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Behavior of the Underpinning of a Building in Venice

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ABSTRACT

This paper examines the behavior of a building constructed in 1960 and subject ever since the end of its construction to considerable total and differential settlements in time. Monitoring of the building revealed an increase in the settlement rate during the first years of the 1990s, such as to require underpinning with micropiles in 1992.
In the first part of the paper, the evolution of settlements until the beginning of the restoration is analyzed, and real and anticipated behaviors are compared. In the second part, the behavior of the building during and after micropile underpinning is explained.
Back-analysis was carried out, adopting tridimensional finite element analyses, in order to interpret behavior before and during underpinning.

KEYWORDS

Finite element method, hyperbolic method, micropile, settlement, soil-structure interaction.

INTRODUCTION

This paper examines the behavior of an office building (Block A) owned at the present time by ENEL (Italian electricity board), from its building in 1960 to its underpinning with micropiles in 1992.
Ever since its construction, the building, resting originally on a rigid slab along the Rio Novo canal in Venice, had been subject to considerable total and differential settlements, due to rather soft foundation soil and to interactions with adjoining buildings (maximum settlement of 100 mm and rigid rotation of 1/150). Accurate monitoring of movements, carried out from 1960, revealed that the rate of settlement increased in the 1990s, with scour of the adjoining "fondamenta" (retaining wall with pavement) of Rio Novo. Therefore, underpinning with micropiles grouted at high pressure was required in 1992. Monitoring of movements during the various stages of micropile underpinning was carried out and continued until 1996.
First, the paper examines the causes of settlement of the original foundations and shows the behavior of the building from 1960 to 1992. Second, the behavior of the underpinned foundations is analyzed; a transfer of load to the micropiles with settlements of the building varying from 10.0 to 20.0 mm is recorded.
Back-analysis was performed in order to interpret the behavior during the whole life of the structure with tri-dimensional finite element analyses. Settlements calculated in this way were then compared with those effectively measured.

DESCRIPTION OF STRUCTURES

The office building consists of three blocks (Fig. 1). The first (Block A) is a construction 37.00 m long and from 12.00 to 16.0 m wide, and is composed of a basement and four storeys. The foundation slab, at -2.0 m a.s.l., is 0.50 thick and rests on a group of 5.0 m wooden piles, only along the external edge of the slab. Concrete shear walls strengthen the foundation structure in the basement and reach the ground floor at +2.2 (Figs. 2 and 3). The ground level along the side of the Rio Novo canal, about 6.0 m from Block A, is at about +1.40.
Behind Block A, the two other Blocks, B and C, have caps on 18.0-m bored piles of 0.35 m diameter.
The live and dead loads of Block A are about 41 MN in total, including the weight (about 3 MN) of the five water tanks for cooling systems, located in the basement, so that the soil pressure is 76.6 kPa on average.

