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METHODS OF SETTLEMENT PREDICTION FOR EMBANKMENT ON SOFT CLAY: A CASE STUDY

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SUMMARY Predictions of the settlements of a test embankment on soft Bangkok clay have been made by five different deterministic methods and by one probabilistic approach. The five deterministic procedures are: (1) the stress path method, (2) the elastic method, (3) the critical state theory, (4) Garlanger's method, and (5) Gibson & Lo's method. The probabilistic approach is based on a deterministic one-dimensional Terzaghi model, but using soil parameters with known probability distributions. The analysis involves further various methods for evaluating the stress distribution under the embankment, namely those of Gray, Poulos and Holl.

It is demonstrated how the various components of a deterministic settlement analysis greatly influence the outcome of a prediction and that in many cases it is not too difficult to find a combination of procedures and soil parameters which produce a fairly good fit to the observed data. The introduction of probabilistic concepts, on the other hand, enables to a certain extent a quantification of uncertainty in a settlement prediction.

INTRODUCTION

There exists now a vast amount of published information on the performance of test embankments on soft ground. In numerous case histories settlements have been predicted and observed, and these involve various soft clay areas, such as eastern Canada, the eastern United States (notably the Boston area), the United Kingdom, France and Japan.

In most of these cases, however, only one or two methods were employed for the predictions of settlements (most frequently the one-dimensional Terzaghi theory), because it is rare that the various kinds of soil parameters needed in the different methods of analysis have all been determined in one single project. It is often difficult to arrive at a consensus on the suitability of a method because both, underprediction and overprediction, have been reported for the same method, and details for comparison are lacking. When more than one method of analysis has been used, there is also the tendency to publish results showing that the best agreement has been achieved by the method which has the highest degree of sophistication.

In this paper, prediction of the settlements of a test embankment on soft Bangkok clay has been made by five different deterministic methods and by one probabilistic approach. The purpose is to show the difficulties inherent in making predictions, namely the great variation in results that can arise from different combinations of the components involved in a settlement analysis. In most cases where the results of a prediction are known in advance (so-called type C1 predictions according to the terminology of LAMBE, 1973), it is possible to find a combination which fits the measured data reasonably well.

METHODS USED FOR SETTLEMENT PREDICTION

Components of a Settlement Analysis

A realistic settlement analysis must involve a procedure which describes the development of strains and pore pressures at a point in the soil during and after the application of an external load. In the conventional approach, this procedure is usually separated into two steps, viz. (1) evaluation of the stress increments in the ground and (2) determination of the magnitudes and rates of settlement using an appropriate stress-strain relationship of the soil together with suitable boundary conditions.

The settlement process is often described by three phases, namely initial settlement, consolidation settlement, and secondary compression or creep settlement. Although the last two phases are not independent of each other, they are often treated separately.

Deterministic One-Dimensional Stress-Strain Methods

Three methods to compute the magnitude of settlement have been selected here. These are: (1) the stress path method (LAMBE, 1964), (2) the elastic method (DAVIS & POULOS, 1963, 1968), and (3) the method using a critical state model of the soil (ROSCOE et al., 1963; ROSCOE & BURLAND, 1968; SCHOFIELD & WROTH, 1968). Only a brief description of each of these methods is given here and for a detailed treatment the reader should consult the respective references.

Stress path method. - In this procedure the effective stress paths of selected elements are estimated and simulated as closely as possible in laboratory tests with measurement of vertical strains. Two approaches are possible: (1) method using strain contours and (2) method using a laboratory test which duplicates the estimated field stress path.

In the first technique a diagram of strain contours obtained from the results of consolidated undrained triaxial tests is established onto which the field stress path is then superposed. The vertical strain due to undrained loading can then be found directly from the axial strain contours, while the volumetric strain occurring during consolidation is derived from an oedometer test. The total vertical strain is then the sum of the undrained and the consolidation components. In the second procedure which, if possible, should be used for final analysis, a laboratory triaxial test duplicating the field effective stress path (i.e. both its undrained and drained component) is performed and the vertical strain is measured.

