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SOIL AND SITE IMPROVEMENTS OF THE MARSH SOILS IN PUERTO DE SANTA MARÍA HIGHWAY (CÁDIZ, SPAIN)

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ABSTRACT

The Puerto de Santa María and Puerto Real news highways are 17.4 kms. long, of which about 9 kms. run through a marshland area. The soft soil surveys involved piezocone reconnaissances and the construction of 3 experimental embankments. The embankements rest on ground improved with vertical drains for heights between 2.5 and 4.0 m., and with gravel columns for heights between 4.0 and 9.0 m. Expanded polystyrene was also used, for the first time in Spain, as a light fill in the embankments providing access to one of the bridges. The foundations for the 9 bridges located in the marshland were large diameter, precast, driven pier-piles (Raymond type). Embankment settlement was monitored and controlled by continuous settlement measuring lines. Pile driving was also PDA controlled.

KEYWORDS

Soil improvement, marsh, embankment, EPS, wick drains, vibroreplacement, pile driving, PDA, settlements

1. INTRODUCTION

The overall alignment of the new highway running around Puerto de Santa María and Puerto Real is 17.4 kms long. Approximately 9 kms. run over marshland fed in particular by the rivers Guadalete and San Pedro. It comprises Quaternary deposits of a fluvial nature (Fig. 1). The project involved the construction of sixteen bridges, of which nine are located in the marshland area.

The unconsolidated, low shear strength marshland soils in question necessarily called for a detailed study of the embankments to be supported on it, particularly at accesses to the bridges where the level unfailingly rises. As described in this paper, a detailed study was made on slope stability and settlement problems and, in general, deformations potentially excessive on the short and long term for road infrastructures and the actual bridges themselves.

2.- GEOTECHNICAL INVESTIGATIONS

Due to the effect of the soft soil's behaviour on cost, construction and operation of the future expressway, it was decided to carry out an extensive reconnaissance campaign, when work commenced, as described hereafter. In all, 23

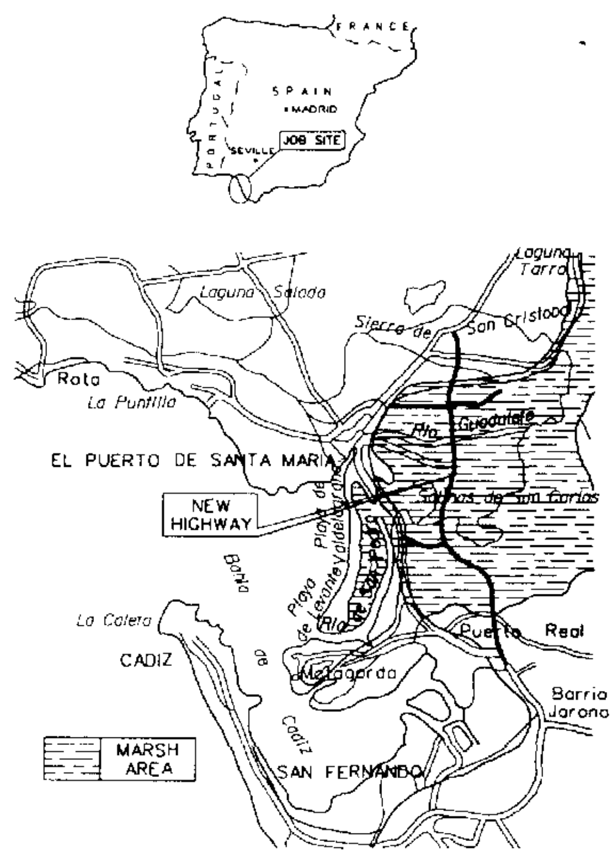

Fig. 1 Location Plan

mechanical boreholes were drilled in this phase, with continuous core extraction and undisturbed sample taking. A total of 81 SPT tests were performed inside the boreholes and, in all, 62 vane tests were carried out. A total of 29 continuous dynamic Borros penetration tests were also carried out.

