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Foundations for Large Embankments

F. Colleselli
Università di Padova, Italy

G. Cortellazzo
Università di Padova, Italy

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Foundations for Large Embankments

F. Colleselli and G. Cortellazzo
Instituto di Geotecnica, Facoltà di Ingegneria, Università di Padova, Italy

SYNOPSIS: This report deals with problems connected with the design and construction of two levées built on thick soft compressible soils in the Po Delta. Settlement and pore pressure measurements in foundation soils, taken for about four years during and after construction by means of complete geotechnical instrumentation, were compared with results obtained through various methods during both design and back-analysis phases. Back-analysis was carried out using numerical simulations with finite-element programs. Considerations of the problems of the global stability of the embankments during the various construction phases are also presented.

INTRODUCTION

In the design of embankments constructed on soft compressible soils, two main problems must be tackled: a) evaluation of the stability of embankments during the various construction phases, b) prediction of settlements and their course in time.

As regards evaluation of stability, the use of common limit equilibrium methods requires evaluation of both the characteristics of resistance the soil has at the beginning, and their variations due to soil consolidation during and after construction.

The second problem is generally solved with relatively simple calculation methods, which nevertheless give quite reliable results; calculation methods based on finite-element programs have recently been used, in which soil simulation takes place by means of elastic or elasto-plastic constitutive laws (Britto and Gunn, 1987; Lepidas and Magnan, 1990; Magnan, 1986; Wroth, 1977). These allow evaluation of soil behaviour in a more detailed and complete way and analysis of consolidation phenomena with models differing from Terzaghi's traditional one. The use of such calculation models is particularly appropriate in the case of thick compressible soil layers or when construction phases go farther in time, as happened in the examples presented in this paper.

The present study examines the time behaviour and stability evaluation of two levées built between 1980 and 1985 in the Po Delta (Fig. 1), through various calculation methods. The results of such analyses are also compared with in situ measurements carried out through geotechnical instrumentation, composed of:
plate bench marks, borehole extensometers, inclinometers and BATs, and Casagrande piezometers (Figs. 2 and 3).

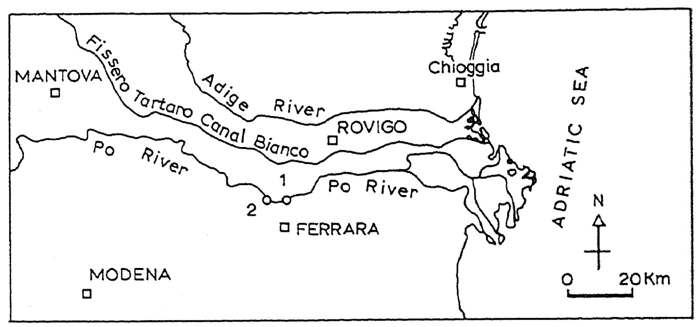


Fig. 1 : Map of investigated sites: 1 Volta Vaccari; 2 Malcantone

Evaluation of the degree of approximation which may be obtained with the various methods, and of difficulties inherent in the design of levées was therefore possible.

CHARACTERISTICS OF LEVEES AND FOUNDATION SOILS

Both the levées under examination were built further back from and replacing already existing levées, due to the need to widen and modify the bed of the terminal stretch of the river Po, the main Italian river (Colleselli and Soranzo, 1987).

Volta Vaccari

The levée is about 1000 m long and is substantially symmetrical. Its foundation plane is 55.0 m wide at the base and 8.0 m at the top; it has two platforms at +0.5 and +2.5 m a.s.l., and is 6.0 m high (Fig. 2).

