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SYNOPSIS : A preloading embankment, and its foundation soil, have been closely monitored in the Po river delta (Italy). After showing that the complete consolidation of the peaty and clayey soil has taken place, the shear strength increase measured with various tests has been considered and analyzed.

PLANNING PROBLEMS AND SOIL CHARACTERISTICS

The new electric power station in Adria situated in the Po delta has been recently built in an area of about 70,000 m^2, (210 x 315 m). This area is located in the sorroundings of the navigable canal "Canal Bianco" and is distant about two kilometers from the Po river. Because of hydraulic protection problems against floods, due to the fact that the area was subject to a subsidence phenomenon in the past, the design of the final plan of the station provided for its being raised from ground level, situated at -1 - -2.5, to +1 a.s.l., by means of a large sand filling. Stratigraphy from the surface downwards was as follows:

- from 0 to -2m : over-consolidated silty clay of low plasticity; (Formation 1)
- from -2 to -5m: mainly peaty soil of high plasticity composed of not very fibrous peats mainly of amorphous and granular type (Amaryan et al. 1973) (unity weight 10 ÷ 14 kN/m^3, liquid limit PI > 100 and water content W > 120); (Formation 2)
- from -5 to -8m: normally consolidated silty-clayey soils of medium plasticity; (Formation 3)
- from -8 to -14m: fine, sometimes silty, sands of medium density; (Formation 4)
- from -14 to -21m: alternating layers of fine sands and normally consolidated silty clay of medium and high plasticity; (Formation 5)
- from -21 to -30m: homogeneous medium fine grained sands of medium and high density; (Formation 6).

The foundation soils showed very poor characteristics of strength and high compressibility. Figure 1 shows a stratigraphic profile with some characteristics determined through a wide research survey in the laboratory and "in situ".

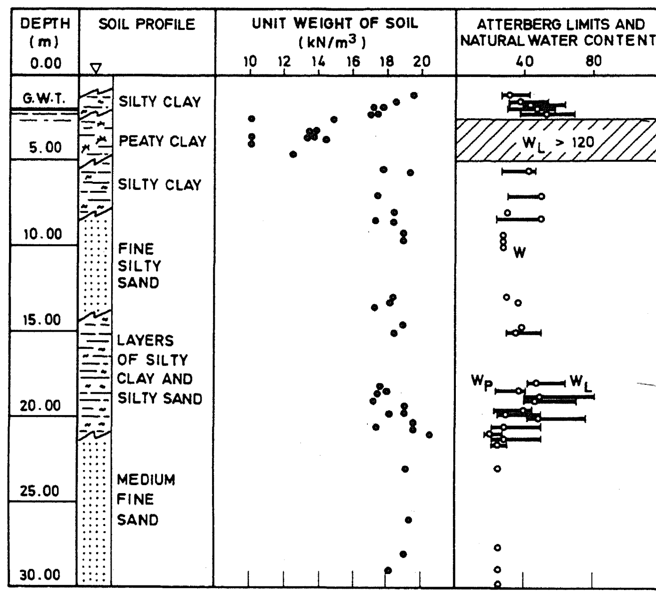


FIG. 1 - Some geotechnical properties of soil formations.

Therefore, the embankment foundation soil posed problems because of considerable settlements deferred in time due to the presence of plastic clay layers and organic soils. Organic soils of similar nature to the one found in this site have been studied by Colleselli et al. (1975).

It was decided to create a preloading embankment raising the design embankment of about 3 m (to level +4), which meant 5 ÷ 6.5 m in total height.

Figure 2 shows the general plan of the preloading embankment and figure 3 shows a cross section of the embankment side berm built in order to ensure the stability of the embankment itself.

