial reloading refers to a restatement of the height of 4.50 m in proximity to point 5 (fig. 1), but without the external embankment. On the diagrams in fig. 3 some interesting remarks can be done:

1. during the drains execution, which began from the corner of point 5, the point 4 showed a heave: this was due to the great rigidity of the building structure
2. after the 4th loading step, which interested only the area near to point 5, point 4 showed a complete stop of settlement for about two months (March and April 75), but afterwards it started again to settle. Point 5 in correspondence of the partial unloading

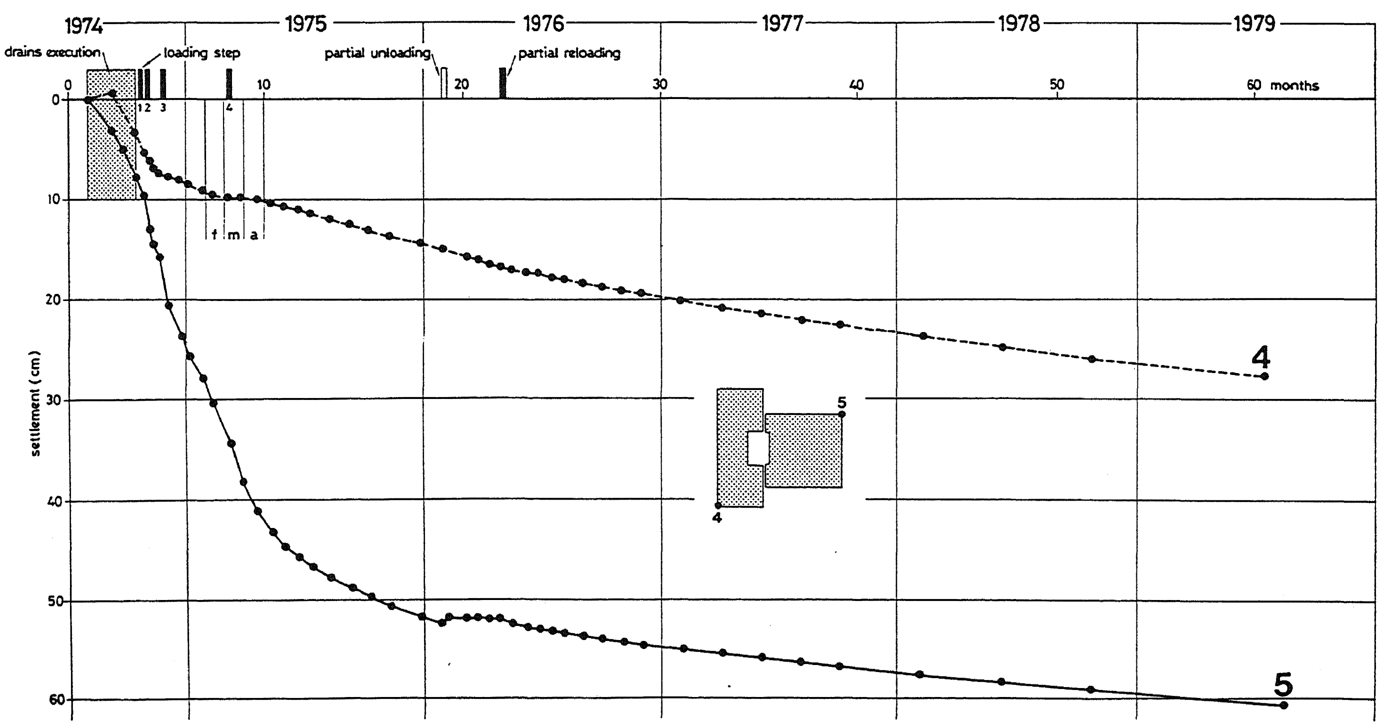

Fig. 3 Settlements of Two Opposite Corners of the Building

(from 4.50 m height of sand loading to 2.20 m) experienced a slight heave and then a complete stop of settlement. This phenomenon has been observed by many authors (e.g. Veder and Prinzl, 1983) and is employed in the technique of soft clays overloading, mainly to reduce secondary settlement

3. the settlement rate of point 4 was affected by the presence of the drains only during and immediately after their execution. From middle 1975 to 1979 the settlement behaviour seemed to be insensitive to the presence of the drains. In the previous paper it was attributed to the clogging of the drains by organic matter. The authors believe that this deduction and hence the questionability of the use of small diameter (10 cm) sand drains in peaty soil must be maintained

4. it is important to outline that the settlement behaviour was controlled not only by the soil characteristics, but also by the building rigidity. In Fig. 4 are reported the isochrones of settlement at July 79, which are almost perfectly parallel. The settlement recovering at that time between points 4 and 5 was about 32 cm.

