Consolidation of a Thick Layer of Soft Clay

Consolidation d'une Epaisse Couche d'Argile Molle

A.BURGHIGNOLI Lecturer,
G.CALABRESI Professor of Soil Mechanics, Engineering Faculty, University of Rome, Italy

SYNOPSIS A detailed in-situ investigation on the behaviour of a soft normally consolidated clay has been carried on for five years following the completion of a large embankment for an airport runway. The vertical strains and the pore pressures have been measured in many points of several control sections along the runway and have been related to the results of laboratory tests on samples of various sizes. The effect of non-homogeneities are apparent in the behaviour of the large samples and of the clay layer in situ. Where the consolidation process is very slow a continuous increase of the pore pressure has been observed, under a constant applied load. Pore pressure diffusion and creep effects are analysed and related to the observed phenomena and the great influence of the latter clearly appears.

INTRODUCTION

Laboratory and in-situ investigations carried on during the enlargement of Fiumicino airport in order to study the behaviour of the foundation soils and to check the validity of design predictions have provided data of relevant interest on time behaviour of a layer of soft, normally consolidated clay, over 30 meters thick.

Some of the observations made during the actualization of the project and in particular those related to pore pressures induced by the construction of embankments and to the effects of driving of sand drains have been discussed previously (Calabresi, 1970; Croce, Calabresi, Viggiani, 1973). This paper deals with the behaviour of the clay layer during a period of 5 years after the end of construction. Field measurements of pore pressures and vertical displacements, made in a great number of stations were mainly directed to the investigation of the effect of local soil characteristics on the progression of consolidation phenomena.

SOIL CHARACTERISTICS

Fiumicino airport is spread over an ancient coastal lagoon partly filled by geologically recent deposits and partly reclaimed at the beginning of this century (Calabresi and Manfredini, 1975). The nature of the soil and its characteristics have been defined with great detail in various points of the area, by means of field and laboratory tests carried on during the design and construction of a new runway, its taxiway and other structures in the enlargement area.

Some special equipments were used to study the heterogeneities, the general physical and the mechanical properties of the clayey soil, in relation to their influence on the consolidation process. In particular, Kjellman continuous sampler and a very large sampler, having a diameter of 0.5 m and a length of 1.7 m were employed to take samples representative of macrostructural characteristics.
Figure 1 shows a typical soil stratigraphy in the areas where the observations presented in this paper were made.

Beneath an upper thin crust of stiff brown clay, overconsolidated by desiccation, is a layer of blackish organic clay with inclusions of fibrous peat of low strength the thickness of which varies between 2.5 and 4 m. This covers a layer of grey, normally consolidated, soft clay, from 28 to over 40 meters thick; underneath there are stratifications of gravelly sand, of considerable thickness, followed by deposits of clayey silts of medium strength.

The soft clay between the peat and the deep gravelly sand has irregularly distributed inclusions of sand and of fragments of shells. These inclusions are normally very thin (1 + 5 mm), but may reach at times a thickness of a few centimeters. Some levels are characterized by fine sand evenly diffused in the clayey matrix, others by thin layers of organic blackish clay and peat; the latter occur more frequently in the upper part of the deposit.

The clay layer was formed by an alternance of depositions of continental and marine waters, typical of a lagoon which was at times directly connected to the sea and at times separated from it by coastal dunes. Compressibility curves and undrained strength clearly show a state of normal consolidation.
No effects of "quasi-preconsolidation" have been observed, as it can be expected considering the re -