The purpose of preloading was to reduce the compressibility and to improve the characte-

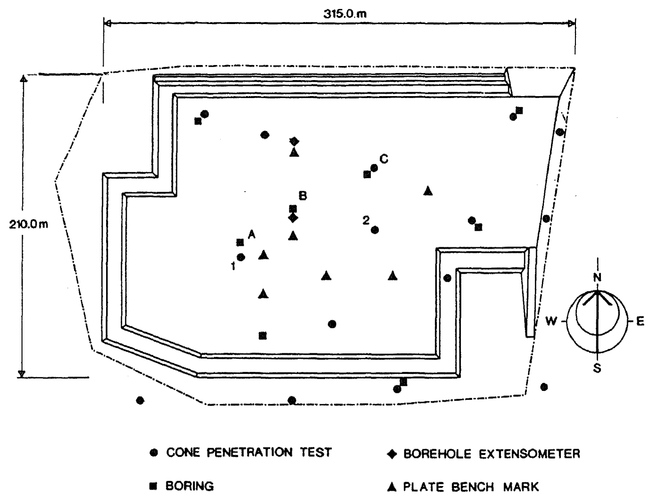


FIG. 2 - General plan of the preloading embankment.

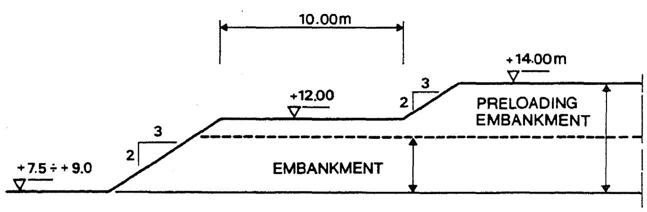


FIG. 3 - Cross section of the preloading embankment. Sea level has been taken as + 10.0 m.

ristics of shear strength of the soil, thus allowing realization of buildings with shallow foundations.

The preloading design presented problems connected mainly with the prevision of settlement pattern in time and of the behaviour of peat soil layers under loading action.

Still during the planning stage, total settlements due to secondary compression in the order of 2 ÷ 3 cm per year were estimated in the various zones, they were related to the height of the embankment and to the thickness of the compressible layers.

Moreover, the preloading embankment was to be mantained for about 20 months assuming that 90 percent of primary consolidation would mature in roughly 500 days.

In order to control the foundation soil behaviour during all of the preloading phases, a remarkable geotechnical instrumentation consisting of plate bench marks, borehole extensometers and Casagrande and electro-pneumatic piezometers was incorporated in the design.

The whole sand embankment of about 500,000 m^3 in volume was built between July 1980 and January 1981 and was kept until the end of 1982.

The embankment was then removed to +1 (corresponding to +11 in figure 3), final design height, and the construction of the buildings of the electric power station began. It was completed at the end of 1983.

Before removing the preloading, a new research survey was carried out both in situ and in the laboratory in order to control the effects of the foundation soil improvement.

In this paper the increase of foundation soil mechanical properties due to the consolidation of the foundation soils themselves due to preloading is maily considered.

PRELOADING EMBANKMENT BEHAVIOUR

In order to calculate the preloading embankment settlements, the compressibility parameters (m_v) of cohesive and peat formations have been obtained from oedometric consolidation tests.

Figure 4 shows the values of m_v as a function of effective stresses. The parameters for the settlement calculations have been chosen as function of geostatic stresses σ'_{vo} and of vertical stresses induced by overloading $\sigma'_{vo} + \Delta\sigma'_v$ and calculation was carried out considering the hypothesis of monodimensional consolidations, which correspond to the geometry of the case examined.

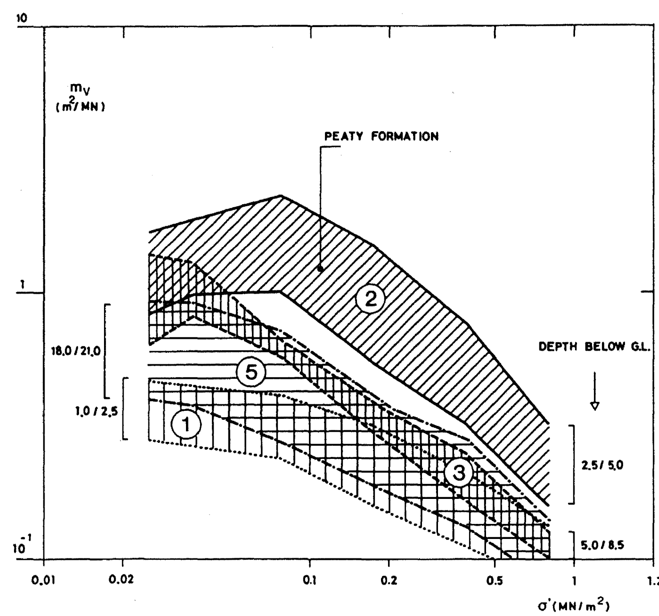


FIG. 4 - Trends of the m_v values of various formations vs. vertical effective stress.