Elastic method. - This method is similar to the stress path method, but instead of directly using observed strains in the analysis, appropriate elastic constants are derived from the laboratory tests and elastic theory is employed to calculate both initial and consolidation settlement.

Two approaches are commonly used: (1) for stratified soil deposits the initial and the total final settlement are calculated from a summation of strains, whereby these strains are obtained from elastic theory (Hooke's law) with undrained and drained Young's moduli and Poisson's ratios and stress increments computed by elastic stress distribution theory, (2) for more or less homogeneous soil conditions elastic displacement theory can be employed by means of an influence factor for settlement.

The success of the elastic method depends mainly on the values selected for the elastic constants. Soil does not behave as an ideal elastic material; the elastic constants are a function of the initial stress state and the level of shear stress.

Critical state theory. - The critical state theory was developed for normally and lightly overconsolidated clays as a general stress-strain theory. It treats the soil as an isotropic, elasto-plastic, strain-hardening material. The soil is further assumed to possess a yield locus which forms a boundary of stress states such that the strains for stress paths lying wholly within this boundary are recoverable and small. Stress paths crossing the yield locus will cause large, irrecoverable strains. For an increment in stress causing yield, the volumetric strain increment has a recoverable and an irrecoverable component, while for the shear strain increment the recoverable component is neglected. The initial settlement can then be calculated from the plastic shear strain increment. The consolidation settlement is obtained from the volumetric strain increment and from that part of the plastic shear strain increment which is caused by a state path on the state boundary surface when the yield locus is shifted (see ROSCOE & BURLAND, 1968 for details).

The critical state model only requires three soil parameters, namely the compression index, C_c, the swelling index, C_s, (obtained from oedometer tests) and the effective angle of friction, ϕ'.

Deterministic One-Dimensional Stress-Strain-Time Methods

The three methods described previously do not consider the effect of secondary compression or creep. In the following, two methods are described which are capable of including also long-term or time effects in the material model.

Garlanger's method. - This procedure, developed by GARLANGER (1972), is based on a void ratio - effective stress - time diagram proposed by BJERRUM (1972). Mathematical formulation of this diagram and making use of the continuity equation and the effective stress law, gives a system of equations which when solved simultaneously for the given boundary conditions yields void ratio vs. time and excess pore pressure vs. time relationships at every point in a consolidating layer. Integration of the void ratio - time - depth relationship produces average degrees of consolidation and the settlement of the soil layer at any time. The system of equations is best solved by a finite difference numerical procedure.

Method of Gibson & Lo. - This method uses a stress-strain-time relationship which is based on a rheological model, i.e. a mechanical analogue consisting of springs and dashpots. The model can be considered a direct extension of Terzaghi's primary consolidation theory, but while Terzaghi's model can rheologically be represented by a single Hookean (linear) spring, Gibson & Lo connected the spring in series to a Kelvin body. The model parameters can be obtained from long-term consolidation tests. Details of the method can be found in the original paper of GIBSON & LO (1961).

Probabilistic Stress-Strain Model

Uncertainties in settlement analyses arise from three sources, namely: (1) variability of the soil parameters, (2) magnitude and distribution of imposed loads, and (3) idealizations made in the mathematical model describing stress distribution and deformation. By employing techniques of probabilistic analysis, the engineer can, together with his own judgement and experience, make at least a statement on the reliability of a prediction.

In most probabilistic procedures (e.g. KRIZEK et al., 1977) quantities representing compressibility and applied loads are considered as random variables whose probability distribution can be derived from available data. These random variables are then introduced into a deterministic settlement formula and the distribution of the settlement variable is obtained. In this investigation a method which follows a similar approach has been used. It was developed by the fourth author (SIVANDRAN, 1979) and has been described in detail elsewhere (BALASUBRAMANIAM et al., 1979). The deterministic settlement model corresponds to the one-dimensional Terzaghi theory and the void ratio - effective pressure curve is approximated by two straight lines intersecting at the void ratio, e_c. The settlement ratio, defined as the total settlement divided by the layer thickness, is then a function of the compression and reloading indices, C_c and C_r, the void ratios, e_o(initial) and e_c, the effective overburden pressure, σ'_{vo}, the preconsolidation pressure, σ'_{vc}, and the vertical stress increment, $\Delta\sigma'_v$, all random variables. A probability distribution for the settlement ratio was then obtained by means of a Monte Carlo simulation.

