Importance of full scale tests for the design of levees

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Summary

This paper illustrates a case history, based on full-scale tests (trial embankment). More specifically, the settlements of a full-scale trial embankment on a very compressib?e soil (organic clay) for a period of 5 years are reported. The observed settlements are compared with those obtained from an A type prediction, as well as a B type prediction. The analysis results suggest the importance of an appropriate modelling of secondary settlements of organic soils. The difficulties in obtaining good quality undisturbed samples of organic clay and the peculiar creep behaviour point out the importance of full-scale tests.

1. Introduction

This paper deals with the geotechnical design of levees in flood - plain areas. As far as this type of river embankment is concerned, the main potential failure mechanisms can be summarised as follows:

– various types of internal erosion phenomena;
– external erosion due to overtopping;
– slope instability;
– hydraulic heave/up - lift;
– excessive settlements

The durability of a river embankment mainly depends on its capability to:

– survive after the occurrence of overtopping phenomena;
– experience, during its life, limited settlements in order to avoid the need of continuous refurbishments for the maintenance of the Serviceability Limit State (SLS)
– avoid the occurrence of excessive settlements that could lead the structure to its Ultimate Limit State (ULS);
– avoid the occurrence and propagation of internal erosion phenomena;
– resist to mechanical/hydraulic actions even under to fully saturated condition.

Some of the above mentioned aspects could be studied throughout an appropriate combination of laboratory/in situ testing and numerical analyses. In other cases, experimentation of full (or reduced) scale trial embankments is unavoidable. Eventually, a good combination of numerical analyses and full scale tests offers a better understanding of the considered phenomena and their engineering judgement.

This paper illustrates a case history, based on full-scale tests (trial embankment). More specifically, the settlements of a full-scale trial embankment on a very compressible soil (organic clay) for a period of 5 years are reported. The observed settlements are compared with those obtained from an A type prediction, as well as a B type prediction.

2. Levees on very compressible foundation soil

The Massarosa City Council (Lucca – Italy) entrusted the Department of Civil Engineering of the University of Pisa with the geotechnical design of a 3 km long flood – plain embankment. The purpose was to realise a lamination basin having a total area of about 180'000 m^2 for the flood risk mitigation in the Gora di Stiava site.

The main geotechnical issues, related to the construction of these 3 km long new embankments of variable height between 1.0m - 2.8m, can be summarised as follows:

– loss of stability induced by the undrained/drained failure of the foundation soil;
– loss of stability and/or serviceability because of (excessive) settlements of the foundation soil.

In fact, nonetheless the embankment height is quite small (never exceeding 2.8 m) the foundation soil mainly consists of very compressible and soft clayey peat/peaty clay.

A (full scale) trial embankment was constructed in the study area and monitored for five years. The measured/observed settlements are compared against the results of a 2D FEM analysis and a 1D simplified approach (Zeevaert, 1972).

The analyses were carried out by using PLAXIS 2D (Al-Khoury *et al.*, 2008). The paper also summa-

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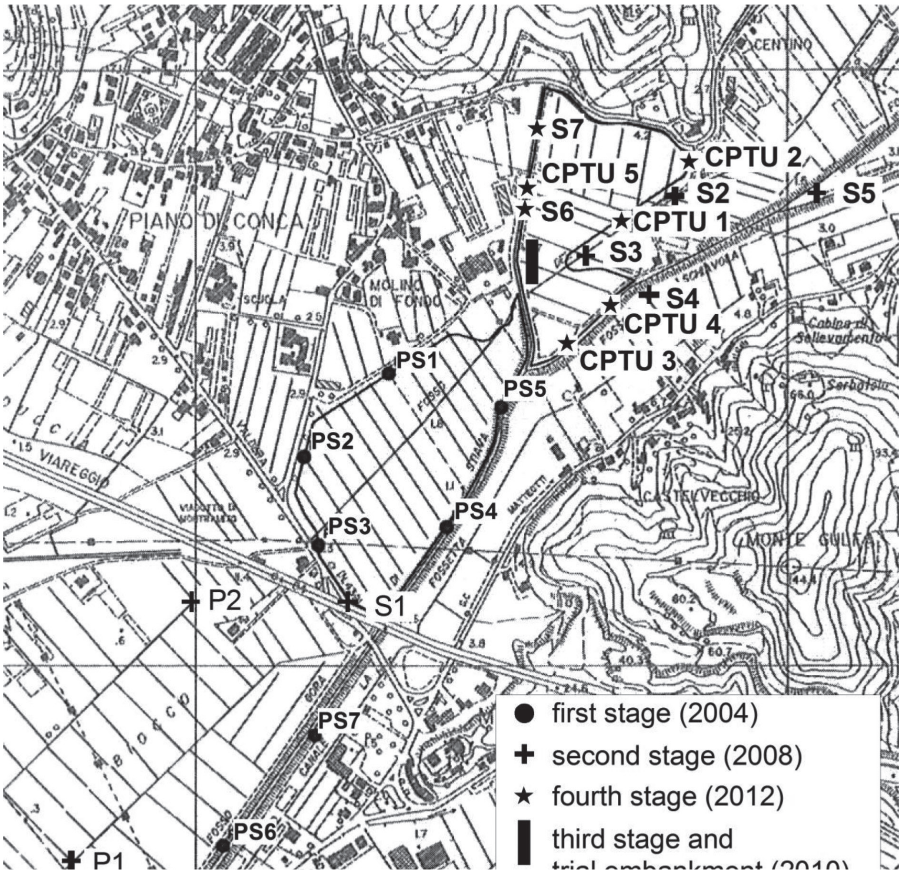

Fig. 1 – Location of in situ investigations
Fig. 1 – Planimetria delle indagini effettuate.

rizes the results of the investigations carried out in the study area at different stages giving a simplified geological/geotechnical model of the subsoil.