As a fundamental part of the reconnaissance, the CEDEX Geotechnics Laboratory carried out a total of 20 piezocones when work commenced, making a total of 51 for the overall site work. Pore water pressure dissipation tests were also made to work out the ground's coefficient of consolidation (Fig. 2).

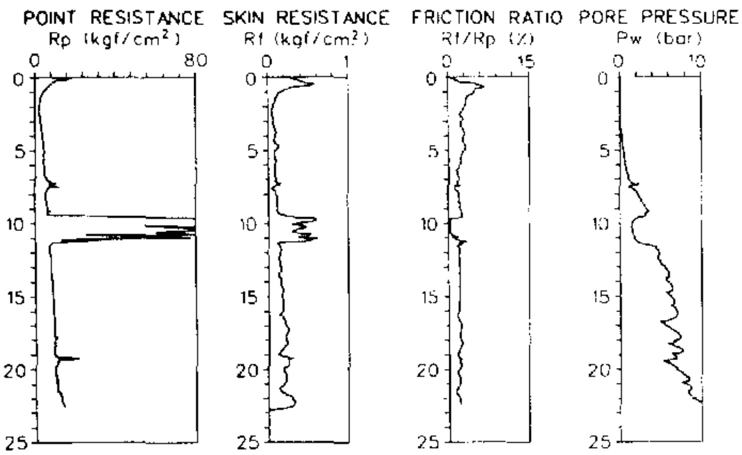

Fig. 2 Piezocone Log

Carrying out piezocone tests enabled the condition of the soils forming the marshland to be known and determined. It was thus possible to distinguish the clayey soils from the sandier soils, bearing in mind the relations between the point resistance and the friction ratio as proposed by Campanella and Robertson (1988), the relative sleeve resistances and dissipation tests. Undrained shear strength was determined from the point resistance on the basis of the correlation as proposed by Baligh (1975).

Three full scale experimental test embankments were also built. The purpose was to use instruments and auscultation to determine the ground's behaviour parameters under the effect of the embankment loads, as well as to compare the analysis made of them with theoretical models. The embankments were built at three points close to the body of the future highways. The experimental embankments were 14.50 x 26 m. in plan, with 2(H):1(V) slopes and a maximum height of 4 metres. Instruments were placed along three continuous settlement lines and an inclinometer at the foot of their slopes.

Figure 3 shows the results obtained from the auscultation of one of the experimental embankments. Building commenced in January, 1994 with an initial height of approximately 2 m. Settlement was auscultated and analyzed periodically during this first phase and once stabilized, they were built up to a height of 4 m. in a second phase.