The preloading embankment behaviour has been monitored with a considerable number of plate bench marks and borehole extensometers.

Figure 5, referring to a central point of

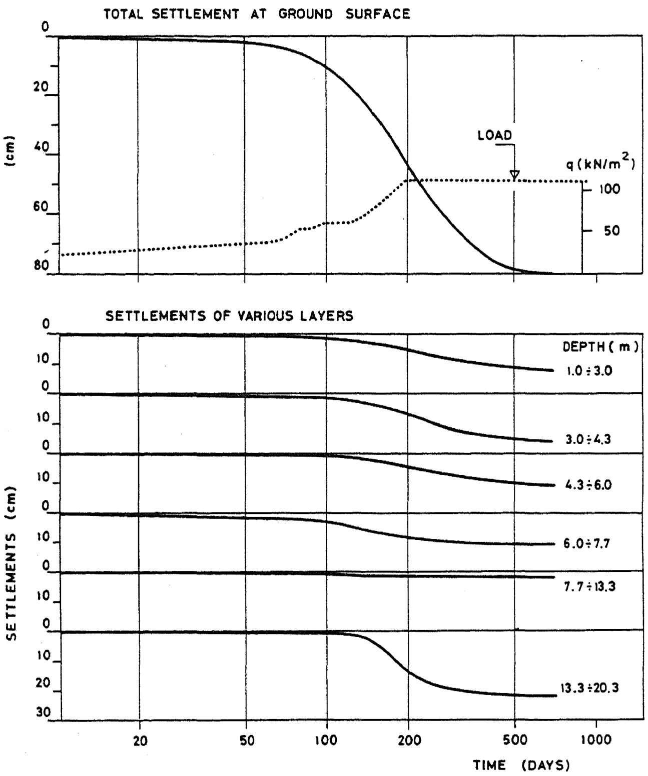


FIG. 5 - Trends of vertical settlements vs. consolidation time.

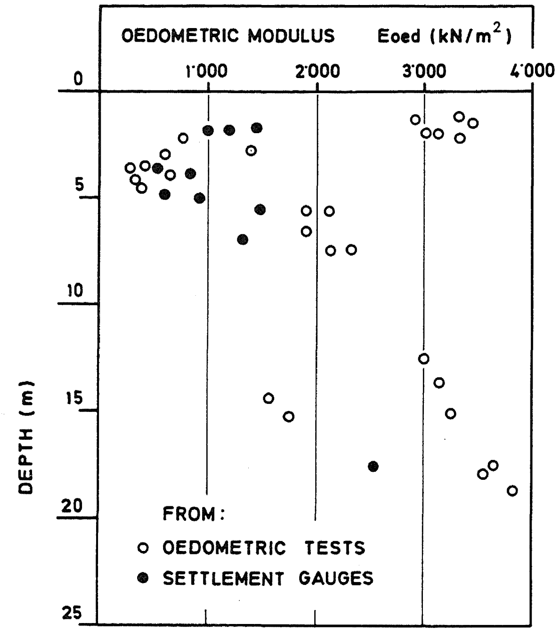


FIG. 6 - Comparison between in situ and in the laboratory constrained moduli.

the embankment area, summarizes the settlement pattern in time of the foundation plan, and of the various layers, obtained through the difference measurements given by deep settlements gauges. The deep settlement gauges showed that soils at a depth greater than 30 m from ground level are responsible for a portion of the settlement equal to about 5 - 10 percent of total settlement.
From the diagrams it can be noticed that the settlement due to primary consolidation ended completely within 600 days from the beginning of the embankment realization.
From the examination and interpretation of figure 5 and of data from other deep settlement gauges, it was possible to estimate soil deformation moduli. In figure 6 the calculated moduli are compared with those established by laboratory tests: a substantial agreement between measurements in situ and in the laboratory with variations of ± 25 percent can be noticed.
At the end of the preloading permanence period, after about 20 months, settlements varying from 60 to 110 cm matured in the foundation soil.
Measurements of electropneumatic piezometers placed in clayey and peaty layers have given values of maximum pore pressures after construction of 20÷30 kN/m². These pressures dissipated in about 18 months.
Once the preloading was removed to +1, a 2 - 4 cm swell of foundation soil occured. To date, after the completion of the construction of the buildings, which have a medium weight of 20 kN/m², distributed on the embankment, a constant, and continuous, practically uniform, settlement o 1 - 2 cm/year occurred.