The following stratigraphy for the foundation

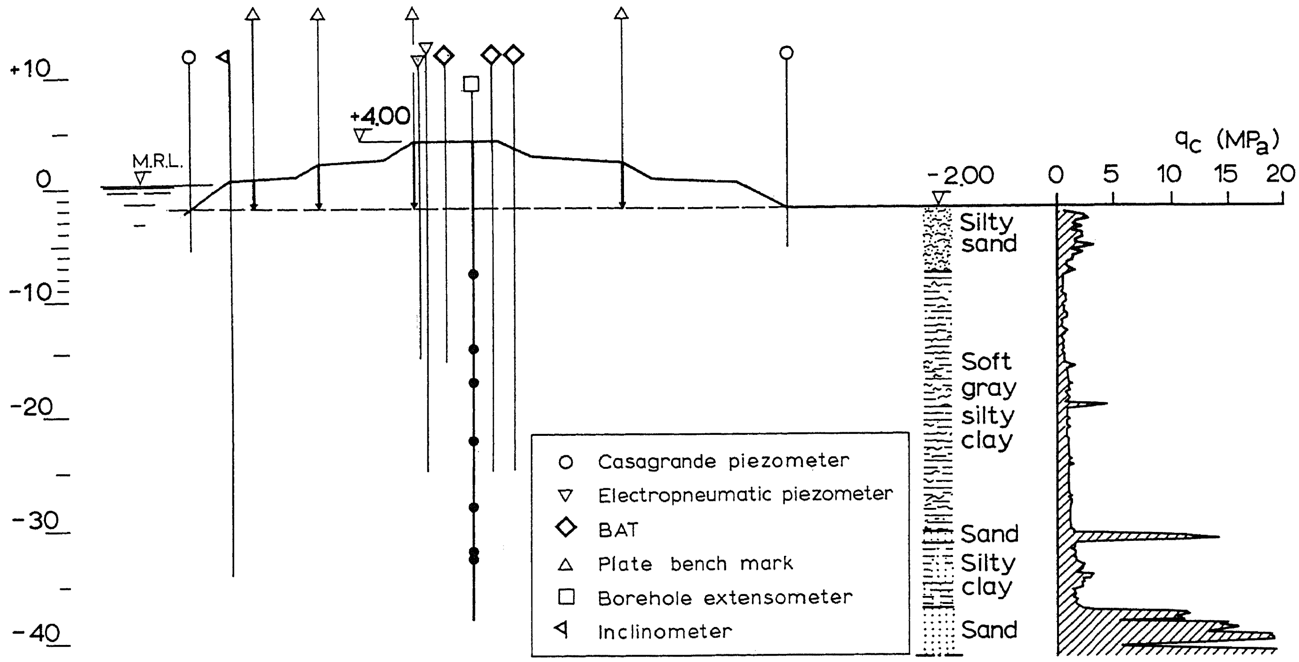


Fig. 2 : Soil foundation characteristics and geotechnical instrumentation (Volta Vaccari).

soils, starting from ground level at -2.0, was found through in situ tests (borehole and penetrometric tests) and laboratory tests (Fig. 2):

a) a 2.0 m thick low-consistency layer of silty clay and clayey silt (tip resistance q_c=0.4÷1.0 MPa; natural water content w_n at about liquid limit w_L);

b) from -4.0 to about -10.0, a layer of quite loose sandy silt and fine silty sand (q_c=0.5÷1.5 MPa; $\varphi=27^\circ$ for silts, $\varphi=37^\circ$ for sands);

c) from -10.0 to -30.0 ÷ -32.0, a very thick, normal consolidated layer of silty clay (c_u/σ'_{vo}=0.22), whose characteristics improve with increasing depth (undrained shear strength c_u=20÷30 kPa, w_L=43÷46, I_p=14÷17, C_c=0.32÷0.39 in upper layers; c_u=25÷33 kPa, w_L=51÷55, I_p=25÷38, C_c=0.32÷0.39 in central layers; c_u=40÷80 kPa, C_c=0.44÷0.66 in deepest layers);

d) from -32.0 to -45.0, a very dense sandy layer ($q_c \geq 10.0$ MPa).

Malcantone Levée

The Malcantone levée is 500 m long, 75.0 m wide at the base and 9.0 m wide at the top, with platform and subplatform. It is 9.0 m high and is asymmetrical.

In this second levée, the characteristics and thicknesses of the foundation soils differ from the previous ones; the stratigraphy of the foundation soil (fig. 3) is composed of:

a) from ground level (average +7.0 m a.s.l.) to +3.5 alternating layers of clayey silts and sandy silts;

b) from +3.5 to -5.0 a clayey layer of medium-high to high plasticity and medium-low to medium consistency (undrained cohesion c_u=15÷30 kPa, w_l=56÷64, I_p= 25÷32 from the shallowest to the deepest layers, C_c=0.33÷0.36);

c) from -5.0 to -7.0 alternating layers of sandy silts and silty sands;

d) from -7.0 to -10.0 a clayey layer of medium to high plasticity (undrained cohesion c_u=22 kPa, w_l=45÷55, I_p=20÷28, C_c=0.30÷0.35);

e) from -10.0 to -25.0 a layer of dense sand with thin silty intercalations (N_{SPT} = 32÷38)