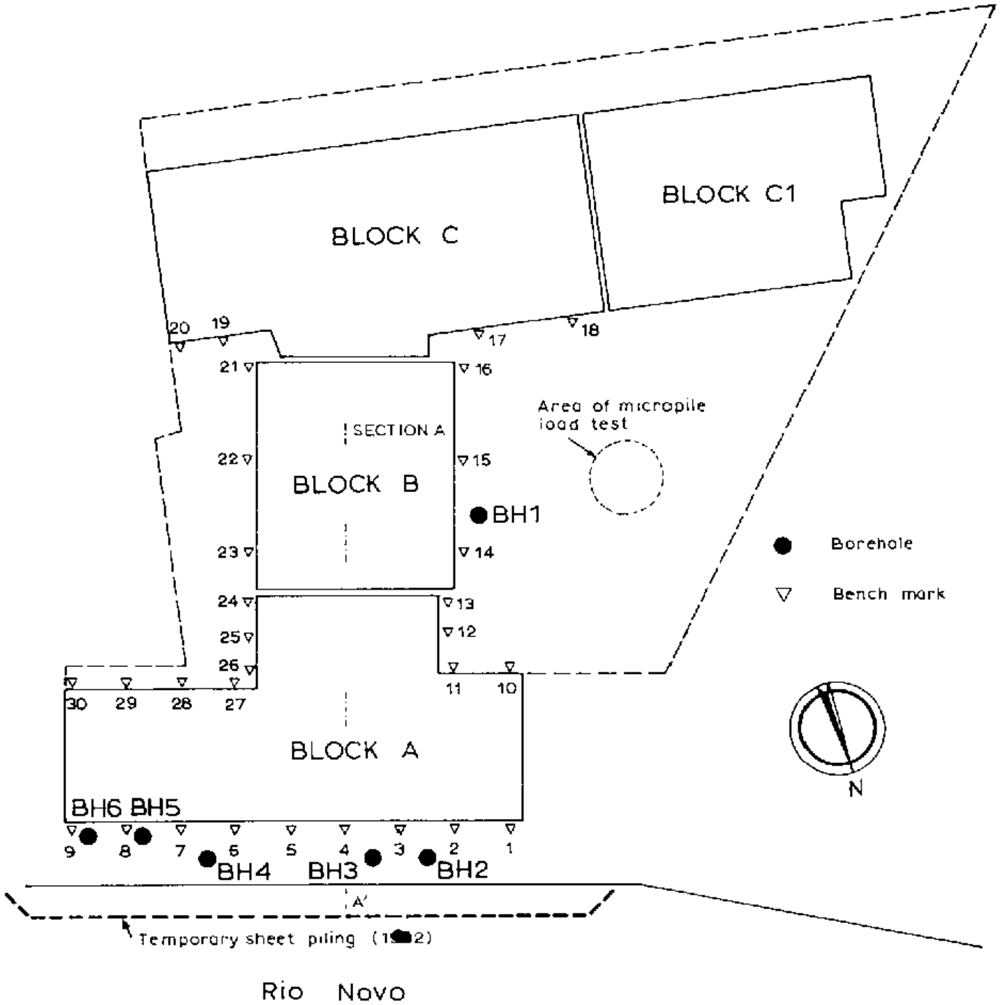

Fig. 1 Plan of office building: locations of bench marks and boreholes.