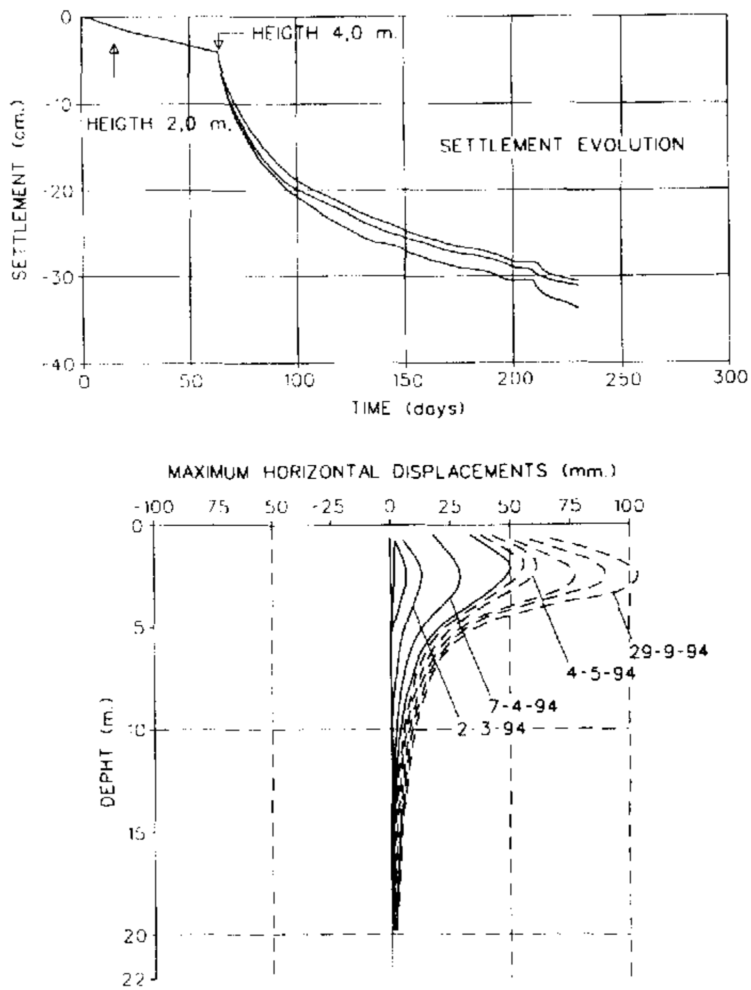

Fig. 3 Behaviour of the embankment test No. 2

3. GEOTECHNICAL CHARACTERISTICS OF THE MARSHLAND SOILS

From the beginning of the new road to approximately 2.5 kms. further on, the route runs through Tertiary land consisting of marls, calcarenites and conglomerates displaying a marked topographic relief forming the *Sierra de San Cristobal* (124 m. in height). Similar materials reappear at the end of the route.

After the 2.5 Km. point, the highway crosses some 3 kms. of fluvial type Quaternary land (flood silts) to then run for approximately 7 kms. through the marshland area formed by the rivers Guadalete and San Pedro in the vicinity of the Bay of Cadiz. Frequent flooding occurs after rainy periods, particularly to the south of the River San Pedro where the marshland reaches as much as 35 m. in depth.

Three types of soil have been basically distinguished in the marshland's group of deposits: a desiccated surface crust varying in thickness between approximately 0.60 m and 2.0 m., very soft to soft marshland clay and very loose to loose marshland sand. Loose sand levels are interspersed with clay composition marshland mud reaching thicknesses from a few centimetres to

10 m. The piezocone tests enabled sandy materials to be differentiated from clayey materials so it was possible for the draining effect of the sand layers to be included in the marshland soil consolidation analyses.

Figure 4 shows the geotechnical lithological profile of bridge no. 8 on the Puerto de Santa Maria Highway and table 1 summarises some of the average values of the geotechnical properties of the materials existing in this area.

TABLE NO. 1 AVERAGE GEOTECHNICAL PROPERTIES OF THE MATERIALS

Description	Depth (m)	Water content (%)	Dry bulk density (Kg/m^3)	Fine content no. 200 ASTM (%)	Undrained Shear Strenght Su (KPa)	Piezocone Point Resistance q_c (Mpa)
Dessicated Crust (Q)	0.0-1.6	20-35	1420	14-90	30-60	0.60
Very sorf marsh clays (Q)	1.6-8.0	40-45	1250	85-95	10-15	0.25
Loose marsh sands (Q)	8.0-13.0	30-40	1370	15-40		2.50
Soft marsh sands (Q)	13.0-28.0	12-25	1520	76-96	40-50	0.30-0.60
Hard clays with dense sand layers (P)	28.0-34.0	12-25	1520	40-90	100-300	7.0-12.0
Dense Sands and Gravels (P)	>34			20-40		>22.0