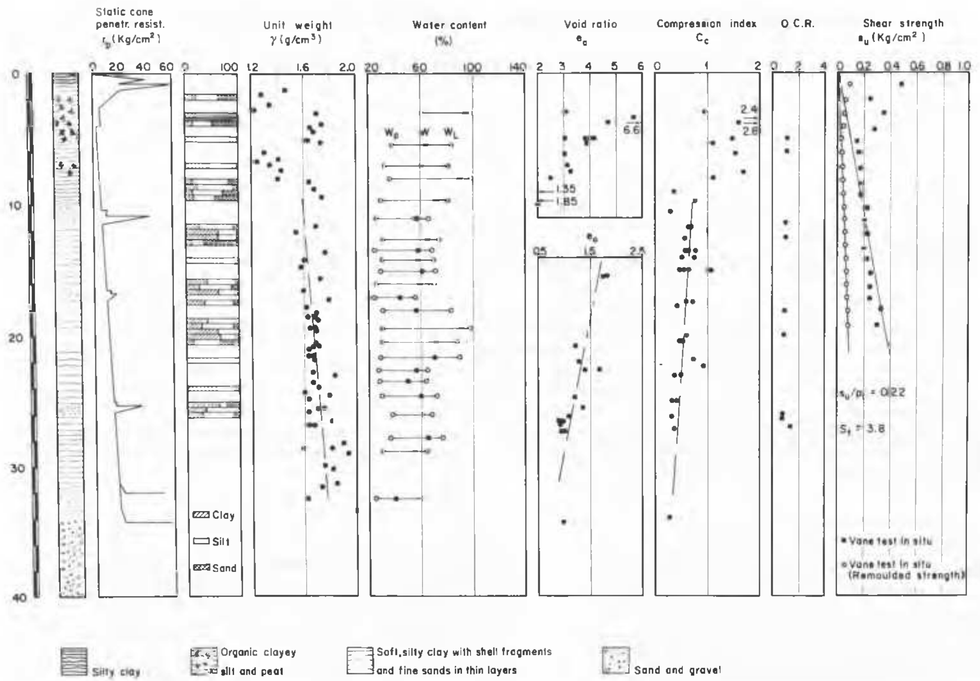

FIG. 1 Geotechnical properties of Fiumicino subsoil in the test fill area.

latively recent age of the deposits and the probable absence of cementing agents. General physical chara - cteristics as well as penetrometer and vane borer dia grams seem to evince a fair uniformity of the clay layer. Apart from the obvious effects of the rare sandy inclusions, the undrained strength increases regularly with depth at a constant rate: $c_u/p_o' = 0.22$. The sensitivity is on the average 3.8. However a more detailed and analytical investigation shows that this uniformity is merely apparent and applies only to the overall behaviour.

Two undisturbed samples of large dimensions (50 cm in diameter and 1.7 meters long), taken in two different positions immediately below the layer of peat, have clearly pointed out the great variability of physical and mechanical characteristics of the clay in a space of few centimeters, due to relevant changes in the nature of deposited materials and in the environment conditions during sedimentation. Figure 2 illustrates some of the results concerning one of the two samples.

This variability of characteristics was also confirmed by the observations made at the scanning electron microscope of Cambridge University on specimens taken

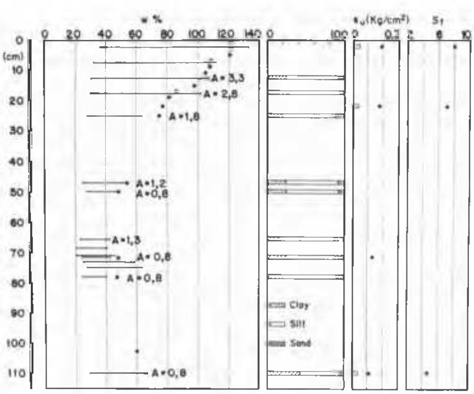

Fig. 2 Variations of plasticity, sensitivity and water content in a large sample.

a few centimeters apart. Some microphotos reported in figure 3 clearly show the great heterogeneities in the components.

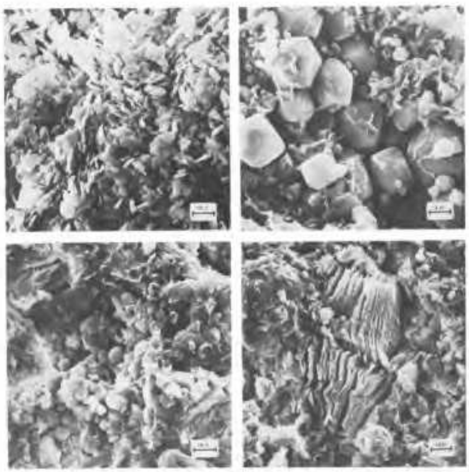

FIG. 3 Electron microphotos of four specimens taken from a large sample.