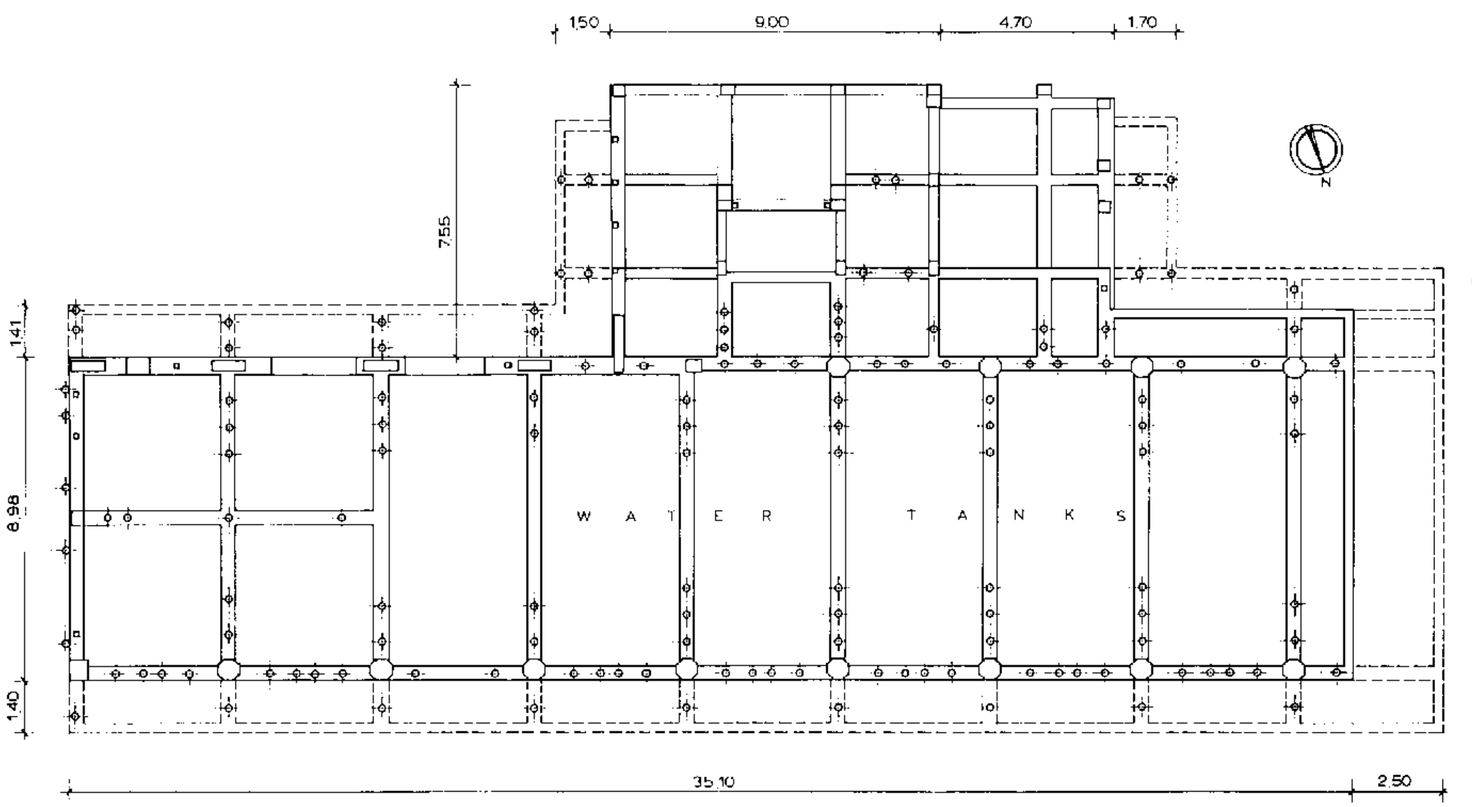

Fig. 2 Plan of concrete slab and positions of micropiles.

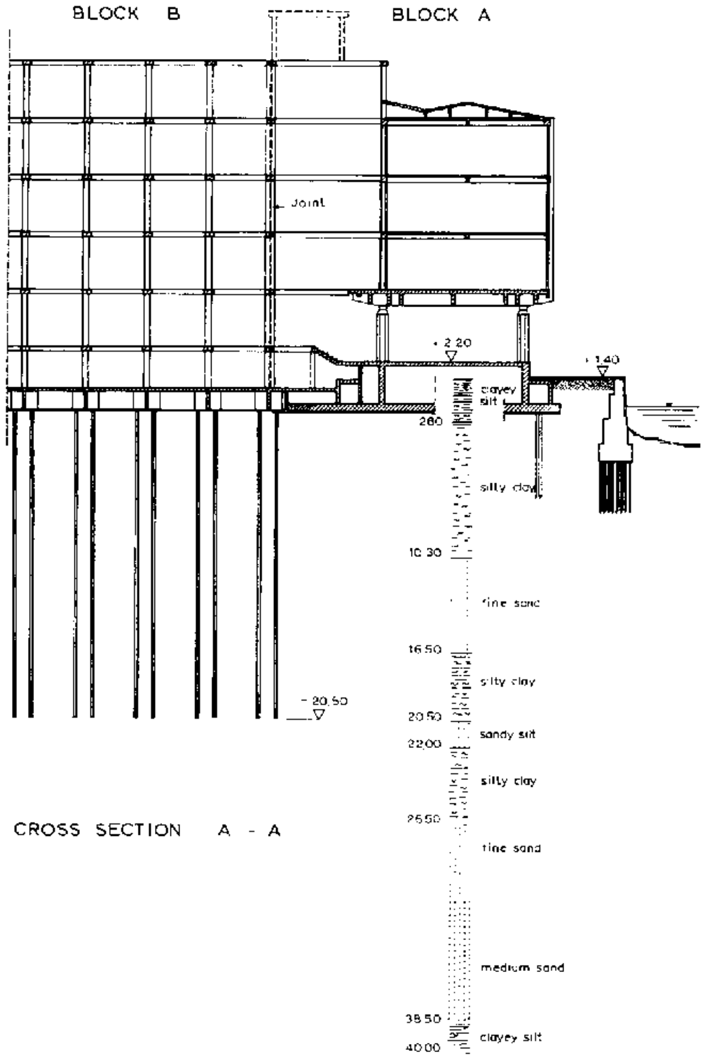

Fig. 3 Cross-section of Blocks A and B and stratigraphy.