In this case, the dependence of pore pressure in soil on the hydrometric level of the river was found through accurate piezometric measurements carried out over a prolonged period of time. Moreover, laboratory tests revealed that the clayey layers are slightly overconsolidated, with values ranging from 2.0 to 1.1 with increasing depth. Correct evaluation of the overconsolidation ratio turned out to be of fundamental importance for evaluation of the

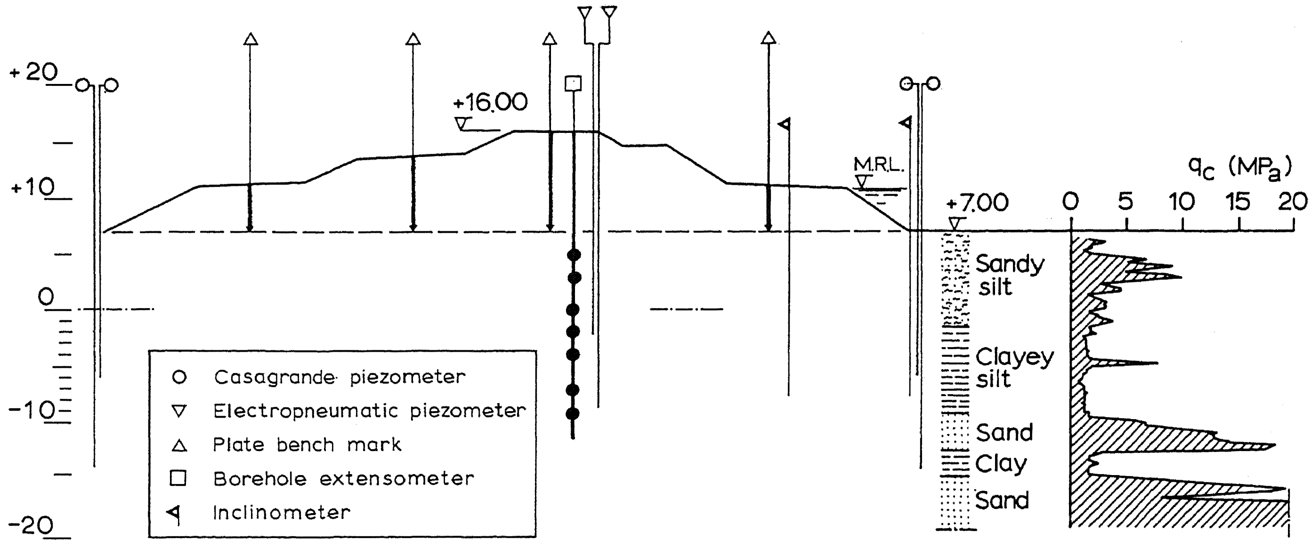


Fig. 3 : Soil foundation characteristics and geotechnical instrumentation (Malcantone).

parameters necessary for application of elasto-plastic models.

For both levées, the values of shear strength and strain parameters obtained from in situ and laboratory tests agreed well with data from many researches carried out by Italian workers on these types of clays (Jamiolkowski, Lancellotta and Tordella, 1980; Bilotta and Viggiani, 1975).

CALCULATION METHODS

In the design phase, evaluation of the condition of stability was made using the traditional Bishop method; settlements were obtained as the sum of an immediate elastic settlement and a later consolidation settlement, determined calculating pressure increments in soil with the elasticity theory and on the basis of the results of oedometric tests.

In the present study, back-analyses with finite-element calculation methods were carried out to find settlement time trends.

The finite-element program provides for the use of hardening elasto-plastic laws (Cam clay, modified Cam clay) and linear elastic laws adopted respectively for clayey and sandy layers. In order to carry out coupled analysis, the six-noded linear-strain triangular element was used for the evaluation of pore overpressures.

The meshes of the Volta Vaccari and Malcantone levées are shown in figures 4 and 5; the first consists of 469 elements and 257 nodes, the second consists of 647 elements and 360 nodes.

In order to simulate levée construction realistically and thus take into account the effective load diagram, the elements representing the structure were added in various time steps and their weight was applied, subdivided inside each time-step.