IMPROVEMENT OF THE MECHANICAL CHARACTERISTICS OF SOIL STRATA.

The complete primary consolidation of cohesive and peaty layers due to preloading embankment action was recorded with plate bench marks and borehole extensometers. The superficial pressure of the embankment, about 6 m high, turned out to be equal to 105 kN/m². In the central part of the embankment this vertical stress is practically constant in the first 20 metres of soil which is the thickness our experimental research is concerned with.
As is well-known, the increase in vertical effective stresses causes a corresponding increase of shear strength in cohesive and granular materials. This strength increase was examined with routine laboratory tests (i.e. unconfined compression tests) and with routine in situ tests (i.e. CPT and Field Vane) carried out on specimens or on soil layers before and after the action of the preloading embankment.
In the cohesive soils examined the measured strength parameters are of the undranined type. The results of the different types of tests, will be examined separately.

UNCONFINED COMPRESSION STRENGTH

Unconfined compression tests have been carried

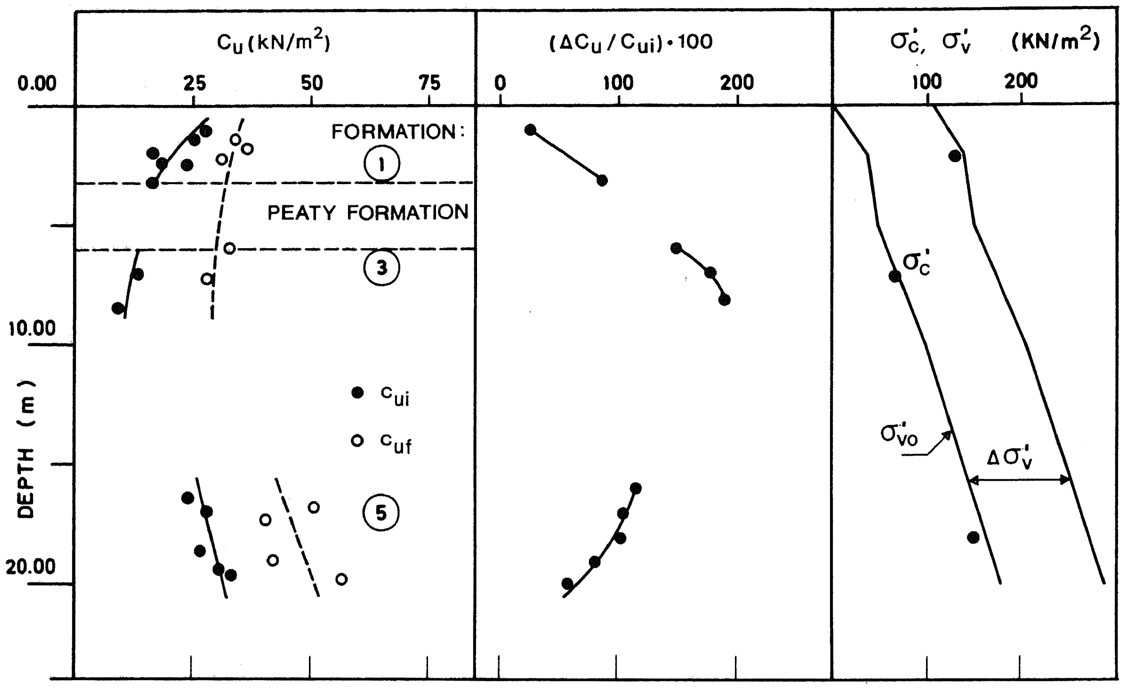


FIG. 7 - Variation of unconfined compression strength in clayey formations due to the increase in vertical effective stresses.