2.1. Geology of the Study area

The study area (Fig. 1) is located in the western part of the Massarosa territory, upstream the viaduct of the Lucca-Viareggio highway (Italy, Tuscany Region). This area is almost flat with the average elevation of the ground surface between 1.1m - 3.4m a.m.s.l. More precisely, the study area is located in a plain delimited N – E by the buttresses of the Apuan Alps and S – W by the Tirrenian Sea. In the study area the plain is filled from top to bottom by recent (Holocene) fluvio-lacustrine and silting deposits (*„depositi di colmata"* is the local name) followed by recent (Holocene) peat lacustrine deposits and by recent (Holocene) alluvial deposits. The texture of the alluvial deposits is quite variable but in the study area a sandy/silty texture is mainly observed. The bedrock is at hundreds of meters depth and mainly consists (from bottom to top) of 1) the torbidites of the Macigno Fm (from the Cretaceous to the Eocene – lower Oligocene), with sandstones in the basal part and sandstones and pelites in the middle-upper part; 2) the "Complesso di base" Fm (Paleocene-Eocene) with claystone and interbedded limestones: with which the calcareous torbidites of the Eocene are associated, and of a prevalently arenitic and conglomeratic upper Oligo-Miocene portion.

2.2. Geotechnical investigations

Geotechnical investigations were carried out in different stages. Initially, in 2004, seven static Cone Penetration Tests (PS1 to PS7) with Begemann cone (CPTm) were driven down to depths between 8.2m - 15.2m from the ground level. The second stage of investigation was performed in 2008 and consisted of five boreholes (S1 to S5) down to depths between 11.7m - 27.3m from the ground level, two CPTm (P1 and P2) driven down to depths between 4.6m - 5.2m from the ground level and (variable head) Lefranc permeability tests performed inside the borehole S1.

A third stage of investigation (2010) consisted of a down-hole seismic test in a 30 m borehole to estimate the V_{s30} parameter (*i.e.* the seismic soil category according to EC8 and the prescriptions of the Italian Building Code - NTC2008) and two Cone Penetration Tests with a piezocone (CPTU) driven down to a depth

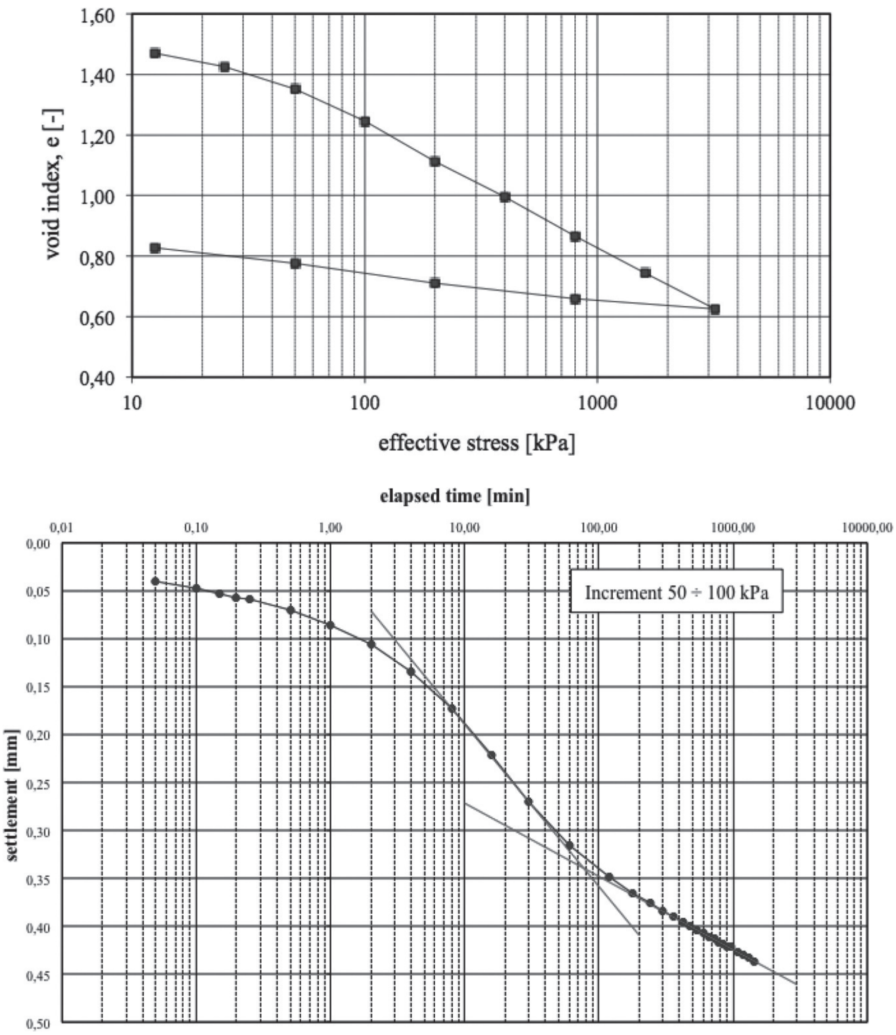

Fig. 2 – Typical result of oedometer test.
Fig. 2 – Tipico risultato delle prove edometriche effettuate.

of 20 m from ground level. The penetration was interrupted at depth of 4.94 m and 9.60 m in order to perform dissipation tests. Both DH and CPTU tests were located within the area covered by the trial embankment.

Further investigations (Fig. 1) were carried out on 2012, and consisted of two boreholes and five CPTU (down to 10.5 m). Five "undisturbed" samples were collected in the two boreholes within the first five meters. Laboratory tests on "undisturbed" samples consisted of soil classification, oedometer and direct shear tests. Some difficulties have been encountered in definition of grain size distribution, mainly due to organic fraction. In fact the weight organic fraction was between 8.4% and 12.9%. Notwithstanding accurate sampling and specimen preparation, the soil revealed a delicate structure. Definition of OCR was doubtful since the edometric compression results were as reported in Fig. 2.

Values of Compression Index deduced by oedometer test were between 0.166 and 0.736, depending on sand content. Values of Secondary Compression Index, $C\alpha\varepsilon$, were between 10^{-2} and 10^{-3}.

2.3 Stratigraphic/Geotechnical model

The following simplified stratigraphic model has been inferred from the whole investigations and local geology:

– a first shallow layer with a thickness of 2-3 metres exhibits a tip resistance between 1 - 3MPa (Horizon A);
– a second very soft layer with variable thickness from 1 to more than 20 meters and tip resistance of less than 0.5 MPa (Horizon B);
– a third layer, mainly consisting of dense sandy soils with a tip resistance greater than 10MPa (Horizon C).