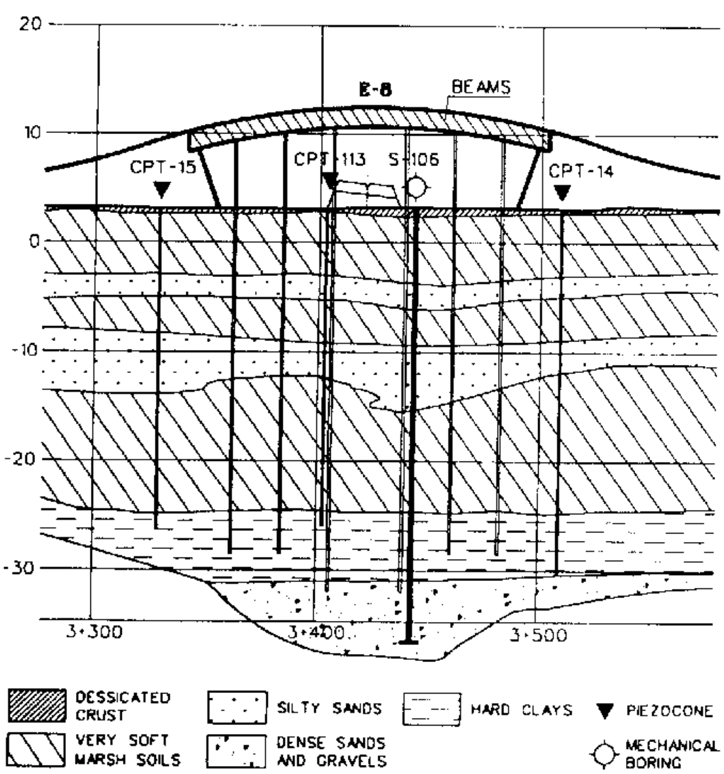

Fig. 4 Characteristic stratigraphical profile

4.- EMBANKMENT FOUNDATIONS

The bridge solution solves the problems as raised by soft soils for the highway's infrastructure, but it has a heavy financial repercussion since the whole of their foundations must be piled (the substratum depth varies between 25 and 40 m.).

This is why the initial length of the bridges was reconsidered and the maximum height at abutments was increased, providing for treatment of the ground under the access embankments.

Considering 2(H):1(V) slopes in the embankments providing access to the bridges, estimative stability calculations were made and it was found that a sufficient short term degree of safety could not be relied on.

Moreover, a settlement calculation indicated that a long term could be expected to elapse from when the stability condition allowed the embankments to be finished until the proportion of residual settlements were sufficiently low (less than 10-15 cms.) for the carpet infrastructures to then be built, as well as the pilings of the bridges closest to the embankments (because of the lateral pressures the latter might exert on them).

The presence of marshland deposits therefore involved having to reinforce the ground and accelerate the consolidation process.

Two types of treatment were carried out:

a) Band shaped prefabricated drains (wick drains or geodrain)

b) Stone columns made by vibroreplacement

The former was used for embankment heights where, by accelerating the primary consolidation of the marshland soils, sufficient undrained strength was achieved for the embankment to become stable and post-construction settlement to be admissible. Stone column treatment was used for greater embankment heights where reinforcement is necessary, apart from accelerating soil consolidation (Fig. 5).

In one case (bridge E-6), the use of a light fill was considered as a solution. Very light expanded polystyrene (EPS 25 kg/m3 density) was used as a fill material, instead of earth, for building the abutment access. In essence, this drastically reduced the loads to be supported by the soft soil and, consequently, the advantage lies in no longer having to provide ground improvement treatment except in very specific areas (Fig. 6). In this case, treatment was only carried out using stone columns under the bridge abutment foundation area.

It was considered that a geotextile should be placed for embankment heights under 2.5 m., once a minimum amount of clearing of the surface on which embankment building work was to commence had been carried out. It was not advisable to remove all the layer knitted together by roots since such layer acted as ground reinforcement and distributed the loads over the ground on which to begin building the embankments.