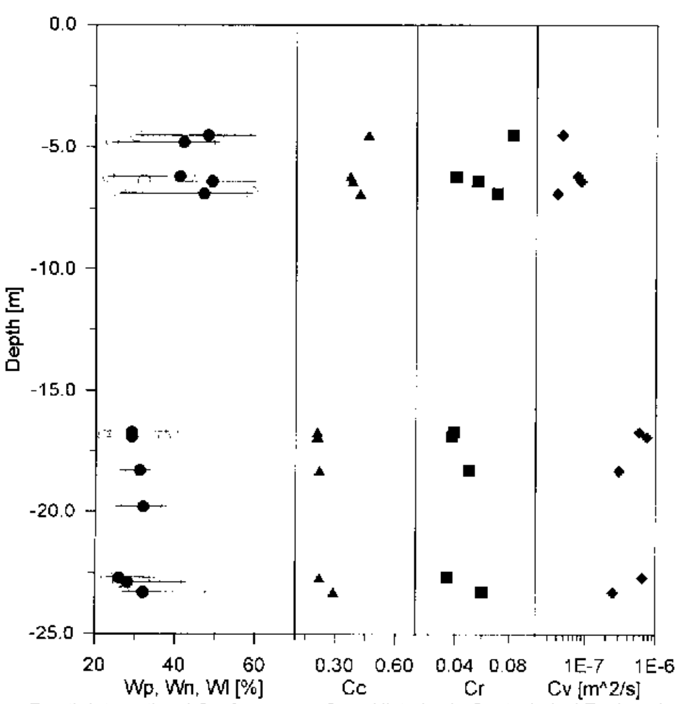

Fig. 4 Properties of cohesive layers.

SOIL CHARACTERISTICS

Before the restoration of Block A, an investigation comprising six boreholes and some SPT tests was carried out on the site (Fig. 1). During boring, several samples were collected and then tested in the laboratory. Some properties obtained from classification tests and oedometric tests are reported in Fig. 4. The stratigraphic profile of soil was homogenous in the area and was made up as follows (Fig. 3):

- from +1.4 to about -3.0 m a.s.l.: prevalence of medium-fine sand (N_{SPT}=4÷14) with layers of clayey silt (pen < 25 kPa);
- from about -3.0 to -7.5: very soft gray silty clay (pen = 20÷50 kPa; torvane = 15÷25 kPa;). Unconfined tests and consolidated undrained isotropic triaxial tests carried out in the laboratory gave q_u values of 33÷47 kPa, ϕ = 26°÷30° and c'=5÷15 kPa;
- from -7.5 to about -10.0: prevalence of soft clayey silt or silty clay (pen < 25 kPa; torvane = 10÷20 kPa);
- from about -10.0 to -16.5: medium and fine sand, partially silty in some areas. The upper part, about 1.5 m thick, consists of loose sand (N_{SPT} = 3÷6), while the remaining part is sand of medium density (N_{SPT} = 21÷28);
- from -16.5 to -26.5: medium to stiff clayey and silty soils (pen = 80÷170 kPa; torvane = 35÷85 kPa). Unconfined compression tests gave a q_u of 73÷148 kPa;
- from -26.5 to -29.0: a mainly cohesionless transition layer, which becomes a thick layer of rather dense medium-fine sand down to -38.5 (N_{SPT}=35÷40);
- from this depth until -40.0: fairly stiff clayey and silty soils (pen = 175÷200 kPa; torvane = 50÷90 kPa).

ANALYSIS OF SETTLEMENTS BEFORE UNDERPINNING

Block A had been monitored since the end of construction in 1960 by means of periodic leveling of bench marks placed principally along the canal (BM 1 - BM 11). Settlement and tilting began to take place immediately and, in 1966, the maximum settlement was 54.2 mm at bench mark 9 (Fig. 5). In this year other bench marks were placed in each block, in order to determine the behavior of the whole office building.
Until October 7 1992 (last leveling before the beginning of underpinning), the bench marks showed maximum settlements of 93.2 mm (BM 9) and 65.2 (BM 1) along the external longitudinal side of the slab and an angular distortion of 1/150 (BM 11 and BM 3) transversally (Fig. 5); in fact the slab rested on the pile caps of Block B at their point of juncture along the internal longitudinal side.
Figure 5 identifies three different time behaviors of the structure. In the first one, from 1960 to about 1966, building movement was due to consolidation settlement of the shallow clayey layers of high compressibility (Cc = 0.37 ÷ 0.46). Thereafter, there was a second period in which the settlement principally affected the edge along the canal with a settlement rate, as shown by BM 1 and BM 9, of 2 mm/year from 1966 to