TEST EMBANKMENT AND SOIL CONDITIONS

The test embankment investigated was located in Bangpli district, approximately 20 km east of central Bangkok. It was constructed in connection with the site investigation program for a proposed new international airport with the purpose to study long-term settlements. Its dimensions are given in Fig. 1 where it can be seen that it was of variable height. The highest section, 100 m long and 2.9 m high, was provided with a 10 m wide berm on one side. Also shown in Fig. 1 are types and locations of instrumentation used to measure deformations and excess pore pressures.

The geotechnical profile at the site is shown in Fig. 2 and may, according to strength and compressibility, be divided into the following zones: (1) a weathered zone of originally soft clay to a depth of about 4 m below the

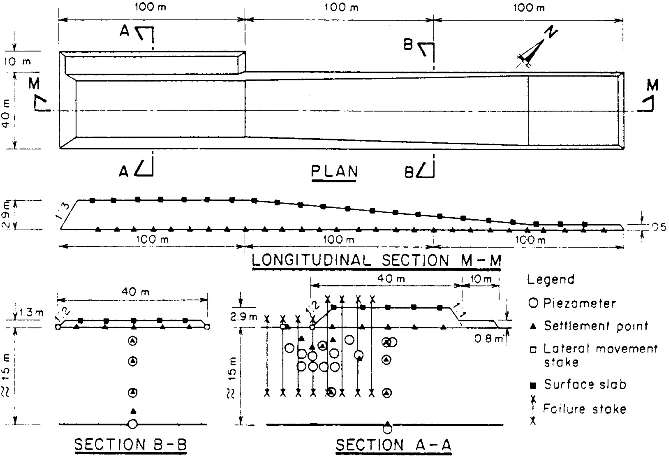


Fig. 1 - Dimensions of test embankment at Bangpli, Bangkok

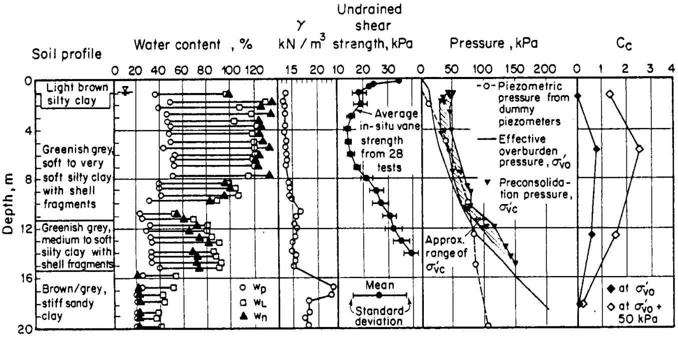


Fig. 2 - Geotechnical characteristics of soft Bangkok clay at Bangpli

ground surface, (2) a zone of highly compressible soft clay reaching to a depth of about 11 m, (3) a layer of medium clay to 15 m depth, followed by (4) a sandy stiff clay, the thickness of which varies from about 3 to 10 m. The stiff clay is underlain by sand. The soft and medium clays contain frequent pockets of shells, sand lenses and silt seams which make an assessment of the in situ permeability and the drainage boundaries difficult. Also shown in Fig. 2 is the variation of the compressibility with depth with the compression index, C_c, evaluated at the in situ overburden pressure and at a stress increment of 50 kPa, which corresponds approximately to that induced by the embankment load at Section A.

Dummy piezometers installed in the area indicated the existence of a pore pressure deficiency below about 7 m depth caused by deep-well ground water pumping.