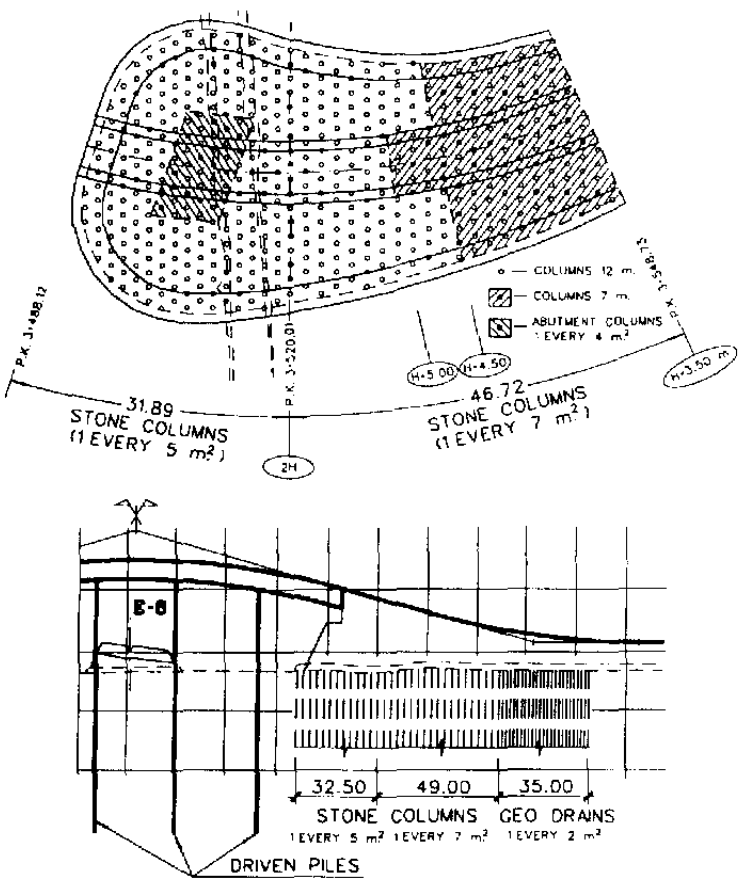

Fig. 5 Soil Improvement at Structure E-8

A triangular mesh geodrain was made every 2 m^2 for heights above 2.5 m and below 3.5-4.5 m. (according to each case).

Stone columns were built for heights above 3.5-4.5 m. Two different layouts were established in a triangular mesh: a) the first with a density of 1 stone column every 5 m^2 in the area close to the abutments where the embankment height is generally greater and where, should the abutment be founded on the embankment, the load on the embankment's foundations is also higher and, b) the second, with a density of 1 column every 7 m^2 in an area 15 m. from the abutment in the direction where the embankment's height diminishes (Fig. 5).

The drain mesh was dimensioned on the basis of Barron's radial consolidation theories (1948), adapted according to Hansbo's formulation (1981).

The experience gained by the CEDEX Geotechnics Laboratory in this type of work was used to design the stone column mesh. The Priebe method (1978) which establishes the proportion of load the columns take as a function of the basic treatment parameters (area ratios, etc.) was used for checking and is applicable to columns in sufficiently resistant ground. The column diameter was estimated at 1.0 m. as a function of the ground's shear strength (Oteo, 1991). This diameter proved to be in the order of that obtained during the building of the columns.

When the embankment was built in two phases, it was assumed that the increase in effective pressure due to the degree of consolidation reached meant an increase in the undrained shear strength equal to 0.22 times the effective pressure increasing.

As far as the embankment slope stability is concerned, a simplified calculation was made in these initial assesments based on turning the short term soft soil strength (at the relevant calculation time) and the columns into an overall treated ground as a function of the area ratio (A/As) and the relevant improvement factor (Priebe). The short term shear strength for each stage was used in the case of drains and columns whithin the upper softer layer

The stone column treatment was completed by placing a top layer of gravel about 30-40 cms. thick which was supplementarily reinforced with a geotextile acting as a column head "capping", distributing the embankment loads and contributing to making deformations uniform, as well as to increasing lateral stiffness because of the geotextile's tensile strength. In the case of the drains, a 0.50 m. layer of selected material was placed, with a geotextile.