Taking account of such situation, the geotechnical investigations were mainly devoted to the definition of a mean behaviour of the soil, as well as to the research of typical properties of the various layers. To reach the first aim, 25 cm thick sections of the large sampler were used as consolidation cells to carry out compression tests (fig. 4). Local compressibility was instead determined on 2 cm thick specimens, taken from undisturbed samples of normal or

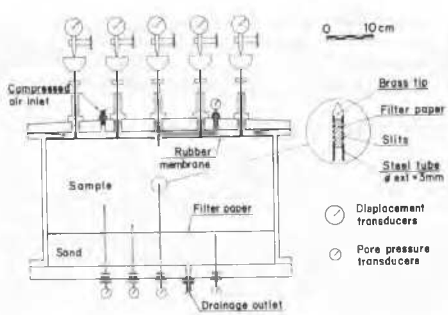

FIG. 4 Consolidation cell for sections of large samples.

large size. The values of the coefficient of consolidation obtained from time settlement diagrams in the two cases are very different: $0.8 + 1.2\cdot10^{-3}$ and $0.5 + 5\cdot10^{-4}$ cm^2/s respectively. Related variations of the compression index C_c were very small, as shown in fig. 5, where results of tests on sections of the large sampler and on small samples taken from such sections are compared.

EARTH WORKS AND SAND DRAINS

Runway and taxiway embankments are over 100 m wide, at the base. Static and hydraulic conditions have imposed a final height of 2.2 + 3 meters above ground. To speed up the consolidation in the upper part of the foundation soil and to limit differential settle-

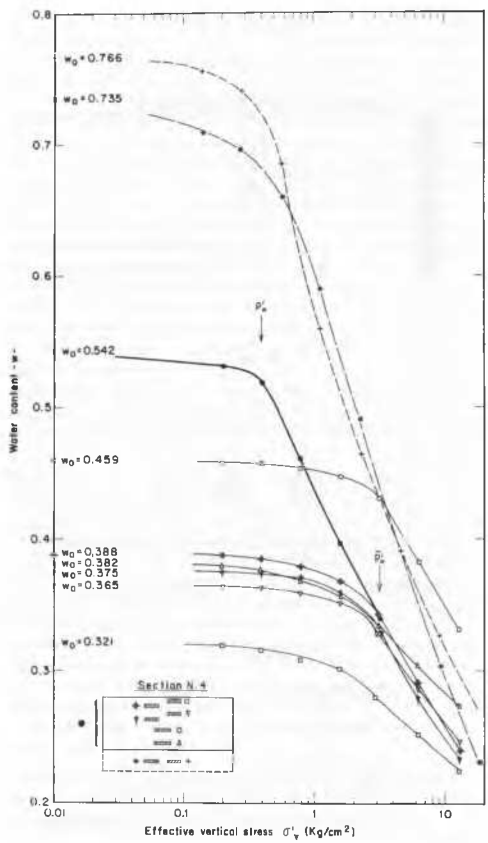

FIG. 5 Compressibility curves obtained from specimens of various sizes.

ments, sand drains were driven into the first 10 meters and a precompression load equivalent to 30% of the final one was applied for a period of 15 months. The sand drains form a network of triangular meshes of 3 m.

Seven test sections along the runway and the taxiways were provided with control instruments. Along portion of an embankment (400 m) was built before general works began and provided with a comprehensive set of field instrumentation to follow in detail the progress of soil consolidation.

This paper examines the behaviour of the clay layer with reference to the measurements taken in this test fill.