Fig. 2 show the basic results and soil profiling as obtained from CPTU carried out in 2012. Similar results were obtained from the 2010 CPTU.

Tab I – Shear wave velocity profile obtained by pseudo-interval method (ROBERTSON *et al.*, 1986).
Tab. I – Profilo della velocità delle onde di taglio ottenuto mediante la tecnica dei pseudo intervalli (ROBERTSON et al., 1986).

Depth (m)	Vs (m/s)
0 - 4	319
4 – 10	55
10 – 15	79
15 – 23	125
23 – 30	358

The very poor mechanical characteristics of Horizon B and the extremely high variability of its thickness clearly indicate that such a Horizon is the most critical in negatively affecting the embankment performance. The shear wave velocity profile measured in 2010 and reported in Table 1 confirms such an evaluation.

The CPTm carried out in 2008 show that the thickness of Horizon B increases from south-west to north-east direction. Figure 4 shows the depth from the ground level to the top of the sandy layer (Horizon C). The contours of the detention basin are also shown in the same figure.

The design of the detention basin embankment followed a devious administrative path, as proved by different phases of soil investigation. In a first time the trial embankment has been conceived as a short time trial structure supporting the design options, as a consequence only settlements have been measured and other measurement, such as pore pressure, have been omitted. After the planned period the embankment was not removed; as a consequence further measurements of settlements have been carried out.

The geotechnical parameters have been deduced by information available at each time. In a fist time the soil parameters have been derived simply by fitting observed settlements. In the following stages more information have been integrated, as they were available. Further details about soil parameters are discussed in the following paragraph.

2.4. The trial embankment and settlement monitoring

A 2.5 meters high and 45 m long trial embankment was constructed and monitored in the period 2010 - 2015. The cross section has a width of 12 m at the base. Inclination of the side slopes (i.e. 2/3) was the same of the embankment under design. In practice it is such as a full scale embankment (in fact maximum height of the embankment under design is 2.8 m).

The monitoring system consisted of 15 settlement probes specially built. Each probe has been made by a steel plate resting on the foundation plan

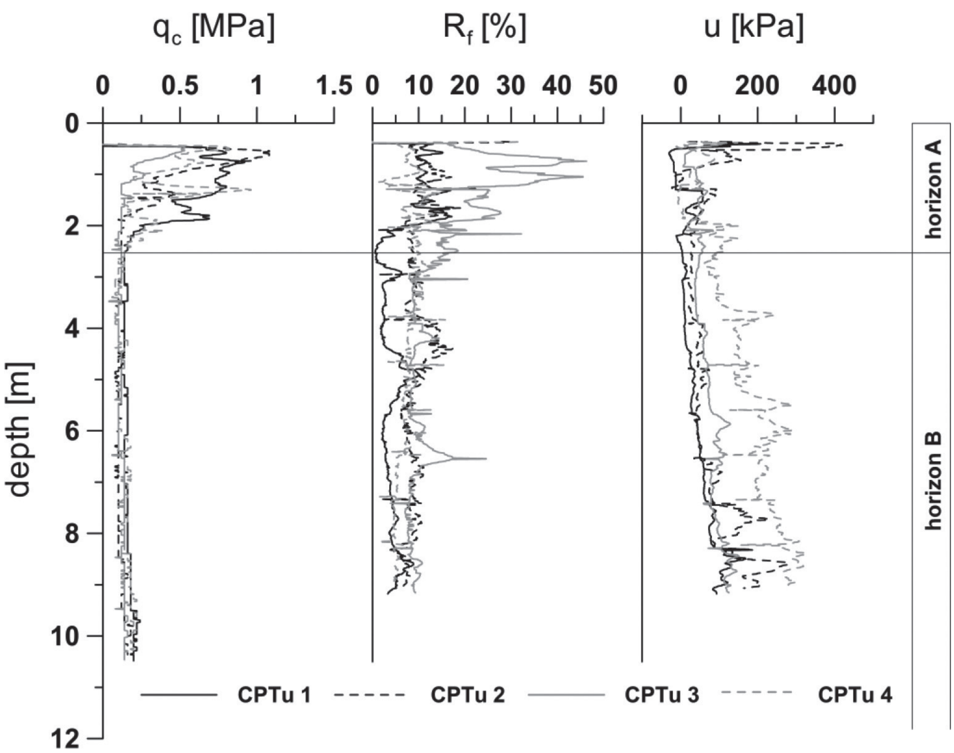

Fig. 3 – Basic results of 2012 CPTu.
Fig. 3 – Risultati delle prove CPTu effettuate nel 2012.

Fig. 4 – Numbers indicate the depth (in meters) of the top of the sand layer from ground level.
Fig. 4 – Profondità in metri del tetto dello strato sabbioso.

with a vertical rod which allows levelling from the top of embankment. The probes have been installed on the base of the embankment before construction in order to have the measurement of the whole effects of weight application.

The trial embankment was constructed within three working days, during which the settlements monitoring system were constantly operative.

Figure 5 shows the location and sizes of the trial embankment with the indication of the measurement and reference points. The trial embankment was located in the most critical area where the top of Horizon C reaches the depth of about 23 m.

Settlements of the embankment have been measured by means of precision levelling. Four reference points have been defined on existing structures close to the site and their relative movements have been checked.