PREDICTION OF EMBANKMENT SETTLEMENT

The settlement - time behaviour of the test embankment was evaluated at four sections. Here the results are reported for two locations, namely below the centre-line of the embankment at the ground surface at Sections A and B (Fig. 1). In order to apply the various methods of settlement prediction, it is first necessary to evaluate the stress increments caused by the embankment loads and to define the drainage conditions in the soil profile in order to model pore pressure dissipation.

Stress Distribution

For obtaining stress distributions induced by surface loads elastic theory is commonly used. Among the many solutions now available those best simulating the actual soil profile and the boundary conditions should be used. Models of stress distribution employed here are those of GRAY (1936), POULOS (1967), HOLL (1940) and Westergaard (FADUM, 1948).

For applying Gray's formula, the subsoil was considered to be a homogeneous, isotropic, elastic half-space, while for Poulos's model the boundary between soft and stiff clay at 15 m depth was assumed as a rough rigid base. Holl's formula assumes a modulus increasing linearly with depth and a Poisson's ratio, ν, of 0.3 (which applies more to sands than to clays). Westergaard's chart, finally, is based on zero lateral deformation in the soil layer. The initial stresses in the ground were evaluated assuming geostatic conditions.

Figure 3 shows the variations of the vertical and horizontal stress increments calculated by various methods at the centre-line of Section A of the embankment. In addition to the four methods mentioned above, stress increments computed by Perloff's method (PERLOFF et al., 1967) and by a finite element procedure are also included in Fig. 3. Perloff's method assumes the embankment to be continuous with the underlying soil while in the finite element analysis a soil model with a rough rigid base and with the provision for yielding was considered. The modulus was taken as 70 times the vane strength, s_u, and yielding was to commence at 0.65 s_u.

It can be seen from Fig. 3 that while the vertical stresses are little influenced by different values of Poisson's ratio and boundary conditions, the horizontal stresses are significantly affected and decrease with diminishing Poisson's ratio. Such a decrease will enhance the predicted settlements. The stress distribution is further influenced by the stratification in the soil profile, particularly when there is a drying crust at the top.

Soil Profile Models for Deterministic Prediction

For the stress path, the elastic and the critical state methods the soil profile was taken as 22 m thick with top and bottom drainage and was divided into five layers. For the methods of Garlanger and of Gibson & Lo, a single layer profile of 15 m thickness was considered which could drain at the top but was impervious at the base. In order to account for the numerous pockets and lenses of shells, sand and silt, one internal drainage layer was assumed at 11 m depth in both profiles. The selection of this depth was based on the examination of boring logs.

Rate of Settlement

The pore pressure dissipation, and thus the rate of settlement, for the stress path, elastic and critical state methods was computed by finite difference technique from a one-dimensional multi-layer model. The details of this

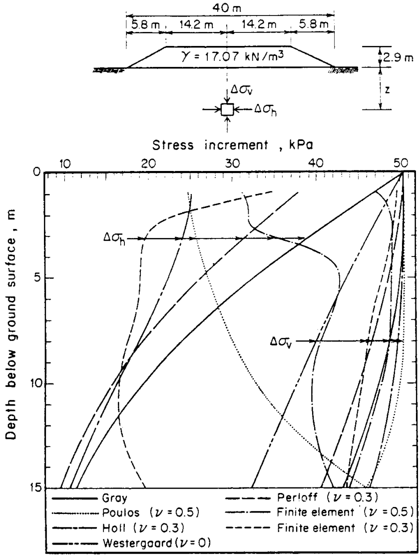


Fig. 3 - Stress increments under test embankment obtained by various methods

model and mathematical techniques are fully described in MURRAY (1971, 1972). The model can take into account different soil properties in the various layers and a variation of these during the consolidation process.

Initial excess pore pressures were obtained from a Poulos-type stress distribution and using Skempton's method which relates pore pressure increments to stress increments (SKEMPTON, 1954).

Settlement Prediction, Deterministic Analysis

Comprehensive information on soil data, assumptions made and analysis procedures were presented by the third author (VARAURAIRUT, 1977). Due to space limitation only some of the more important details can be mentioned here.