A further back-analysis of the Volta Vaccari levée only was conducted evaluating the stress state induced by overloads in the soil, and the immediate settlement was calculated using a finite-element program, hypothesizing soil in elastic conditions.

Consolidation settlement was then assessed by the piece-wise linear method, adopted for analysis of consolidation phenomena in very thick layers (Yong, Siu and Sheeran, 1983). This calculation method consists of a finite difference program, which accounts for variability in void ratio e and coefficient of permeability k, according to effective pressures acting in the clayey layers. Taking into account laboratory and in situ test results, the following equation for the variation of k was adopted (Juarez-Badillo, 1986):

$$k(\sigma) = 0.49 * 10^{-9} * (\sigma'_v/300)^{-0.9367} \text{ [m/s]} \quad (1)$$

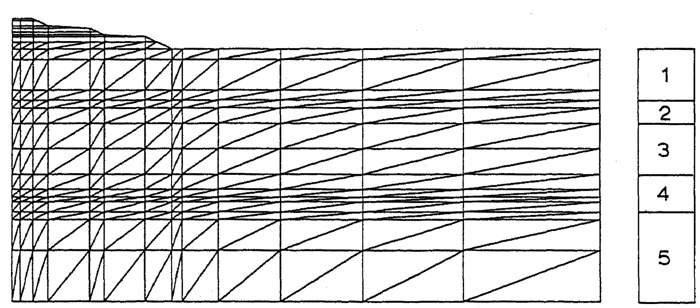


Fig. 4 : Mesh and soil layers (Volta Vaccari).

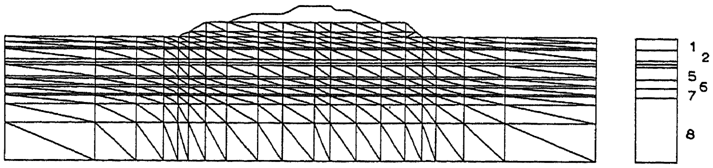


Fig. 5 : Mesh and soil layers (Malcantone)

Reference to oedometric tests was made for void ratio versus vertical effective stress relationships.

The amount of data available, in both design and testing phases, was sufficient to give a precise and significant scheme of the soils and to determine the parameters necessary for the various analyses conducted. Empirical correlations were used for the determination of elastic moduli of sands, linking them to penetrometric tip resistance (Ladd, Foott, Ishihara, Schlosser and Poulos 1977).

Soil characteristics adopted in the various schemes are reported in figures 2 and 3 and tables 1, 2 and 3.

EVALUATION OF STABILITY

Volta Vaccari Levée

During the design phase, a preliminary evaluation carried out in the expectation of rapid levée construction, without variations in the initial strength properties of the foundation soils, indicated a safety coefficient of 1.0. Staged construction was found to be necessary, in order to improve strength characteristics. The levée was thus first constructed to a height of about 3.0 m, and completed to a height of 6.0 m about 6 months later. It was hypothesized that, during this period, undrained shear strength cu of the clayey layers would increase by an average of about 20%. With these modified strength characteristics, the stability of the finished levée turned out to have a safety coefficient of 1.4.

Table 1 : Soil foundation parameters of F.E.M. analysis (Volta Vaccari)

Layer	κ	λ	Γ	M	E [MPa]
1					1.4
2	0.030	0.165	2.56	1.30	
3	0.035	0.170	2.79	1.07	
4	0.052	0.269	3.39	1.02	
5					40.0

κ = swelling line slope;
λ = normal consolidation line slope;
Γ = specific volume value corresponding to $p'=1.0\ kNm^{-2}$;
M = critical state line gradient;
E = elastic modulus.

Table 2 : Soil foundation parameters of elastic F.E.M. and piece-wise linear consolidation method of analysis (Volta Vaccari).