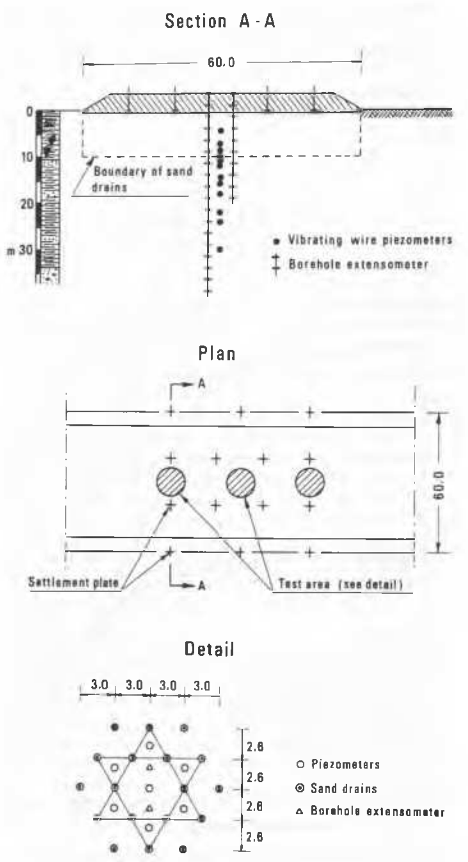

FIG. 6 Equipment installation at the test fill.

EXPERIMENTAL OBSERVATIONS

Vibrating wire piezometers and borehole extensimeters (of mechanical and electro-magnetic type) were installed in the experimental portion of the embankment, following the scheme of fig. 6. The maximum height of fill was 3.6 m during preloading and 2.6 m once the overload was removed.

Figure 7 shows the diagrams of vertical displacements measured in the peat and clay layers pierced by the sand drains and below them. Up to date (1976) more than 70% of the total settlement is due to strains occurring in the upper zone because of different conditions under which consolidation occurs as well as of higher compressibility of the peat.

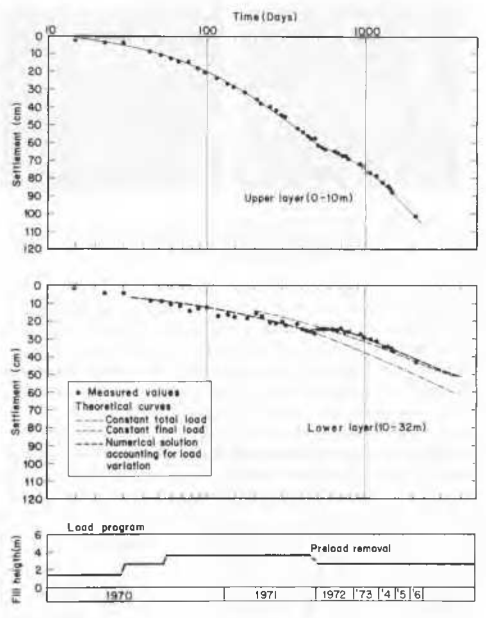

FIG. 7 Observed and back-calculated settlements.

Preload gives up to a remarkable change in the settlement curve. The effects of sand drains on the rate of consolidation and the greater compressibility of the peat (between 3 and 6 m) and the clay with a high organic component (between 6 and 8 m) more clearly appear by plotting vertical strains versus depth at various times (fig. 8).

In the examined area some thin sandy layers are present between 10 and 12 meters of depth and may be considered as a permeable surface connected with the sand drains and acting as a draining boundary at a