For the stress path method both the technique using strain contours and that employing a duplication of the field stress path were applied. The initial settlement was assumed to occur instantly, i.e. at the end of construction. Computed values of initial and consolidation settlements are given in Table I. For the method in which the estimated field stress path is duplicated, only one type of stress distribution, i.e. that of Gray, was considered. This test procedure involved an undrained and a drained stage of loading and the soil profile for this case was divided into eight layers. Settlement versus time plots for Sections A and B obtained from the stress path method using strain contours are shown in Fig. 4.

In the elastic method the settlements were obtained from the summation of strains. The undrained and drained Young's moduli were determined from strain-controlled consolidated undrained and consolidated drained tests, respectively, but in some instances also from two-stage loading tests. The influence of local yield was not considered. Plots of settlement vs. time are given in Fig. 5.

For the method employing the critical state model the settlements were evaluated from the summation of vertical strains. Typical values of the soil parameters $M = 6 \sin \phi'/(3 - \sin \phi')$, $\lambda = 0.434\ C_c$ and $\kappa = 0.434\ C_s$ required in the analysis were of the order 0.9 to 1.0, 0.51 to 0.58 and 0.05 to 0.10 respectively. In calculating the strains several assumptions had to be made regarding the current yield locus. The yield curve was drawn similar in shape to an undrained stress path through the preconsolidation pressure corresponding to K_0 conditions. Predicted rates of settlement at Section A are presented in Fig. 6. These curves were computed by using the modified Cam clay model of ROSCOE & BURLAND (1968).

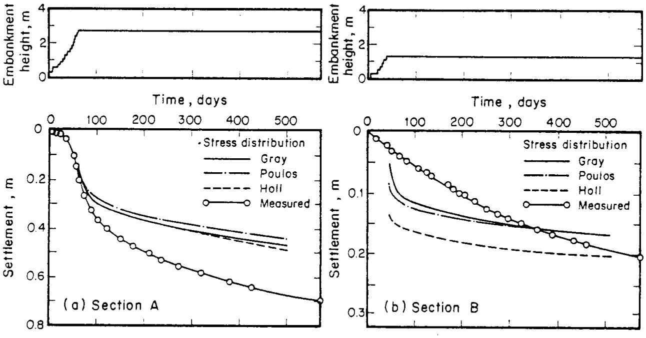


Fig. 4 - Ground surface settlements at centre-line of embankment: prediction by stress path method

TABLE I

SUMMARY OF RESULTS OBTAINED FROM VARIOUS METHODS OF SETTLEMENT ANALYSIS

Method of analysis		Section A						Section B					
		Gray		Poulos		Holl		Gray		Poulos		Holl	
		ρ_i (cm)	ρ_c (cm)	ρ_i (cm)	ρ_c (cm)	ρ_i (cm)	ρ_c (cm)	ρ_i (cm)	ρ_c (cm)	ρ_i (cm)	ρ_c (cm)	ρ_i (cm)	ρ_c (cm)
Stress path method	using strain contours	16.6	57.0	17.0	54.8	20.6	53.3	5.2	21.7	8.5	17.9	13.3	16.9
	using duplication of field stress path	12.6	55.7					1.8	13.0				
Elastic method		5.1	75.1	4.3	49.6	11.6	86.3	1.5	20.5	1.0	16.3	2.9	26.6
Critical state theory		12.1	73.3	21.3	58.9	21.8	100.7	3.3	4.7	5.1	4.7	8.4	4.2
Garlanger			284.3						172.0				
Gibson & Lo			118.2						53.3				
Probabilistic approach with 1-D consolidation model	10 %		48.2										
	50 %		58.6										
	90 %		70.7										
Measured (570 days)		69.8						20.8					