Layer	E [MPa]	Eu [MPa]	Cc
1	1.4		
2		12.0	0.38
3		21.0	0.39
4		22.0	0.62
5	40.0		

Table 3 : Soil foundation parameters of F.E.M. analysis (Malcantone)

Layer	κ	λ	Γ	M	E [MPa]
1					20.
2	0.035	0.156	2.50	0.984	
3					7.0
4	0.042	0.200	2.61	0.898	
5	0.037	0.163	2.59	0.898	
6					15.0
7	0.052	0.138	2.65	0.984	
8					80.0

Back-analysis was carried out, in which undrained shear strength c_u was determined by using Ladd's (1991) empirical equation:

$$c_u/\sigma'_{vc} = S(OCR)^m. \qquad (2)$$

OCR=1, S=0.2+0.05*Ip (S=0.21-0.22) was assumed. Stress σ_{vc} was determined by finite-element numerical analysis as a function of stress variations due to levée construction. The increase in c_u was thus estimated on average to be 25% and 10%, respectively at the top and base of the clayey layer; it did not vary substantially throughout the central part, in which consolidation was practically nil. Evaluations of stability hypothesizing the finished levée and using the above strength characteristics, gave a safety coefficient of 1.25. Although acceptable, this value was slightly lower than the design value, due to the fact that numerical analysis identifies the degree of consolidation in the clayey layer in greater and more accurate detail.

Malcantone Levée

During the design phase, evaluation of stability was always carried out hypothesizing rapid construction of the whole work without variations in the initial strength characteristics of the various soil layers. The safety coefficient - 1.25 - was acceptable, considering completion of the work without interruption. In this case, no evaluation was carried out, in view of the evolution of shear strength.

EVALUATION OF SETTLEMENTS AND PORE OVERPRESSURES

Volta Vaccari levée

According to design evaluations, the final maximum settlement in the centre of the levée was about 1.15 m. Times required to reach 50% and 90% of the average degree of consolidation were assessed using Terzaghi's classic one-dimensional theory, and turned out to be respectively 920 and 3960 days.

Measured settlements after 1550 days were 1.10 m and 0.78 m respectively at the centre and towards the edges of the levée.

In the back-analysis, settlements and pore overpressures were also calculated using the two more complex above mentioned methods. With the piece-wise linear method, settlements varied from 0.33 m to 1.03 m along the foundation soil and the average degree of consolidation of the clayey layer was about 43% after 1550 days. Calculated final settlements were 1.37 m at the centre and 0.44 m at the edges of the levée.

Back-analyses conducted with the finite-element program and constitutive Cam-clay and modified Cam-clay laws allowed identification of the global behaviour of the soil under the levée, giving slightly varying results. Along the foundation plane settlements turned out to vary between 0.35 m and 1.10 m after 1550 days and between 0.42 m and 1.41 m at the end of consolidation.

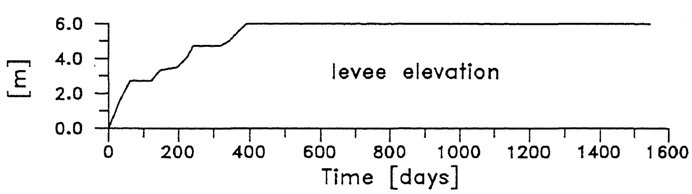


Fig. 6 : Loading diagram (Volta Vaccari).

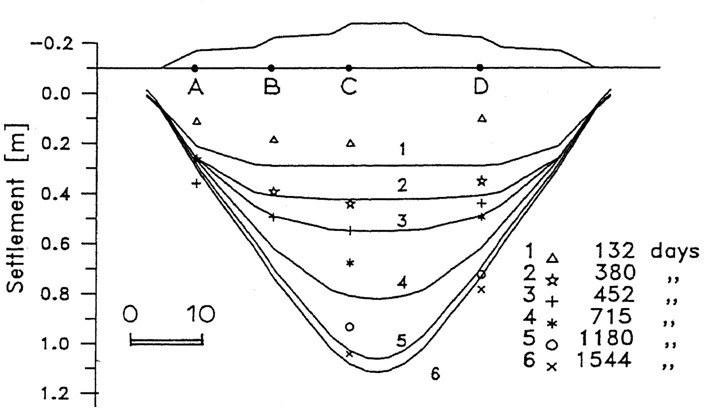


Fig. 7 : Calculated (F.E.M., modified Cam-clay) and measured foundation settlements (Volta Vaccari).

Superficial settlements and those along the borehole extensometers at various depths agreed both with real measurements and with those calculated using the piece-wise linear method (Figs. 6, 7, 8 and 9).

Pore overpressure measurements from two electropneumatic piezometers at different depths in the clayey layer were close to those given by the two numerical methods (Fig. 10).