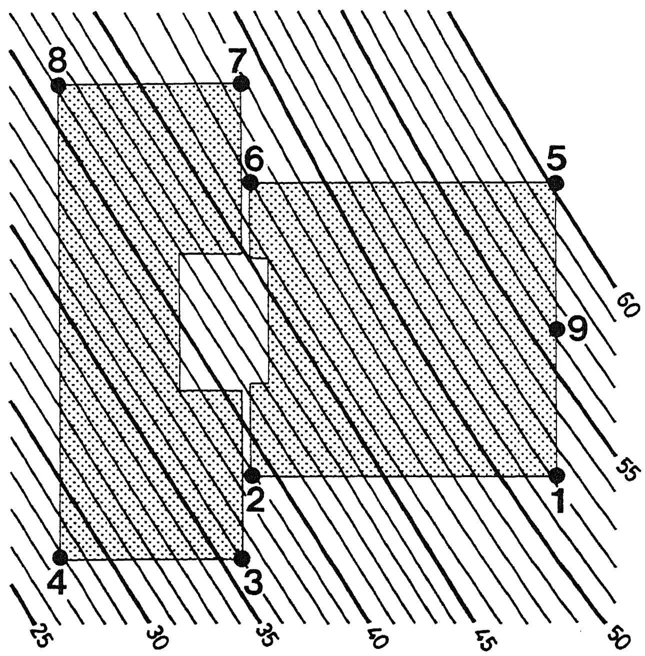

Fig. 4 Isochrones of Settlement at July 79

PRIMARY AND SECONDARY SETTLEMENT

The question about the settlement behaviour of point 4 was whether or not this was essentially secondary settlement. The primary settlement was calculated on the basis of the compression ratio values obtained from oedometer tests (in fig. 5 is reported the oedometer curve for the sample at depth 8.70-9.30 m). The stresses induced by the foundation were calculated by the method of Fox. The settlement calculation was limited to 28 m depth.

In the following table are reported the data of the calculation:

layer	thickness (m)	C_R	P_0 (kg/cm²)	ΔP (kg/cm²)	settlement (m)
1	2.00 - 4.65	0.2117	0.256	0.85	35.55
2	4.65 - 6.10	0.2588	0.412	0.76	17.04
3	6.10 - 11.10	0.3189	0.612	0.66	50.66
4'	11.10 - 16.00	0.1785	0.934	0.53	17.07
4"	16.00 - 20.10	0.1785	1.257	0.43	9.35
5	20.10 - 28.00	0.3388	1.682	0.32	20.24
					149.91

A calculation of the primary consolidation rate was done without drains and taking into account the presence of a layer of fine silty sand at 18-20 m depth. So the following scheme of consolidation was assumed:

- layer 1 from 2 to 10 m flow upward
  $C_{Vav} = 2,25 \cdot 10^{-4}$ cm²/sec.
- layer 2 from 10 to 18 m flow downward
  $C_{Vav} = 1,01 \cdot 10^{-3}$ cm²/sec.
- layer 3 from 20 to 28 m flow upward
  $C_{Vav} = 4,2 \cdot 10^{-4}$ cm²/sec.

It resulted that at July 74 (14,5 years after the beginning of the construction) about 40 cm of the primary settlement had yet to develop, of which 35 cm due to layer 1 and 5 cm to layer 3, while layer 2 had almost completed primary consolidation.

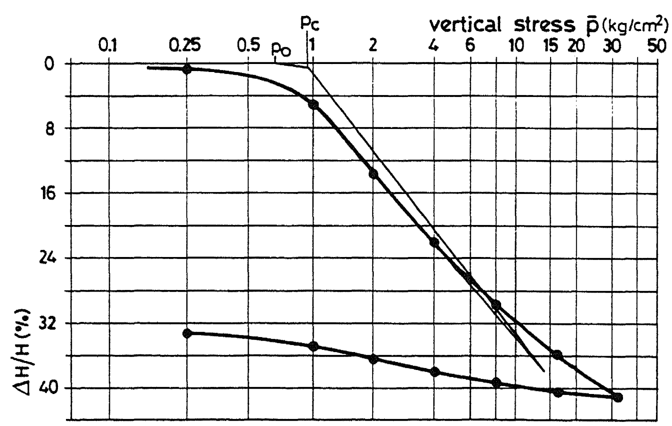

Fig. 5 Oedometer Curve for the Sample at Depth 8.70-9.30 m

In fig. 6 are reported the settlements from August 74 to July 87 of four significant points.
In fig. 7 the settlements of point 4 are reported versus log time. The estimated settlement of 130 cm at July 74 is based on:

1. out of level of 90 cm measured at July 74 between points 4 and 5
2. estimated settlement of 30 cm for point 5 (rough measurements made by the owner of the building gave 16 cm from January 63 to July 74)
3. evaluated immediate settlement of about 15 cm.