out, on specimens of the cohesive formations no. 1, 3 and 5, both before and after the preloading of the embankment.
The results of the performed tests have been summarized in fig. 7. c_{ui} are the values of the initial parameters while c_{uf} are the final ones. From these results the increase in shear strength is clearly visible and varies from 25 percent to almost 190 percent depending on the vertical effective overburden stresses and on the preconsolidation pressures acting at various depths as shown below.
From the research of Ladd e Foott (1974), Ladd et alii (1977) and Ladd (1982), the undrained shear strength can be expressed by the empirical relation :

$$c_u / \sigma'_{vo} = S\ (OCR)^m \qquad (1)$$

where σ'_{vo} is the vertical effective stress,
$S = c_u/\sigma'_{vo}$ at OCR = 1,
OCR = overconsolidation ratio = $\sigma'c/\sigma'vo$ and
m = 0.8 ± 0.05 (empirical - experimental parameters as given in the above quoted references).

The well accepted Skempton equation (1957) has been utilized to express S, such as:

$$S = c_u / \sigma'_{vo} = (0.11 - 0.37\ PI) \qquad (2)$$

For plasticity indexes ranging from 20 to 35 S can be taken as:

$$S = 0.21 \pm 0.03$$

and equation (1) can be rewritten as:

$$c_u = 0.21 \times \sigma'_{vo} \times OCR^{0.8} \qquad (3)$$

so that the overconsolidation ratio of a soil specimen can be determined, if its c_u and the values of the vertical effective stresses are known, by the relation :

$$OCR = (c_u/(0.21 \times \sigma'_{vo}))^{1.25} \qquad (4)$$

Relation (4) and the data of fig. 7 have been utilized to determine the OCR preconsolidation pressure (σ'_c) for the specimens at the depth of 2, 7 and 18 m below ground level as reported in Table 1.

TABLE 1

Depth (m)	σ'_{vo} (kPa)	c_{ui} (kPa)	OCR	$\sigma'c$ (kPa)
2	36	22	3.8	137
7	65	13	1.0	65
18	160	30	0.9	144

The values of the preconsolidation pressures have been plotted on fig. 7.
It can be seen that above the peaty formation the clay is well overconsolidated while below it results normally consolidated (NC).
In the other hand the final effective vertical stresses: $\sigma'_f = \sigma'_{vo} + \Delta\sigma'_v$, equal to the sum of the vertical effective and induced stresses

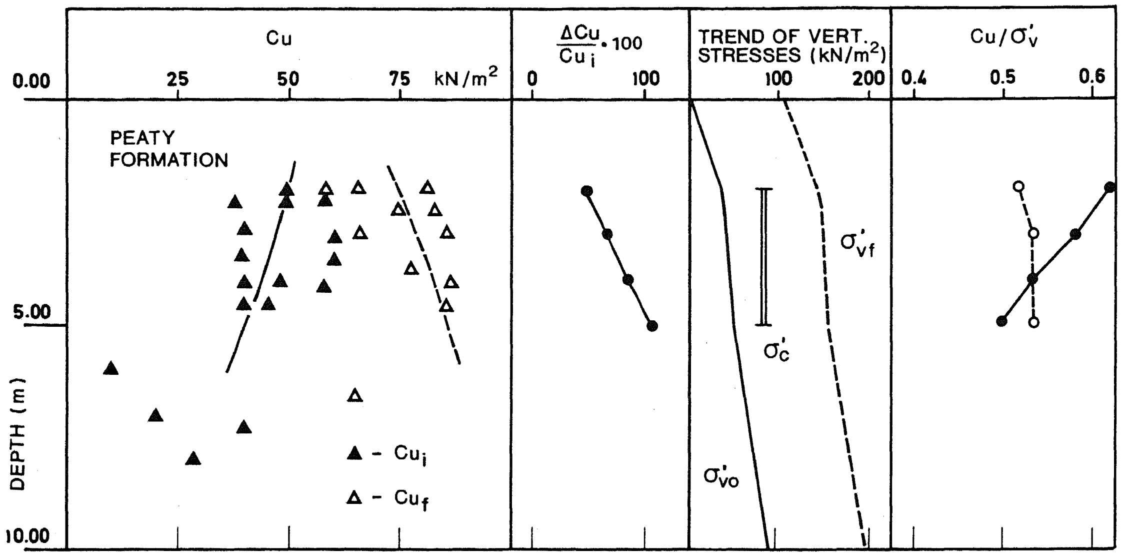


FIG. 8 - Variation of field vane strength in peaty formation as a consequence of increase in vertical effective stress.