Horizontal displacements assessed with the finite-element program showed trends similar to those measured but differing in absolute value. Maximum recorded displacement was 0.15 m - less than the predicted 0.22 m (Fig. 11).

Back-analyses carried out using both calculation methods gave practically identical results and were sufficiently reliable to indicate the behaviour of the whole soil mass (Fig. 12).

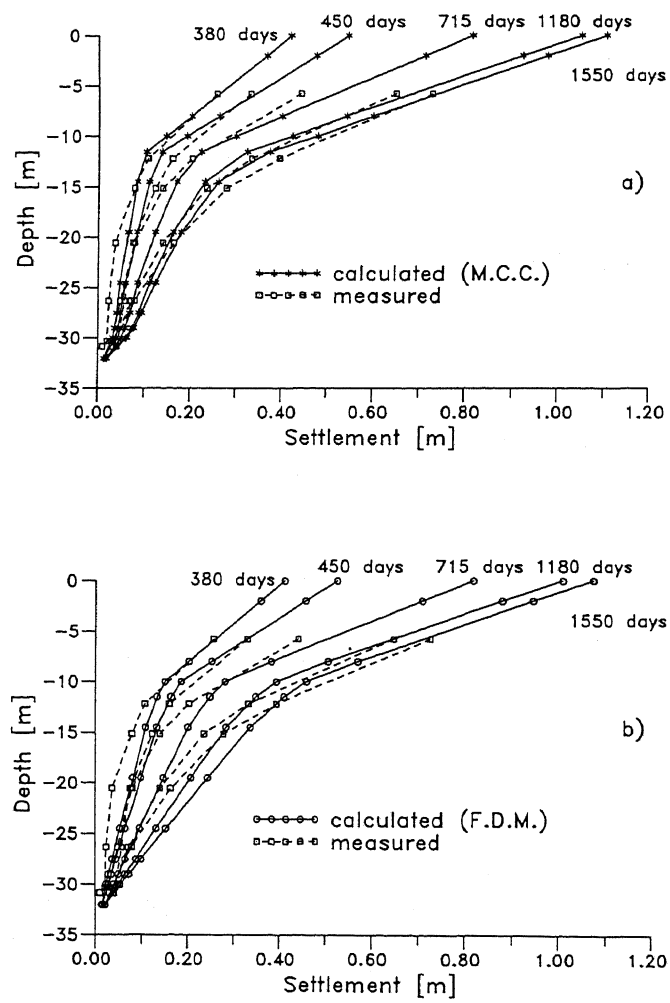
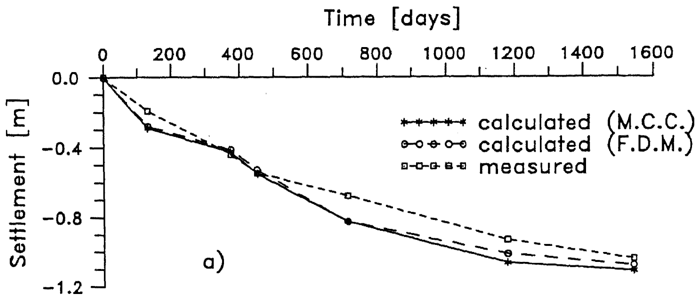


Fig. 8 : Settlements versus depth: a) F.E.M. (modified Cam-clay) - measured; b) P.W.L. consolidation method - measured (Volta Vaccari).



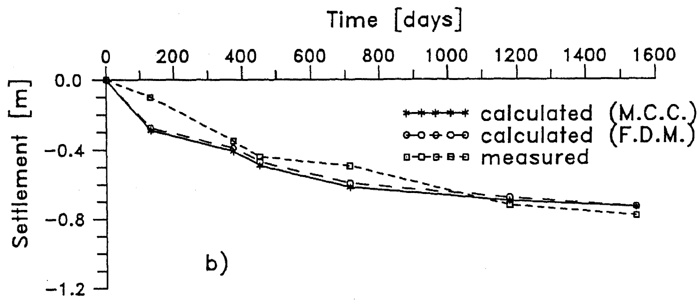
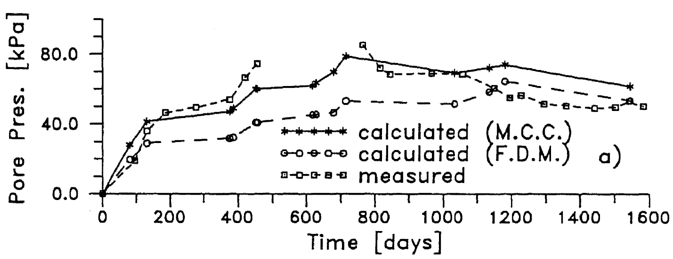


Fig. 9 : Settlement-time relationship according to F.E.M. (modified Cam-clay), piece-wise linear consolidation method, measured: a) point C; b) point D (Volta Vaccari).