Figure 6 shows the set of measured settlements. The last measurement has been carried out about five years after the installation of the trial embankment (i.e. in December 2015). The figure shows that the reference points are not moving each other, while the observed settlement gradually increases approaching the centreline of the embankment. From a

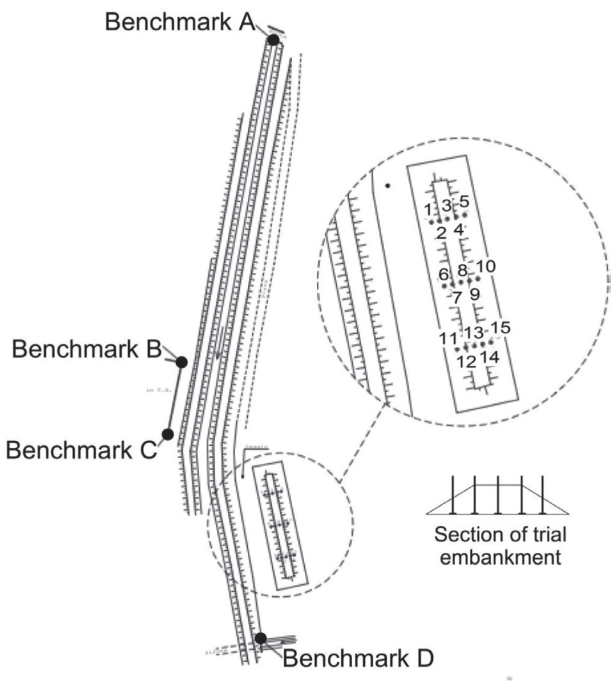

Fig. 5 – Location of the trial embankment and reference points.
Fig. 5 – Pianta dei capisaldi per il monitoraggio dei cedimenti.

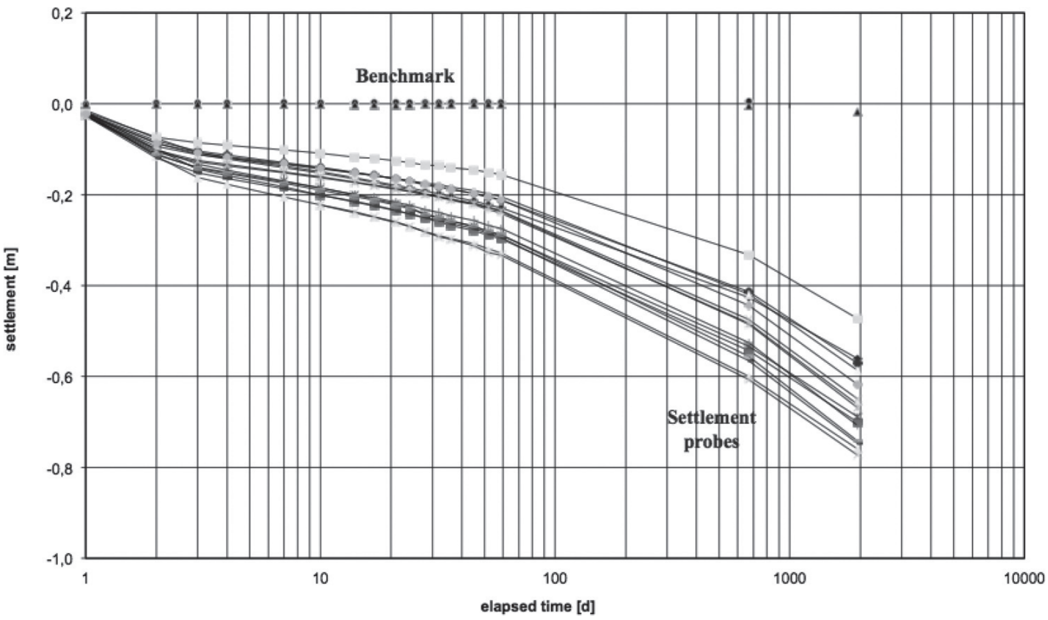

Fig. 6 – Measured settlements of all foundation – points.
Fig. 6 – Cedimenti misurati sul piano di fondazione.

first qualitative analysis of the experimental settlement curve, it is possible to make the following considerations:

– the measured settlements at the end of the construction are relevant (about 15cm);
– the first part of the curve suggests the development of an immediate settlement especially after the first day;
– the observed change of curvature of the curve (settlement – time in a log scale) suggests the rapid dissipation of pore pressure and the presence of an important viscous component;

The first two observations can be related to a lack of an appropriate preparation of embankment base. In fact the settlement gauges have been applied directly on the vegetal soil, as a consequence it is possible that a contemporaneous compaction of natural soil during compaction of first stratum of embankment occurred. This phenomenon has been taken into account in the following discussion.

3.5. Back-analyses

The numerical modelling of the consolidation process has been accomplished by means of the FEM code PLAXIS (PLAXIS, 2010) in two following stages. In fact a first analysis has been carried out in 2010, supported by the first soil investigations and the monitoring of trial embankment during the first 60 days. A further back analysis has been carried out in 2012, supported by further soil investigation and two years monitoring of trial embankment.

The settlements observed in first 60 days were used to calibrate the model and to fix primary and secondary compressibility parameters used in the first analysis and reported in Table 2.

Note - Elastic properties of embankment material have been chosen in order to simulate a very stiff behaviour.

More specifically, the geotechnical parameters of the embankment have been adopted on the basis of the Authors experience. The Authors have done a continuous research activity, in recent years, for the assessment of strength and permeability parameters of compacted soils. The tested soil types are classified as A4 to A6 according to AASHTO M145 (1991). A total of eight B.Sc Thesis and a M.Sc Thesis (VANNUCCI, 2016) have been completed on this topic. The research - results have been partly published in a number of publications (COSANTI, 2013; COSANTI *et al.*, 2013; SQUEGLIA *et al.*, 2013). The main outcome of

Tab. II – Geotechnical Model.
Tab. II – Parametri utilizzati nel modello ad elementi finiti.