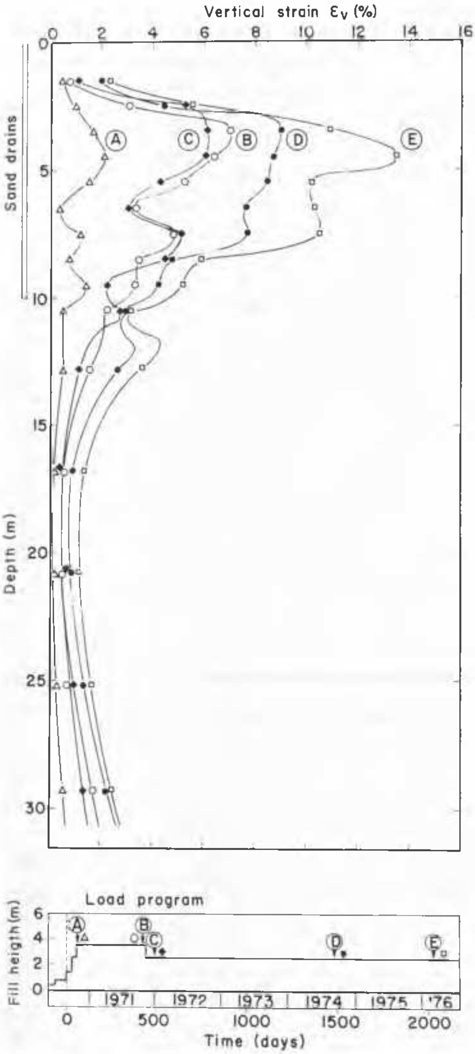

FIG. 8 Vertical strains and their variation with time

depth of 11 meters for the soil below. Another draining surface may be located at a depth of nearly 32 m, where the gravelly sand begins. Therefore the layer between 11 and 32 m may be considered as undergoing a onedimensional consolidation process under double drainage conditions. This assumption is proved by the form of strain isochrones as well as by the curve of total settlement versus time. In fig. 7 the experimental data are compared with the theoretical consolidation curves for final and precompression loads, according to Terzaghi's theory, and with the curve obtained from a numerical solution taking account of the variations of applied load. The coefficient of consolidation assumed in either case was $c_v = 3 \cdot 10^{-3}$ cm^2/s, which leads to a mean consolidation degree $\bar{U} = 0.38$ before preload removal. Due to this low value of $\bar{U}$ the consolidation curve obtained by supposing the final load as applied from the beginning is rather near to the measured settlements. However the numerical solution fits very well settlements curve, particularly in the swelling period following preload removal.

The pore pressures measured at different times after completion of the embankment are compared in fig. 9 with the mean total stress increase. Even immediately after completion of the embankment the former are lower than the increment of the mean total stress, especially in the areas closer to the draining boundaries. However onedimensional consolidation progress during construction should not be the only reason of this difference. Partly saturated levels due to gas produced by the organic matter and pore pressure equalization through lateral diffusion towards unloaded areas must have influenced the pore pressure field at the beginning of the loading period. It clearly appears from fig. 9 that the rate of consolidation is much higher in the upper levels, where sand drains were driven, than in the deep ones. The pore pressures tend to an equilibrium value which is higher than the initial one due to the change of boundary conditions produced by the modification of surface drainage system. On the average the observed pore pressures seem to agree with the consolidation model used to analyse the settlements of the layers below the sand drains, especially during the precompression stage. At the removal of the preload the mean consolidation degree, calculated as the ratio between the areas of the diagrams of the pore pressures and of the mean stresses, between 11 and 32 m, is $\bar{U} = 0.32$, which leads to a coefficient of consolidation $c_v = 2.1 \cdot 10^{-3}$ cm^2/s. Therefore, in the first stage of consolidation the diagrams of the pore pressures along the vertical and their evolution with time are consistent with settlement curves. Soil parameters back-calculated from separate analyses of both, although not coinciding, are nevertheless sufficiently near. The coefficient of consolidation resulting from in situ measurements and from laboratory tests on samples of different sizes are compared in fig. 10. It can be seen that the very large specimens obtained from the special sampler have given values of c_v very close to those resulting from field measurements.

After the first stage of consolidation the diagrams of pore pressures versus time at constant load show very different soil behaviours even between close points. They appear also to be anomalous with respect to the consolidation process. Non-uniformity of pore pressures along the vertical, in comparison with previous values, is very clear in the diagram of fig. 9