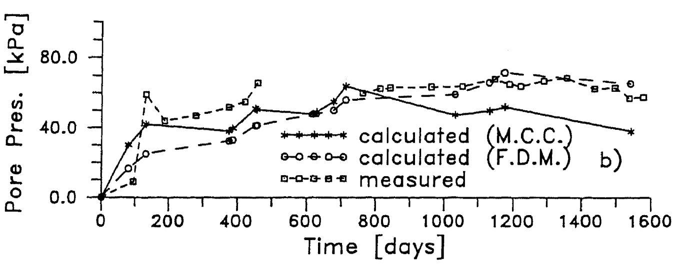


Fig. 10 : Excess pore pressure: a) PEP1(-15.0); b) PEP2(-25.0) (Volta Vaccari).

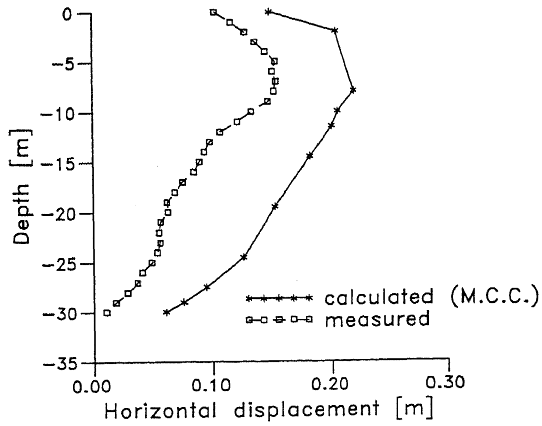


Fig. 11 : Calculated (F.E.M., modified Cam-clay) and measured horizontal displacements (Volta Vaccari).

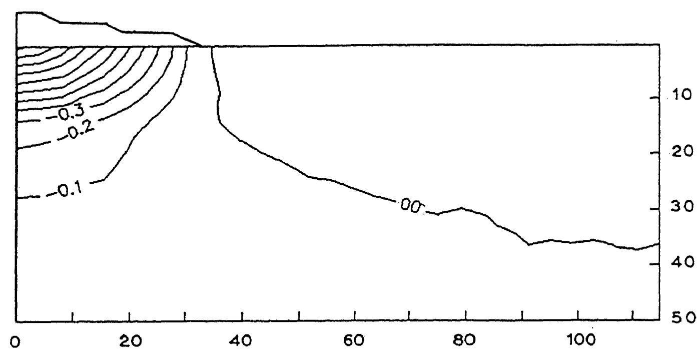


Fig. 12 : Calculated settlements (F.E.M., modified Cam-clay) in the soil (Volta Vaccari).

Malcantone Levée

During the design phase, settlements due to final maximum consolidation were estimated at 0.95-1.0 m and 0.60-0.65 m, respectively at the centre and towards the edges of the levée, with consolidation times of 150 and 500 days to reach respectively 50% and 90% of the average degree of consolidation, again according to Terzaghi's theory. Numerical back-analyses, using the finite-element program and considering the influence of the hydrometric level of the river in the sandy layers between the clayey ones, gave total settlements along the foundation plane varying between 0.20 m and 0.90 m, fitting experimentally determined values (Figs. 13 and 14). Settlements over time along the borehole extensometer were sufficiently well simulated by the numerical analyses (Fig. 15). Only at the deepest measurement point (-16.0 m) was a more marked discrepancy found, probably due to instrumental error.

Although always greater than real horizontal displacements, calculated displacements (maximum 0.22 m) were definitely more reliable than in the Volta Vaccari levée, as regards both trend and value (Fig. 16).