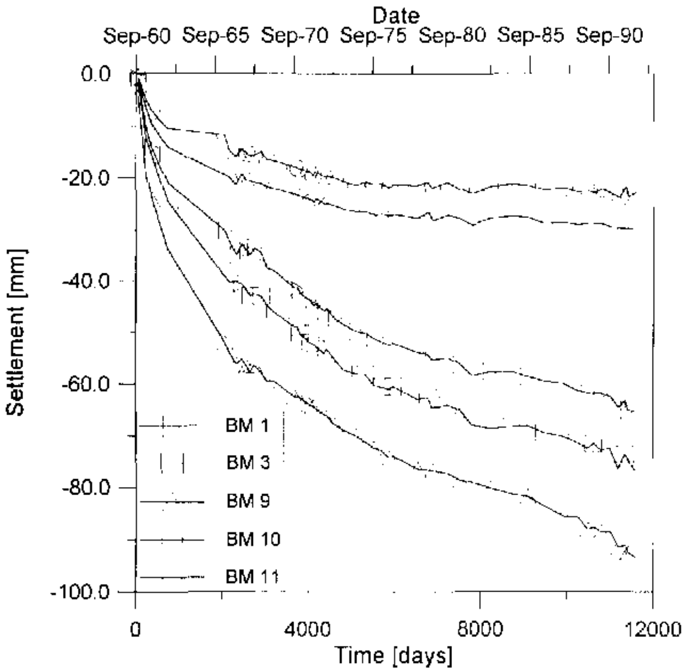

Fig. 5 Settlement vs. time: behavior of Block A from 11/1960 to 7/1992.

1976 and about 1 mm/year from 1977 to 1989. This was due to secondary compression and degradation of the retaining wall along Rio Novo, as a consequence of scour due to passing boats (Fig. 6).

Finally, from 1989 until 1992, degradation of the retaining wall along Rio Novo became considerable and settlement increased to 1.5 and 2.25 mm/year respectively for BM1 and BM9. Consequently, Rio Novo had to be closed and temporary sheet piling was built for protection of the retaining wall (Figs. 6 and 7).

UNDERPINNING

In order to stop the increasing settlement, in 1992 underpinning was carried out with "Tubfix" micropiles, grouted at high pressure, with a hole diameter of 130 mm and steel tube reinforcement 12 mm thick with an external diameter of 88.9 mm (Fig. 8).

The micropiles, about 18.0 long, start at +2.2 (ground level) and go to a depth of -16.0 in the layer of dense sand and, for about 3.0 m, cross the concrete sheet walls of the building basement. The micropiles have three non-return valves per meter and were grouted at high-pressure from the tip of each micropile up to -7.50; grout volume was three times borehole volume. A total of 137 micropiles was emplaced (Fig. 2), with an available design pile capacity of 0.3 MN. The loading test on one micropile, in an area near Block B, under loads of 0.3 MN (working load) and 0.45 MN, gave total settlements of 2.5 and 4.6 mm respectively, and residual settlement on completely unloading the pile of about 0.6 mm.

In order to avoid outflow of grout before the execution of the micropiles, the space between the steel sheet piling and the side wall was filled with sand (about 200 m^3) for a length of

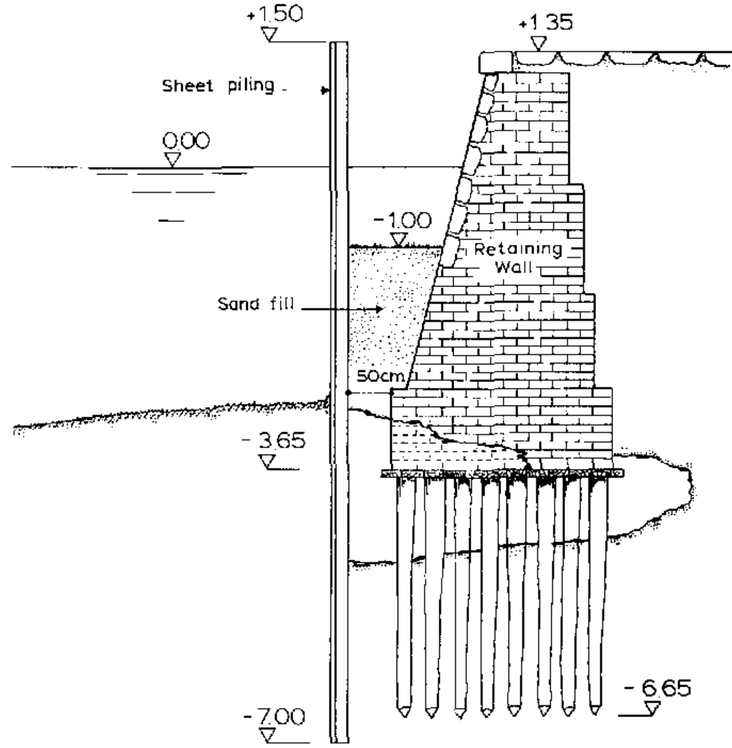

Fig. 6 Temporary strengthening of retaining wall by sheet piling.