are about equal to the preconsolidation pressure in formation 1. This explains the reason why the smallest increase in shear strength was measured at low depth.
With regard to NC soils, greater increases in percentage of strength were of course found where the ratio $\Delta\sigma'_v/\sigma'_{vo}$ was greater.
In this case a direct proportion of the type

$$\Delta c_u = (\Delta\sigma'_v/\sigma'_{vo}) \times c_{ui} \qquad (5)$$

can be used for NC soils to estimate the increase in shear strength and confirms the general validity of Skempton's equation (no.2 of this article).

FIELD VANE STRENGTH

The structure of the soil being formed predominantly of a peaty component did not allow execution on an undrained shear strength test in the laboratory. The comparison was therefore carried out through the use of field vane tests. In fig. 8 we can see the values for c_u(FV) as a function of depth. As it is possible to see, the undrained shear strength values before the application of the preload diminuisches with depth going from values of 50 kN/m^2 to 40 kN/m^2; after consolidation the situation changes with higher values being found equal to 85 kN/m^2 deep down, and lower values 75 kN/m^2 on the surface.
The average values of the consolidation pressure of the whole layer σ'_c was calculated with reference to the shape of the envelopes of the oedometric compressibility coefficients in fig. 4, interpretated according to the indications given by Ricceri ed al.(1985).That is, for soils with a PI grater than 50 the σ'_c can be determined from the change of slope in the diagram log m_v vs. log σ'_v.
This value resulted to be about 80 kN/m^2 and is reported together with the vertical effective stress values seen in the third column of fig. 8. The peaty formation resulted to be slightly overconsolidated with an average OCR value of about 1.8. The load application induces an increase of vertical effective stress to the point where it exceeds the pre-consolidation pressure value. The final condition reached is a normal-consolidated state that brings a percentage increase of the undrained shear strength that is variable in depth from 50 to 100%.
If the relation between the undrained shear strength value and the vertical effective stress c_{uf} (FV)/ σ'_{vf} is determined on the whole peaty formation, we obtain a pattern more or less constant with depth (fig.8,fourth column), with the exception of the first stretch, whose different slope can be presumably explained taking into account the clayey composition of the superficial layer that probably influenced the results of the field vane tests in some zones. The constant value of this relation is:

$$c_u(FV)/\sigma'_{vf} = 0.53 \qquad (6)$$

and is fairly high. Nonetheless we need to remember that the ratio is related to the plasticity of the material. Taking into account the indications given by Bjerrum and Simons (1960) relative to the relationship between this same ratio and the plasticity index (PI) and extrapolating their considerations to higher plasticity, we obtain values that vary between 0.40 and 0.55 for plasticity indexes that range from 100 to 200, these later values being limiting values cha-

racteristic of the formation under examination. Thus, the calculated values correspond well with extrapolated values according to what was previously indicated.
The pattern of the ratio c_{ui}/σ'_c was introduced for comparison sake with the c_{uf}/σ'_{vf} ratio in fig. 8. The variation of the first with depth (i.e. the larger ratioes are to be found closer to the surface) can be justified taking into account that the more superficial part of the peaty layer may have been subjected to a drying phenomenon that may have in this manner increased, in much the same manner that was seen for superficial clayey layers, the degree of consolidation. On the contrary the ratio was represented considering a uniform pre-consolidation pressure equal throughout the whole formation under examination.

THE CPT STRENGTH

The static cone penetration test (CPT) has also been utilized to estimate the increase in strength consequent to increase in vertical effective stress and other stresses correlated to it. The increase in strength has been directly measured and correlated to the soil nature as reported below: two typical CPT tests carried out in the same position before and after the preloading are shown in fig.9.