Horizon	Model	c' [kPa]	ϕ' [°]	γ [kN/m^3]	OCR	Cc	Cs	C$\alpha\varepsilon$	K [m/d]
A	soft soil	0	25	17	6	0.200	0.030	–	0.432
B	soft soil creep	0	25	17	1.3	0.400	0.050	0.020	0.432
Embankment	E -P	15	30	20	-	-	-	-	-

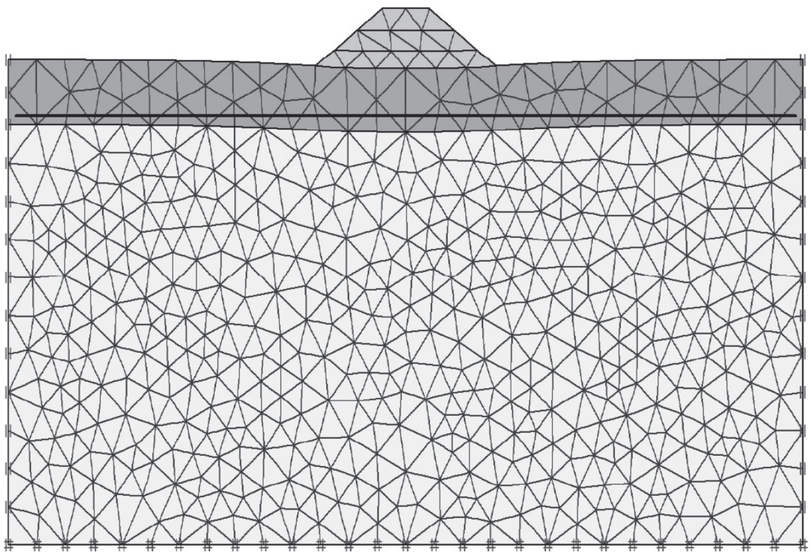

Fig. 7 – The mesh used for first back analysis.
Fig. 7 – Mesh utilizzata nel modello ad elementi finiti.

the above research is that tested silty mixtures (compacted at > 90 % of optimum) exhibit an apparent cohesion of not less that 15 kPa and a friction angle of not less than 30°. This evidence has been obtained using fully saturated specimens.

The permeability (K) was deduced from Lefranc tests. Dissipation tests gave too much lower values of the permeability, not consistent with the observed trend of the settlements of the trial embankment, as will be discussed in more detail later on. The undrained shear strength and OCR were inferred from CPTU tests, the unit weight (γ) was obtained from CPTU by means of the following correlation (MAYNE, 2009)

$$\gamma = 26 - \frac{14}{1+[0.5\cdot\log(f_s+1)]^2} \quad (1)$$

The effective strength parameters were adopted so that the strength envelope was consistent with the measured Cu/σ'_{vo} values. In doing that, it was assumed that the undrained strength has been reached through an s' constant effective stress path. This assumption leads to a possible underestimation of the strength parameters, since it leads to the maximum estimation of s' at failure.

Compressibility parameters were derived by fitting the observed settlements of the trial embankment. In particular a trial and error iterative procedure was adopted in order to match the experimentally observed settlements. Indeed, on 2010, at the time of such a class A prediction, laboratory tests were not available.

The viscous component of settlements was considered only for Horizon B, since the soft soil creep model has been used. The experimental embankment was modelled as a volume of soil with a trapezoidal section. Horizon C was assumed as a rigid boundary.

In the first analysis, construction was modelled in three stages to allow for the pore pressure dissipation during night time with related settlement. This was necessary because of the almost immediate appearance of viscous component (change of curvature of the consolidation curve). Figure 7 shows the mesh used for the first back-analysis. Boundary conditions have been selected according to standard fixities. Initial pore water pressure has been considered hydrostatic and all boundaries have been considered permeable.

Figure 8 shows the modelling results obtained in terms of settlements over time in the first analysis. As shown the agreement reached could be considered satisfactory, in particular with regard to the purposes of back-analysis. In any case, the error in the evaluation of the maximum settlement is less than 5%. Continuous line in figure 8 regards the centre point of embankment base (point 8 in Fig. 5).

Monitoring of embankment has been resumed in 2012 and a check of forecasted settlement was then possible. As reported in figure 9, the forecast of maximum settlement produced by first analysis is underestimated by more than 30%.

A second back analysis updated to 2012 data (class B prediction) has been carried out in order to reach a better forecast of maximum settlement. A new calculation has been carried out by means of software PLAXIS, using the same mesh reported in figure 7. A new set of soil parameters has been defined to match the experimental data. The definition of such parameters has been guided by laboratory tests carried out on soil samples. Several oedometer tests provided a first estimation of secondary

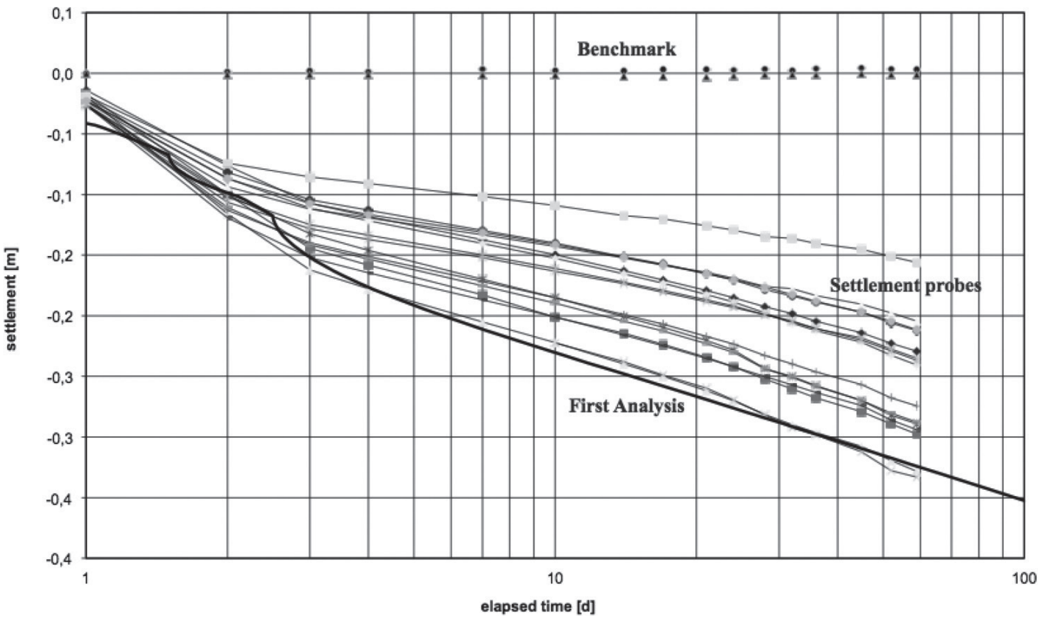

Fig. 8 – Comparison between experimental and calculated performance points for the first analysis.
Fig. 8 – Confronto tra i cedimenti misurati e le previsioni del modello FEM.