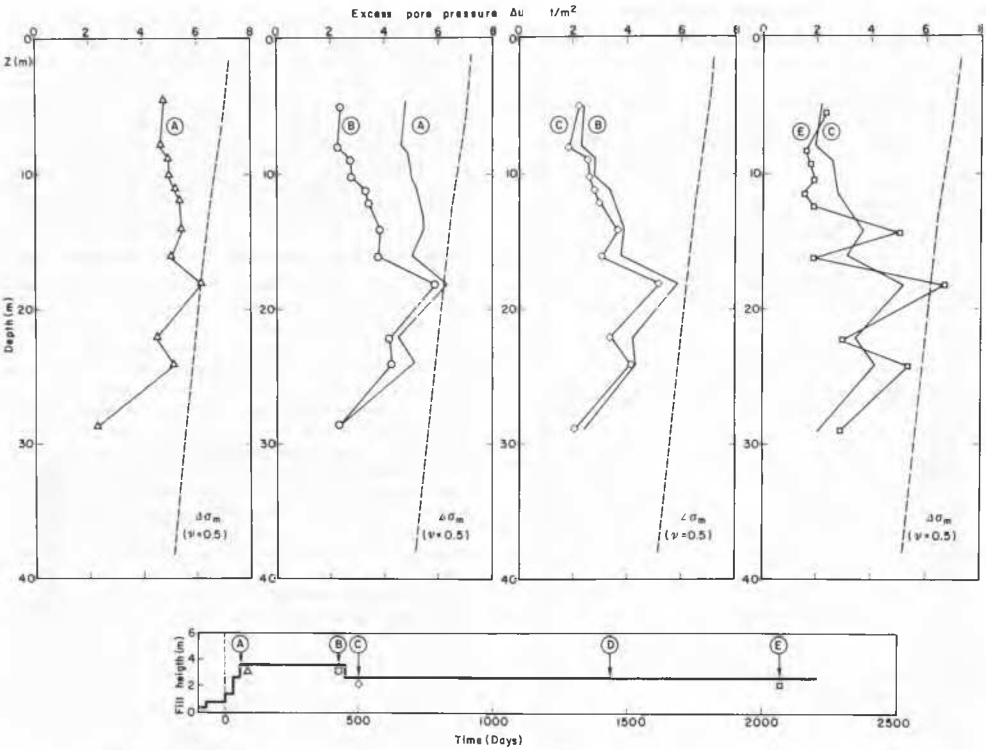

FIG. 9 Pore pressures distribution and their variation with time.

In several points, after an initial decrease, pore pressures begin to gradually increase (fig. 11). The most significant increments occur where the initial values are higher, in a really striking contrast with what one could expect from diffusion or consolidation processes.

Similar phenomena have been previously remarked in several cases. Berry and Wilkinson (1969) observed a progressive rise in pore pressure for a period of over two months after the application of a load, in undisturbed samples of organic clay kept in undrained conditions in a consolidation cell. Walker (1969) remarked in laboratory as well as in field tests on sensitive clays, the increase of pore pressures under a constant load in undrained conditions. Chang, Broms and Peck (1973) found that almost no decrease of pore pressures occurred in some clays layers of the foundation soil of two embankments, 30 years after their construction. All these phenomena have been ascribed to structural viscosity of clay.

Creep of the solid skeleton seems to be also the cause of the phenomena observed in the clay of Fiumicino, in the almost undrained conditions produced by the great thickness of consolidating layer. However they are rather peculiar, because of the high variability of soil behaviour and the high increase of pore pressures above the initial values over a long time. The ratio between measured pore pressures and mean total stresses and the increase with time of non-homogeneities in pore pressure distribution allow to leave out diffusion or equalization processes as possible causes of the observed phenomena. Furthemore, by comparing local deformations and variations of pore pressure, it is easily seen that pore pressure increases, under constant load, do not correspond to soil swelling, but no periods of no strain or of small compression (fig. 12). This behaviour is quite different from the one observed for short periods at the edge of the embankments, where increases in pore pressures and volume expansion took place in the initial loading phase, especially near the sandy levels, as the effects of a diffusion process.