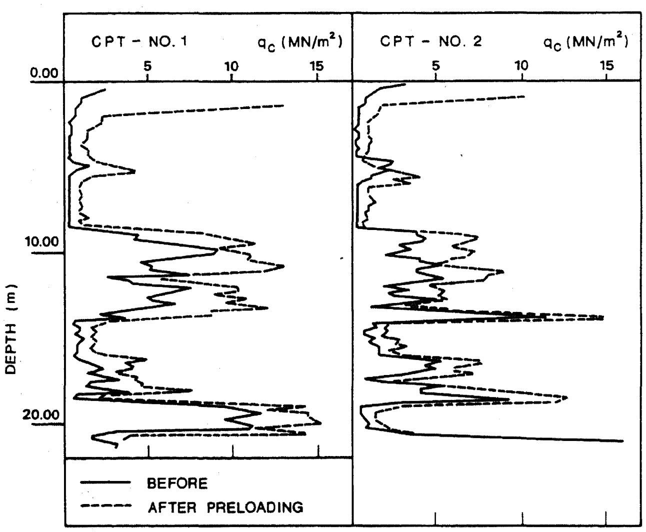


FIG. 9 - Results of two typical CPT tests before and after the preloading.

Cohesive soils

Clay and peaty clay are both considered cohesive. In this case the undrained shear strength is generally estimated by the expression

$$c_u = (q_c - \sigma'_{vo})/N \qquad (7)$$

where q_c is the point resistance of the CPT and N is a bearing factor which due to the same shape of the penetrating tip should substantially remain constant within each soil formation. Nevertheless, it has been seen from much experimental research (ESOPT I, II) that N may vary over a wide range depending on the soil plasticity (Baligh et al. 1980).
Equation (7) can be expressed as a function of N, such as:

$$N = (q_c - \sigma'_{vo})/c_u \qquad (8)$$

The value of N has been determined referring the undrained shear strength parameter determined in unconfined compression tests (for clayey soil) and field vanes (for peaty soils). The results of these determinations are reported in table 2 and they refer to average c_u and q_c values.

TABLE 2

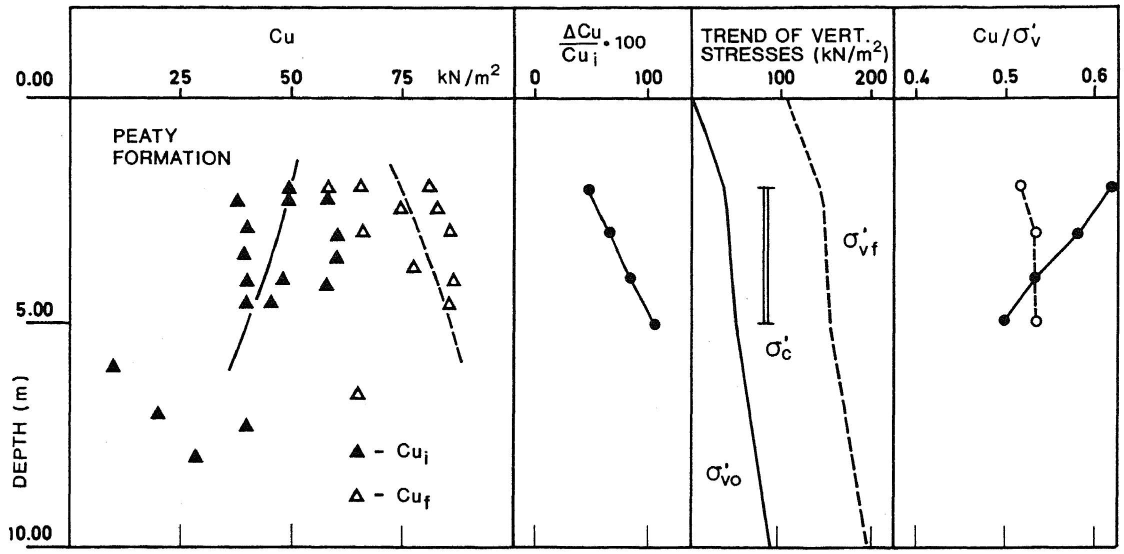
Depth (m)	CPT No.	PI	q_c (kN/m²)	c_u (kN/m²)	N
BEFORE PRELOADING					
Clayey formations					
6.5	(1)	20	300	13	18
6.5	(2)		300	13	18
16.0	(1)	30	800	26	25
15.0	(2)		900	25	30
Peaty formation					
3.5	(1)	>100	300	45	6
3.5	(2)		300	45	6
AFTER PRELOADING					
Clayey formation					
6.5	(1)	20	1000	30	28
6.5	(2)		900	30	24
16.0	(1)	30	1700	44	32
15.0	(2)		1800	42	36
Peaty formation					
3.5	(1)	>100	1400	80	16
3.5	(2)		1000	80	11