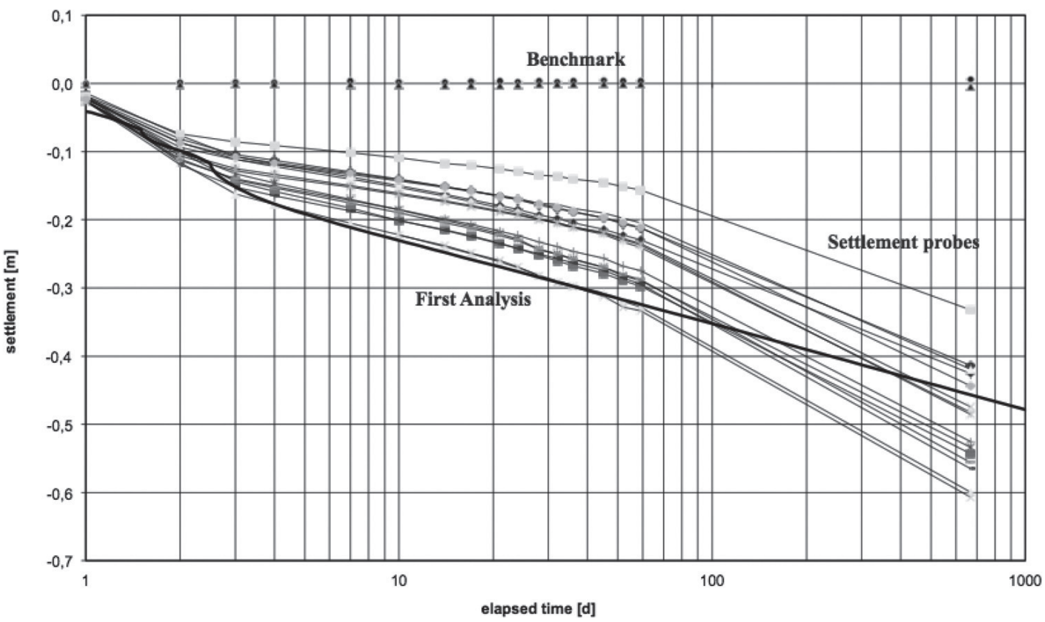

Fig. 9 – Settlements measured up to 2012 and settlement forecast based on Tab. II data.
Fig. 9 – Confronto tra i cedimenti misurati al 2012 e le previsioni del modello FEM basate sui parametri della Tab. II.

consolidation parameter $C_{\alpha\varepsilon}$ and an estimate of K, which partially differs from the values reported in table II. It is worthwhile to remember that the permeability values in Table 2 have been deduced from Lefranc tests. Table III shows the set of soil parameters defined by means of second back analysis (fitting of observed settlements) and new laboratory tests. In any case, it is worthwhile to remark that conventional oedometer tests could not be suitable for the assessment of secondary compressibility parameters of organic/peat soils. Indeed such a parameter could not be constant with logarithm of time and organic/peat soils could exhibit an apparent tertiary creep (see as an example HUAT *et al.*, 2014).

Monitoring of embankment has been again resumed in 2015 and a further check of forecasted settlement has been possible. As reported in figure 10, the forecast of maximum settlement produced by second analysis is underestimated by about 4%. Although the calculated error in settlement forecast

Tab. III – Soil parameters as deduced by second back analysis.
Tab. III –

Horizon	Model	c' [kPa]	ϕ' [°]	γ [kN/m^3]	OCR	Cc	Cs	Cαε	K [m/d]	Cu [kPa]
A	soft soil creep	4	27	18.3	2.5	0.290	0.065	0.020	0.043	35 – 60
B	soft soil creep	5	23.4	16.0	1.1	0.516	0.094	0.025	0.043	10 – 20
Embankment	E -P	15	30	20	-	-	-	-	-	-

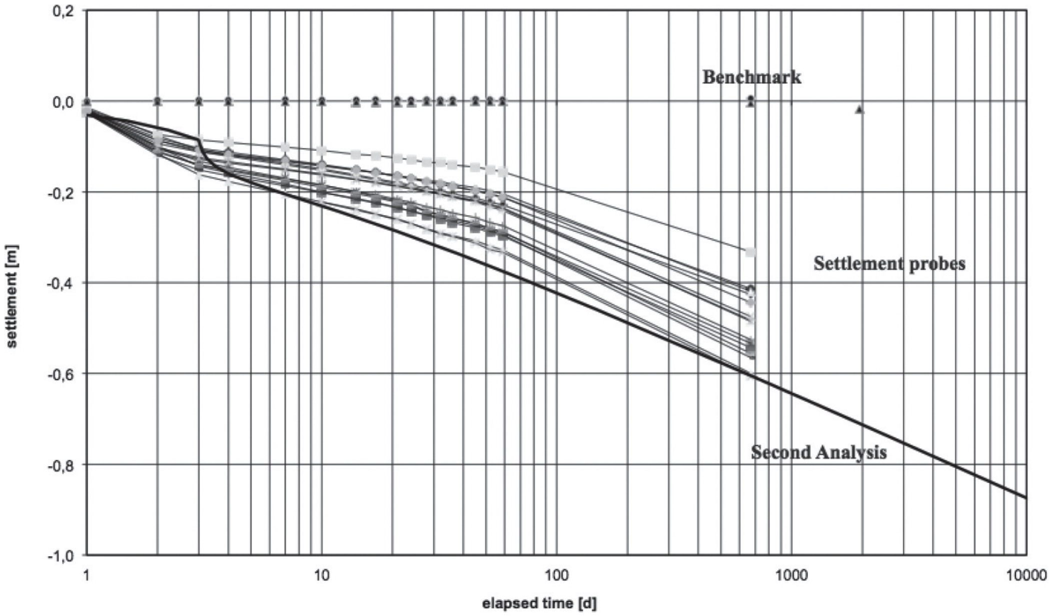

Fig. 10 – Results of second analysis performed on the base of information available in 2012.
Fig. 10 – Risultati della seconda analisi FEM realizzata sulla base delle indagini disponibili al 2012.

is relatively acceptable, the whole progress of settlement indicates an increase of difference between measurement and prediction, with a systematic underestimation.