Note: ρ_i = initial settlement, ρ_c = consolidation settlement

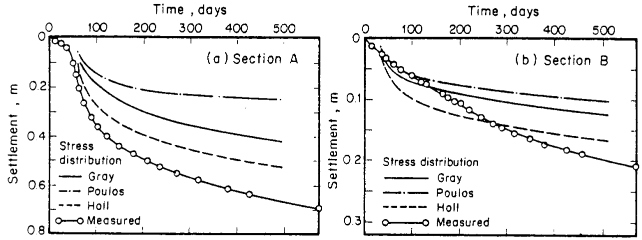


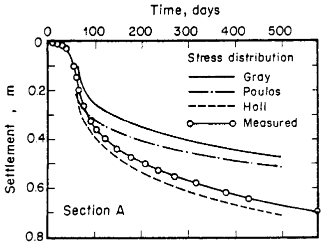
Fig. 5 - Ground surface settlements at centre-line of embankment: prediction by elastic method

In the preceding three methods of analysis settlements due to secondary compression were neglected. These might, however, be substantial with the highly plastic Bangkok clay. The following two methods include effects of secondary compression and the computed final settlement values are considerably higher.

In Garlanger's method (and also in Gibson & Lo's method) the stress distribution within the soil mass was determined by means of Gray's and Westergaard's method. The constants occurring in Garlanger's effective stress (σ_v')-void ratio (e) - time (t) model were evaluated from log e vs. σ_v' curves and from plots of log e vs. log t. The time, t_i, of the instant line in the family of time lines was assumed to be one year. Garlanger's procedure yields good agreement with the observed settlement - time record for Section B, as can be seen in Fig. 1. This method, however, does not provide for initial settlement and for plotting the settlement - time curves it was assumed that settlement at the end of construction would equal the initial settlement.

The settlement - time relationships predicted by the method of Gibson & Lo are also shown in Fig. 7. They are similar to those obtained from Garlanger's procedure. The final settlements are, however, quite different (Table I). The rheological model parameters of the Gibson & Lo theory were obtained from long-term oedometer tests.

Table I contains a summary of calculated and measured values. Settlement observations were carried out over a period of 570 days. Pore pressures were only monitored for 191 days and during that time very little dissipation had taken place. In fact, some piezometers even indicated a slight increase after an initial phase of dissipation. Thus settlement continued at more or less stationary pore pressures. The final total settlements at the various sections are, therefore, not known, but are likely to be considerably higher than those observed after 570 days.



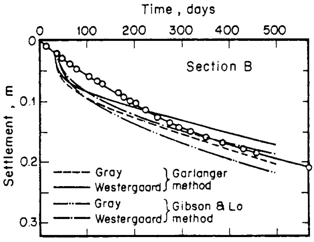


Fig. 6 - Ground surface settlement at centre-line of embankment: prediction by critical state theory

Fig. 7 - Ground surface settlement at centre-line of embankment: prediction by the methods of Garlanger and of Gibson & Lo

As can be seen from Table I, Holl's stress distribution resulted always in higher total final settlements as compared to the other two stress distributions, because of the reduced horizontal stresses ($\nu' = 0.3$) and the associated high values of deviator stress, which in turn imply a decreased value of Young's modulus. The closest prediction of the settlement - time record at Section A was achieved with the critical state model and a Holl-type stress distribution. This can be seen in Fig. 8 where the stress path, elastic and critical state methods, based on a Holl-type stress distribution, are compared. Inspite of this good agreement, Holl's stress distribution is not necessarily a correct assumption, because the embankment will deform initially under undrained conditions with $\nu = 0.5$.

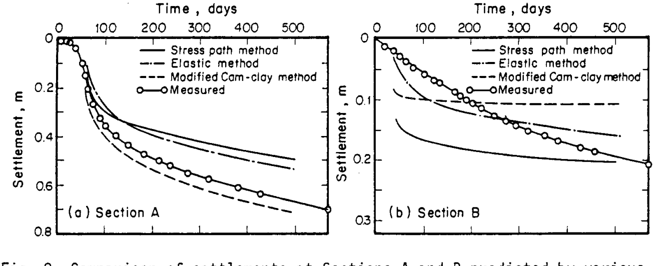


Fig. 8 - Comparison of settlements at Sections A and B predicted by various methods (stress distribution calculated by Holl's method)

Figures 4 to 8 illustrate the rather large spread in predictions which can be obtained just by varying the type of stress distribution and the method of vertical strain calculation. It probably would not be too difficult to find a combination of procedures, together with some adjustments in the soil parameters within their range of variation which would produce a fairly good fit to the observed data at any section. With more sophisticated models some of the uncertainties in the mechanics of the deformation process (stress distribution, yielding, etc.) can be removed, but increased difficulty in selecting the correct soil parameters may not yield more precise predictions.