These results indicate that notwithstanding a remarkable increase in both undrained shear strength and unit point resistance, the factor N does not remain constant with the strength increase in any specific formation.
Because of the limited number of tests and of the low values of the point resistance of the CPT no general comment can be made, except that N values are higer for cohesive soils (N = 18-36) than for organic soils (N = 6-16).

Granular soils

Sandy soils exhibited a substantial increase in the unit point resistance of the CPT (q_c) as a consequence of the stress increase. So the normalized cone resistance, that is the q_c , divided by effective vertical stress σ'_{vo} should remain about constant. This lineary in the behaviour is not fully represented in nature as reported by Schmertmann(1976), Baldi et al. (1982). This effect has been seen analyzing the ratio

$$(q_c/\sigma'_v)_f/(q_c/\sigma'_v)_i = \alpha \qquad (9)$$



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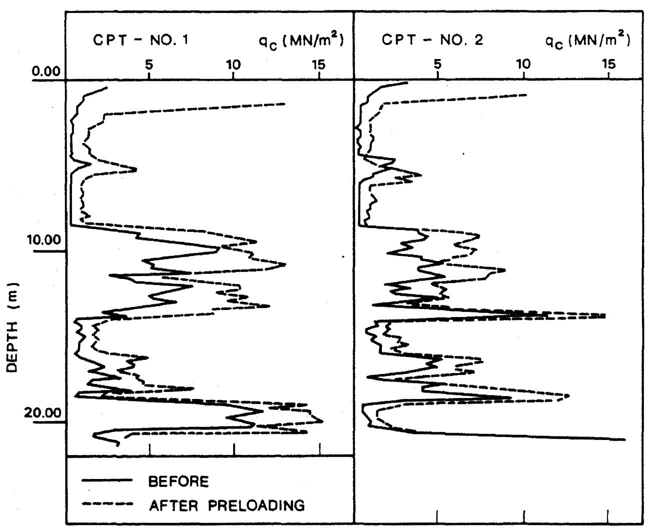

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$$(q_c/\sigma'_v)_f/(q_c/\sigma'_v)_i = \alpha \qquad (9)$$

In our case this ratio varies in CPT 1 and 2 from 0.83-0.85 in formation no. 4 and from 0.87-0.90 in formation no.6. α values lower than 1.0 indicate a decrease of the normalized cone resistance with the increase of stresses. (This has been defined as scale effect).

CONCLUSIONS

The findings presented in this paper are based on two well defined and controlled conditions:

- a consolidation process developed in uniaxial conditions due to the remarkable extension of the embankment compared to to the analyzed depth;
- a stress variation in a particular site that induced a complete primary consolidation of the peaty and clayey strata.

Based upon the above conditions the shear strength increase has been considered analyzing the results of various types of tests: unconfined compression, field vane and cone penetration tests.

It has been shown that each test must be considered separately and that it does not seem correct to generalize the use of empirical relations proposed in scientific literature. Each one needs to be verified on each site and on each formation before being used in the design.

In the laboratory only the clayey formations have been analyzed.

It has been shown that from the results of unconfined compression tests the OCR can be determined as well as the increase in the undrained shear strength.

The peaty formation has been studied with field vane (FV) tests.

The ratio c_u/σ'_v resulted to be in the order of 0.5 both before and after consolidation ($\sigma'_v = \sigma'_c$ has been utilized before applying the preload); this apparently high value can be easily justified considering the very high plasticity of the peaty formation as pointed out by Bjerrum and Simons (1960). The CPT has shown that N values for cohesive and organic soil may vary considerably both due to the soil nature and to the stress level.

In granular soil a small scale effect has been clearly seen.

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