In order to have a simpler understanding of phenomena related to measured settlement and a better prediction of future settlement, a simple 1-D model based on the hypothesis that the viscous effects occur during both the consolidation phase and under the final effective stress (ZANG and O'KELLY, 2013). This approach is quite reasonable for peaty soils taking into account their microstructural characteristics (LANDVA and PHEENEY, 1980).

In the light of these considerations, a model based on the previous hypothesis has been applied. The model adopted in the back analysis has been proposed by ZEEVAERT (1972) and take into account the dependence of rate of viscous deformation by the correspondent effective stress, particularly appropriate for materials, which exhibit a two level structure like peaty soils (ZANG and O'KELLY, 2013).

The application of ZEEVAERT (1972) model lead to the following expression for volumetric strain

$$\Delta\varepsilon_v = m_v \Delta\sigma'_v \left[U + \beta \cdot \log\left(1 + \frac{4.62}{\beta}T\right)\right] \qquad (2)$$

in which m_v is soil compressibility, $\Delta\sigma'_v$ is the final increment of vertical effective stress, U is the degree of consolidation (TERZAGHI, 1923), T is time factor, β is defined as

$$\beta = \frac{C_{\alpha\varepsilon}}{\Delta\sigma'_v}\frac{1}{m_v} \qquad (3)$$

in which $C_{a\varepsilon}$ is the secondary coefficient deduced by oedometric consolidation tests.

In order to further simplify the calculation of back analysis the following approximate relationship (SIVARAM and SWAMEE, 1977) has been used to estimate the degree of consolidation

$$U = \frac{(4T/\pi)^{0.5}}{\left[1 + (4T/\pi)^{2.8}\right]^{0.179}} \qquad (4)$$

Since the subsoil consists of two different layers (see Fig. 7), two different time factors have been considered. The shallower 2 m thick layer is characterised by a sand content greater than the lower layer. In addition the excess pore pressure developed during CPT penetration is lower than that occurring in the deeper layer. Therefore it has been speculated that the permeability of shallower layer is greater than that of the lower layer, accord-

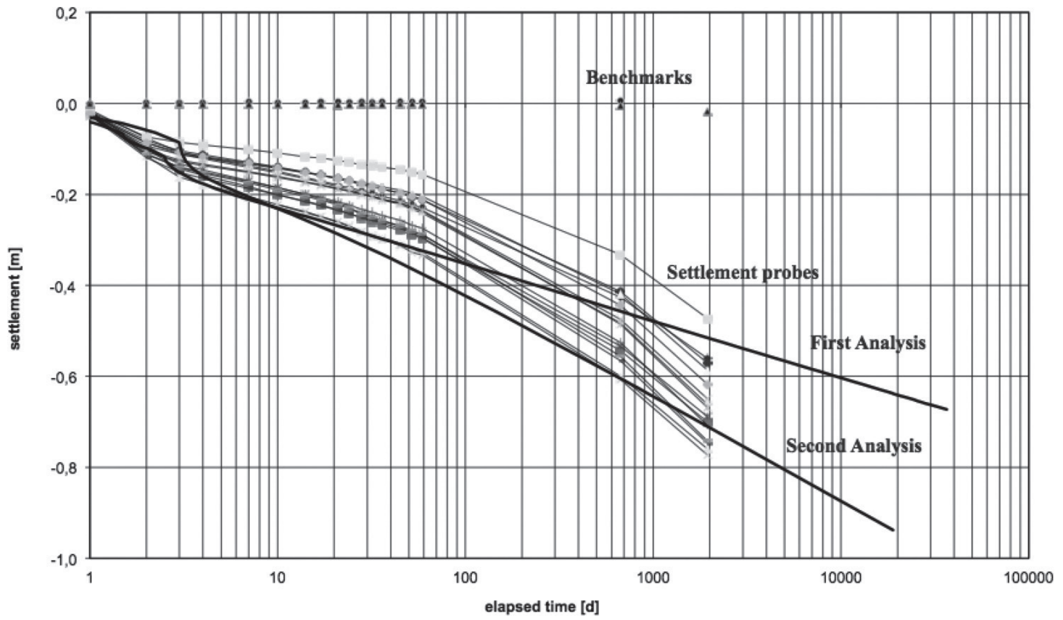

Fig. 11 – Settlements measured up to 2015 compared to first and second analysis results.
Fig. 11 – Confronto tra i cedimenti misurati al 2015 e le previsioni delle due analisi FEM effettuate.

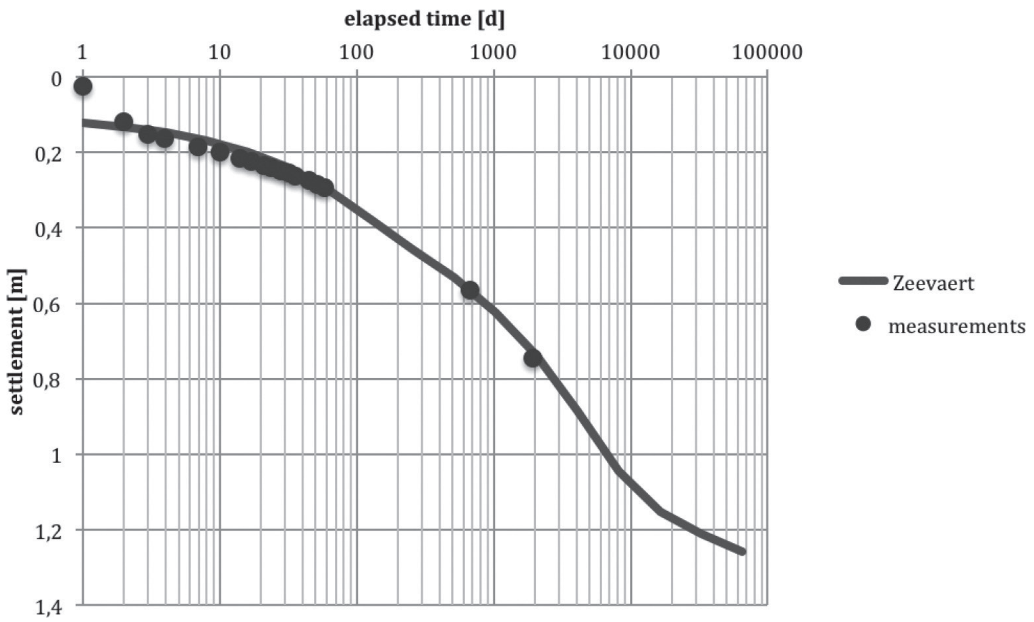

Fig. 12 – Results of 1D model back analysis and forecast of centre point settlement evolution.
Fig. 12 – Risultati della modellazione monodimensionale effettuata mediante il modello di ZEEVAERT (1972) e previsione dei cedimenti futuri.

ing to the results from laboratory tests. As a consequence the drainage boundary condition of two layers were different: the shallower layer has only an upper drainage, whereas the deeper layer has a double draining boundary.