Settlement Prediction, Probabilistic Approach

Using a deterministic settlement model with soil parameters and stress increment to which probability distributions had to be assigned, one part of the analysis consisted in establishing these distributions. For the compression index, C_c, the distribution was found to be a normal one. Contrarily, the reloading index, C_r, exhibited a definite skewness and the actual empirical distribution was used in the analysis, i.e. one for weathered clay and another one for the entire section of soft and medium clay. The values found for the preconsolidation pressure, σ'_{vc}, could again be fitted by a normal distribution. In order to obtain a distribution of the void ratios, e_0 and e_c, second order regression models were employed into which values of the compression index (C_c or C_r) generated from a Monte Carlo simulation were substituted. In order to account for the variation of the void ratio about the respective regression model, an independent degree of randomness was introduced by generating a normally distributed variable with a small coefficient of variation which had the same value as obtained from the regression model. (The coefficient of variation is defined as the ratio of the standard deviation to the mean, KAY & KRIZEK, 1971).

Finally, the initial vertical and horizontal stresses and the stress increment, $\Delta\sigma'_v$, are variables which can almost be considered as deterministic. The initial stresses were determined from the assumption of geostatic conditions and $\Delta\sigma'_v$ was computed from a finite element analysis (Fig. 3). In order to introduce into these variables a certain degree of randomness, a normally distributed variable with a small coefficient of variation was generated.

For settlement analysis the soil profile was divided into eight layers. For each layer the mean values and the distribution of C_c and σ'_{vc} were established based on test data. In layers with insufficient experimental data the coefficient of variation approach as described by BALASUBRAMANIAM et al. (1979) was used. For C_r only two empirical distributions were employed. By means of the Monte Carlo simulation technique representative values of all the parameters involved were generated from their respective distribution function and settlement ratios could be computed for each set of variables.

The probability (frequency) distribution and the cumulative probability distribution of settlements were constructed from frequency tables. Figure 9 shows the result at Section A for ground surface settlements. The distribution

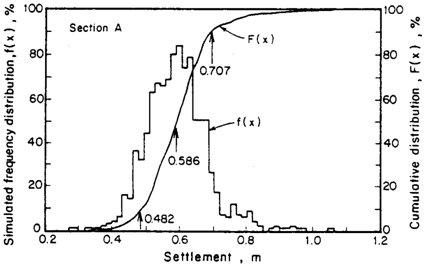


Fig. 9 - Frequency and probability distribution of predicted ground surface settlement at centre-line of Section A

of these settlements is again normal as verified by a goodness-of-fit test. In Table I the settlement values for 10 %, 50 % and 90 % probability are listed and show good agreement with those deterministic predictions which include primary consolidation only.

CONCLUSIONS

It has been demonstrated that by using different approaches to compute the stress increment and the vertical strain under an embankment on soft clay, a wide spectrum of predicted settlements can be obtained. Forecasting settlements involves always a number of uncertainties and requires considerable experience on the part of the engineer. In order to improve the accuracy of his prediction the engineer should, whenever possible, look at previous, similar cases in his region. By careful analysis of well-documented case histories, it may be possible to calibrate certain methods of prediction in a given area of soft clay.

It was further shown that probabilistic techniques are able to quantify the uncertainty of a prediction to a certain extent in that the variability of the soil parameters can be taken into account. Such an approach, however, requires the availability of a considerable amount of experimental data from the area for which the prediction has to be made.

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