In all, 95,000 linear metres of stone columns, with an average of 0.9 m^3/m (about 160 m/day) and 300,000 linear metres of geodrain (2,000 m/day) were built.

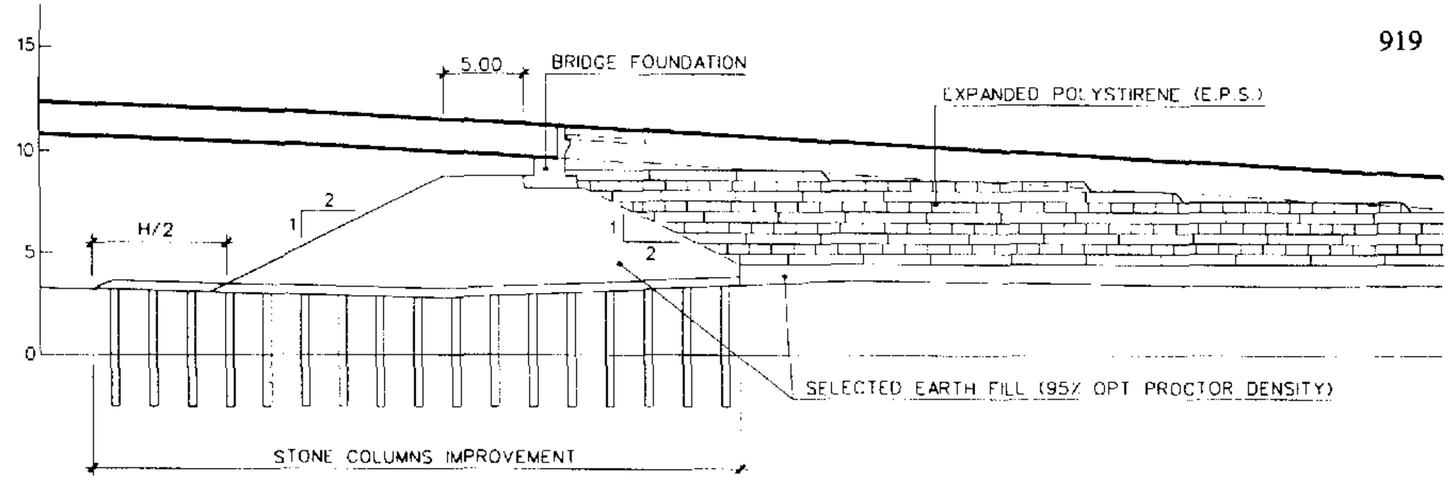

Fig. 6 EPS Embankment at Structure E-6

5.- DEEP FOUNDATIONS

In view of the resistant substratum's very low bearing capacity and the deep depth down to which it reaches, it was considered that the most suitable type of pile is the Raymond pile, used as a pier-pile. These are large diameter (1.37 m.), precast, pre-stressed concrete piles with a ring shaped cross section and 12.5 cm. wall thickness. *In situ* concrete piles were only used when the resistant substratum was shallow (less than 10.0 m.).

This type of pile displays great advantages over *in situ* cast piles and over small diameter, precast concrete piles: a) their high quality, due to pre-casting, b) not using permanent casing (very soft soils), c) control of bearing capacity during driving, d) use of the pile itself as a pier, etc.

The Pile Dynamics Analyzer P.D.A. system was used during driving and control was completed with restrike control. In many cases, due to the ground's tixotropy and liquefaction during driving, the coefficient of safety in driving was low. But it was seen in redriving that, in the mid term, the shaft resistance gave suitable bearing capacity (Fig. 7).