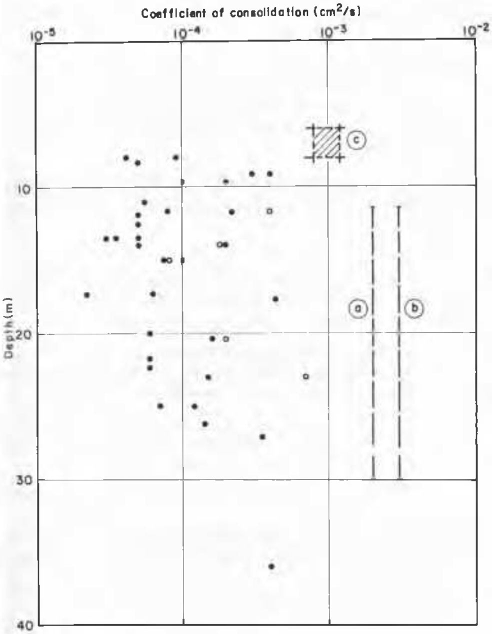

FIG. 10 Coefficients of consolidation obtained from laboratory and field tests: a) in-situ pore pressure measurements; b) in-situ strains measurements; c) large sample; standard laboratory tests in vertical (•) and horizontal (o) direction.

CONCLUSIONS

Observations carried on over a long period of time on Fiumicino clays have emphasized the difficulty in forecasting the behaviour of that soil even in a geometrically simple problem, because of its heterogeneities, wich however are not easily detectable by routine soil investigations. Strain measurements in the layers below the sand drains agree on the whole with a simple onedimensional consolidation model, if an average of different behaviours of the single soil levels, emphasized by the measurements of pore pressures, is taken into consideration.

There is a large scattering of c_v values obtained in laboratory tests and also a difference between their average and the values derived from field behaviour of the clay layer. This is clearly explained by the spacing of its heterogeneities in comparison with

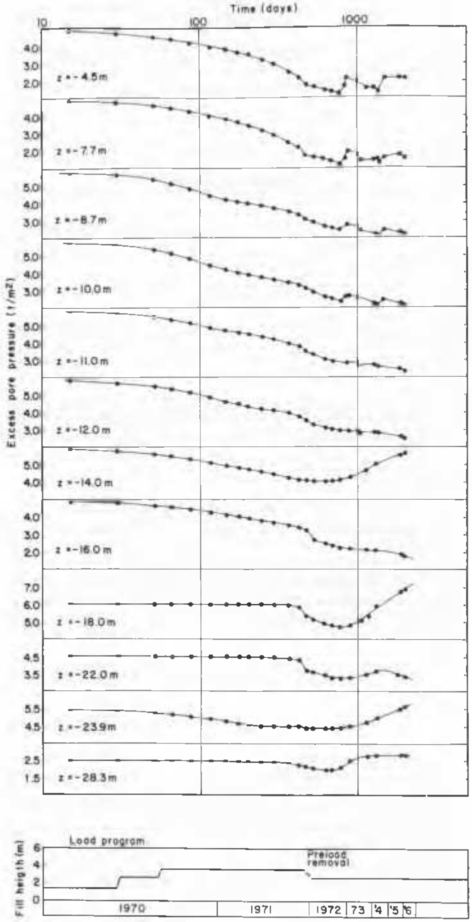

FIG. 11 Time pore pressure curves over a long period after fill completion.

the size of consolidation test samples. Actually tests made on large undisturbed samples (0.5 m in diameter and 0.25 m thick) have yielded values of consolidation parameters that are much closer to the real ones.

The creep that appears in those levels where the rate of consolidation is lower has caused such increase in pore pressures so as to disguise the consolidation process, making its interpretation particularly uncertain.

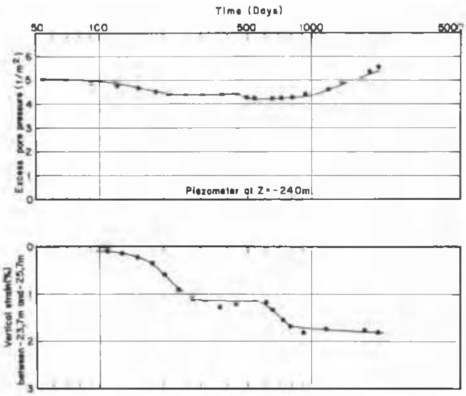

FIG. 12 Local variations with time of pore pressures and vertical strains.

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