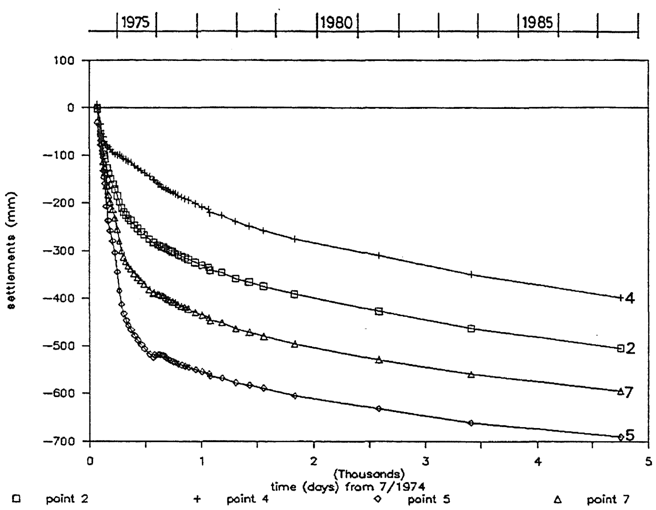

Fig. 6 Measured Settlements of Four Points

The curve in fig. 7 indicates:

1. the drains had little influence on the settlement rate of point 4, as also Asaoka analysis showed (see previous paper)
2. the primary and secondary parts of the curve seem to be well distinct. It is likely that the primary consolidation has been completed at about the end of 1979, 20 years after the beginning of the construction . The value of the primary settlement from August 74 to the end of 79 (40 cm approx.) fits very well with the calculation of the consolidation settlement amount and rate above reported.

In the previous paper a forecast of the secondary settlement rate was done, based on the values of the coefficient of secondary compression c_α given by the oedometer tests. The measurement of July 87 has invalidated that forecast and a new one is reported in fig. 7.
This new forecast corresponds to a value of c_α = 0.0125, but this value in laboratory has been obtained as a maximum only for the sample at 8.60 - 9.30 m depth. For the other samples, values vary from 1/3 to 2/3 of the above maximum value.

Thus we must conclude that the in situ creep is in our case remarkably higher than the laboratory one.
This has already been noted in many case histories and the most likely reason is the collapse of the soil structure as the effective stress approaches the final value (Chang, 1981). It has to be noted that in our case the coefficient of mobilization of the ultimate strength was high ($c_u \simeq 2$ t/m2; net pressure q = 0.85 kg/cm2; coefficient of mobilization f = 0.75).

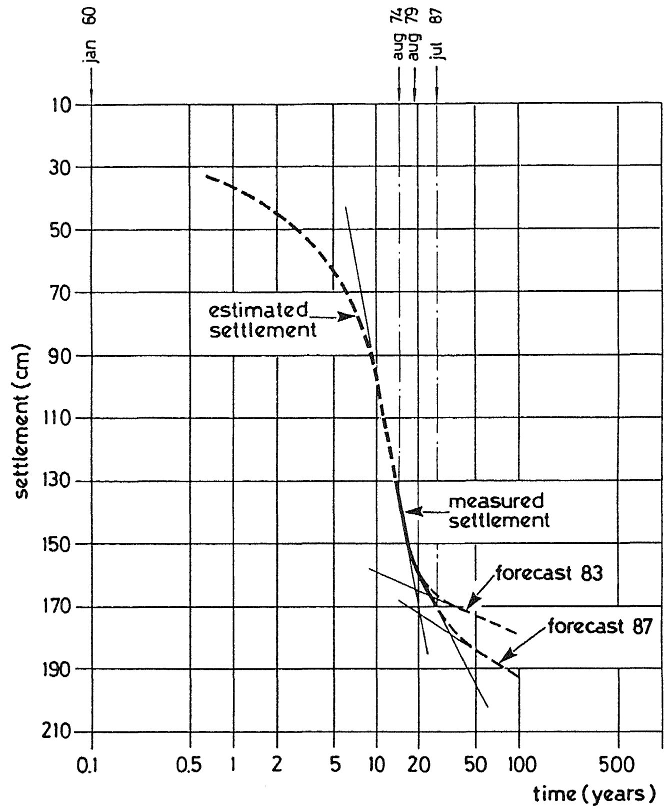

Fig. 7 Tentative Curve of the Settlement of Point 4 versus Log Time

CONCLUSIONS

The use of small diameter sand drains resulted not effective in the area affected by medium-high peat contents. On the contrary in the remaining part of the foundation the results were fully satisfactory.

The consolidation settlement and its rate, calculated on the basis of the parameters obtained from incremental oedometer tests, were well fitting with the measured values. It has to be said that it was very attractive to use the Bjerrum's concept of instant and delayed settlement to explain the unexpected settlement of point 4; but the agreement of calculated and measured values seems to be in accordance with the conclusions of Mesri and Choi (1985) about the uniqueness of end of primary consolidation.

Secondary settlement in situ resulted sensibly higher than in laboratory tests. This is an already known phenomenon and has to be attributed mainly to the high coefficient of mobilization of ultimate strength.

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