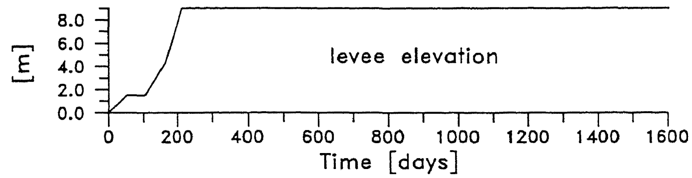


Fig. 13 : Loading diagram (Malcantone).

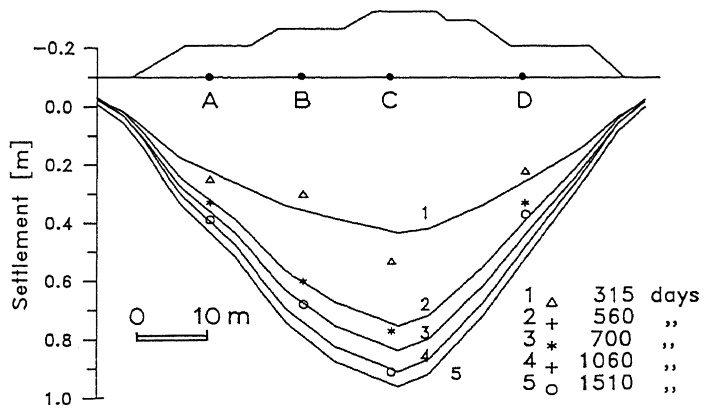


Fig. 14 : Calculated (F.E.M., modified Cam-clay) and measured foundation settlements

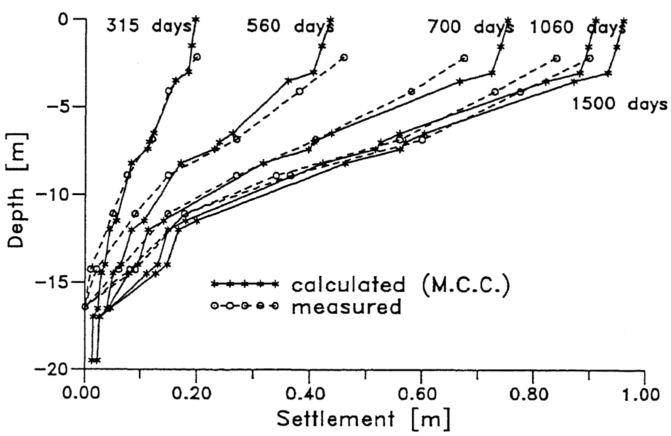


Fig. 15 : Settlements versus depth F.E.M. (modified Cam-clay) - measured (Malcantone).

Poor functioning by the electropneumatic piezometers in the clayey layers (due to the presence of gas) did not allow comparison with the numerical results, so that one last element on the validity of the numerical analyses was lacking.

CONCLUSIONS

The classic calculation methods used in this project give substantially correct and reliable predictions of the behaviour of levées as regards total settlements and consolidation times.

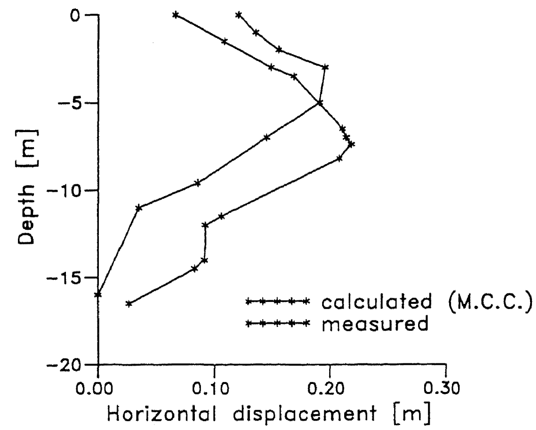


Fig. 16 : Calculated (F.E.M., modified Cam-clay) and measured horizontal displacements (Malcantone).

The more complex methods used during back-analysis gave more complete results, very close to measured values both as regards degree of settling and pore overpressures and their time trends. Their capacity for good simulation of this type of phenomenon was thus confirmed.

These numerical calculation methods, which have now become design methods, also allow better identification of the shear strength parameters which are linked to the consolidation process and introduced into stability evaluations.

It must be borne in mind that numerical calculation strongly depends on accurate knowledge of soil stratigraphy and of the geometrical characteristics of the construction, its good simulation, and the choice of appropriate boundary conditions. For realistic predictions on the behaviour of levée foundation soils, a high number of in situ and laboratory geotechnical tests are necessary.

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