A further consideration regards the measured settlement after the first day. Measurements, reported in figure 6, clearly show that a very rapid settlement occurs within 48 hours from the beginning of embankment construction. As reported above, a lack in base preparation could be the cause of this settlement; as a consequence the settlement deduced by 1D model has been corrected by 0.1 m in order to take into account this phenomenon. The parameters used in the described models have been deduced by soil investigation and slightly adapted to match the experimental data and they match the values reported in table III. Figure 12 shows the results of calculation and the prediction of its growth for the next decades.

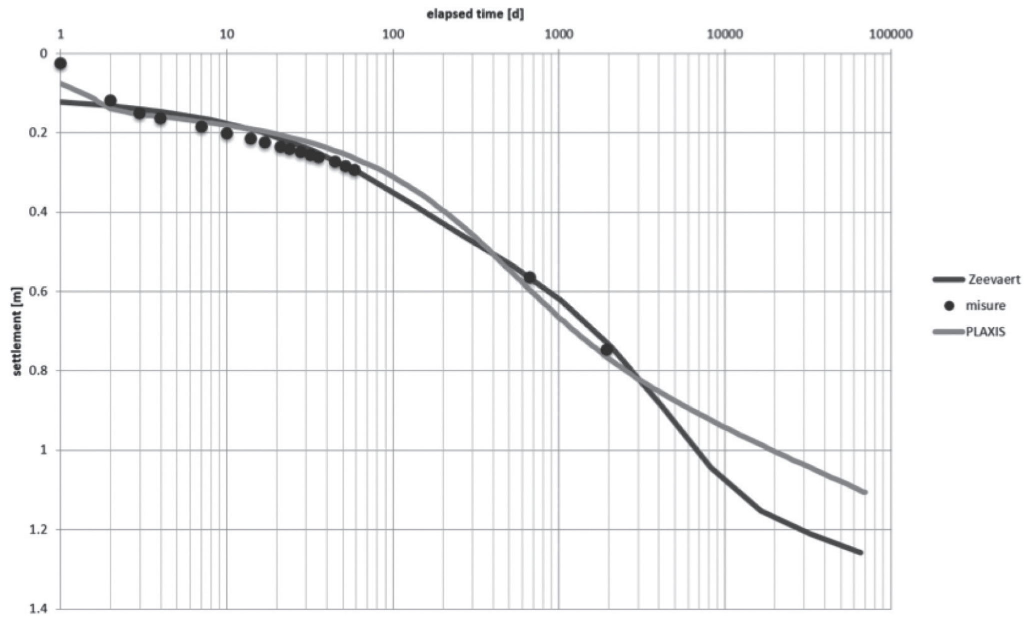

Fig. 13 – Results of Plaxis 2D model compared to 1D model results.
Fig. 13 – Confronto tra i risultati della terza analisi FEM e quelli del modello di ZEEVAERT (1972)

For the sake of completeness a third analysis performed by PLAXIS has been carried out. Even if the parameters have been deduced taking into account the available investigations, the performed analysis can be clearly considered a back analysis in which values of parameters have been selected in order to match the measurements. Figure 13 shows the results of such analysis compared with the same data reported in figure 12. In order to match the rapid evolution of settlement at the beginning of construction, a 0.5 m thick very compressible layer was added that simulates the lack in base preparation. Another important hypothesis concerns the permeability of soil. As a consequence of fibrous structure of peaty soils, a horizontal permeability ten times greater than vertical one was considered. Although matching of measured settlement can be considered satisfactory in both cases, the forecasted settlement at 50y differs by 20%.

4. Concluding Remarks

The paper presents a rare case history on levee and regards a full-scale trial embankment on very compressible peaty soil. The embankment was constructed in 2010 and measurement of settlement has been updated to December 2015. During this period the forecast of settlement deduced by former measurements has been compared with succeeding settlement. Finite element analyses provide underestimated forecast of settlement at the end of reference period for the embankment and the better interpretation of measurements have been reached by means of simple 1D model. In any case both 1D model and finite element 2D model seem quite sensible to small changes in parameters values and the set of data used for measurement interpretation are not coincident. The absence of pore pressure measurement represents a limit in the back analysis presented. The heterogeneity of subsoil and the difficulties in retrieving undisturbed samples suggest the use of trial embankment. The analysis carried out by both 1D model and 2D FEM model showed the necessity of taking into account the viscosity of subsoil. FEM analyses showed that effects of viscous deformation are present since the beginning of consolidation phenomenon, as a consequence the approaches based on uncoupling of hydrodynamic and viscous effects are not appropriate.

In the light of these results, the particular structure of peaty soil and its viscous properties require full-scale experiments in order to highlight the actual behaviour of soil.

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Importanza dei rilevati di prova nella progettazione degli argini

Sommario

L'articolo illustra un caso di studio riguardante il monitoraggio dei cedimenti di un rilevato sperimentale (in vera grandezza) per un periodo di 5 anni. I cedimenti misurati sono confrontati con quelli ricavati da una previsione di tipo A e una di tipo B. L'analisi dei risultati suggerisce l'importanza di una corretta modellazione dei fenomeni viscosi delle argille organiche. Le difficoltà che si incontrano nell'ottenere campioni indisturbati di buona qualità di argille organiche ed il comportamento viscoso, peculiare di questi terreni, evidenziano l'importanza della sperimentazione in vera grandezza.