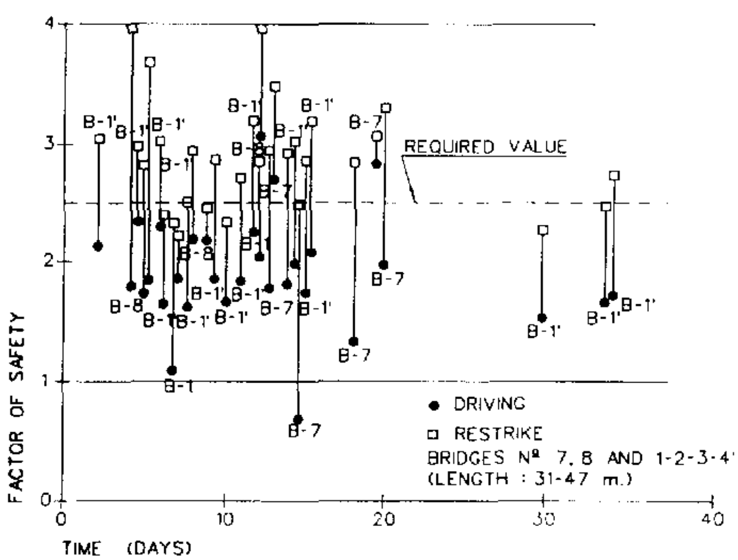

Fig. 7 Relationship between the deduced safety factor and the time delayed for the restrike.

6. RESULTS OBTAINED

Figure 8 shows the settlement forecast made before work commenced, on the basis of the experimental embankment data

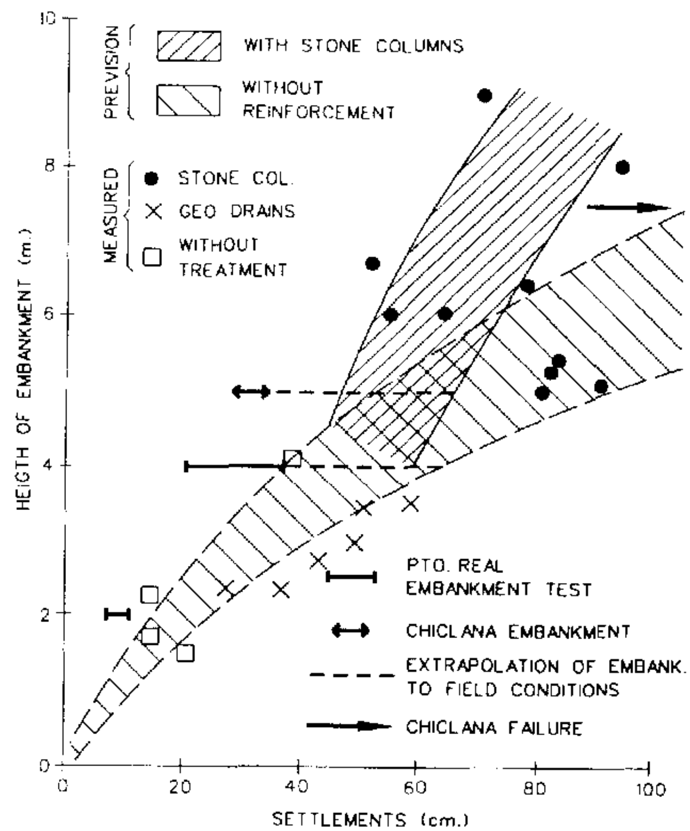

Fig. 8 Comparison between theorethical and measured settlements

and of some nearby site and of the results of experimental geotechnical tests. Settlement measured after the work was performed, can also be seen in that figure. As can be observed, despite the dispersions existing, the settlement forecast and that obtained is similar in order of magnitude, and this justifies the treatment given. Despite the gravel columns, more than 80 cms. of settlement were obtained in some embankments, though within the time periods provided for and with no stability problems.

7. CONCLUSIONS

A) The use of techniques like the piezocone and experimental embankments gave magnificent results in the marshland's very soft soil.

B) The treatments given (the higher the embankment, the greater the intensity) proved to be suitable, in view of the settlement obtained and the work time involved.

C) In this type of work, good technical reconnaissance and continuous control of the treatment's execution and embankment movements are indispensable.

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