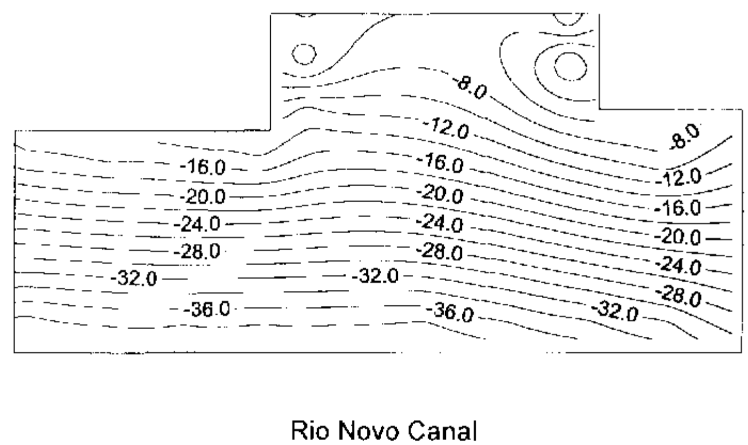

Fig. 7 Contour of Block A settlement from 10/1966 to 7/1992 (mm).

60.0 m (Figs. 1 and 6).

Micropile construction proceeded in four stages: drilling of the concrete sheet wall, slab and soil, with emplacement of steel reinforcement and filling with grout; injection of grout at high pressure through 25 valves via a plug; bonding of micropiles to the concrete structure; injection of grout through three valves placed in the first meter of the micropile under the slab.

BEHAVIOR OF BUILDINGS DURING EXECUTION OF MICROPILES

During and after underpinning, monitoring of the structure was

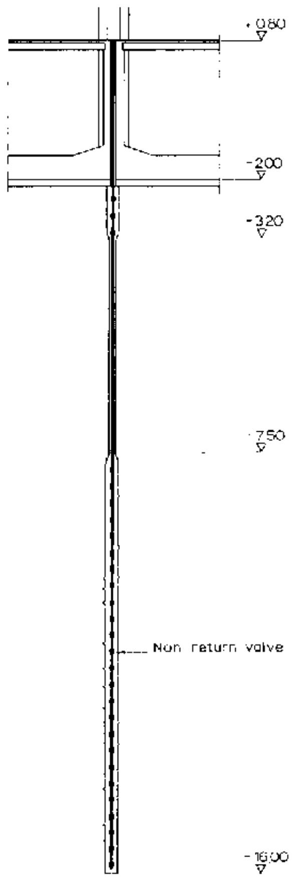

Fig. 8 Detail of micropile.

carried out.
Figures 9 and 10 show that the settlement of Blocks B and C was 2.0÷3.0 mm, so that the restoration of the existing foundation did not affect these two blocks. Instead, Block A showed considerable settlement. In fact, before the beginning of work, settlements of about 4.0 mm along the canal side and 2.0 mm internally occurred, due to sand filling (Figs. 11 and 12). During underpinning, settlements of up to 18.0 mm were recorded with the greater movements along the Rio Novo side. Moreover, the building behaved differently according to the position of the micropiles actually working. Monitoring showed that, during the first stage with partial construction of micropiles only in the central area of the building, the block tended to rotate rigidly around a diagonal (Fig. 13), while later, when the group of micropiles was completely in place, it had more uniform behavior (Fig. 14). In a short time, the settlement of Block A, measured up to the present moment, ceased (Fig. 15). The settlement of Block A, during underpinning, can be ascribed to soil disturbance effects during construction of micropiles and transfer of load from slab to micropiles.

METHODS OF ANALYSIS

In order to interpret and simulate the behavior of Block A, tridimensional analyses were carried out in which the interactions among raft, surrounding soil and, if present,

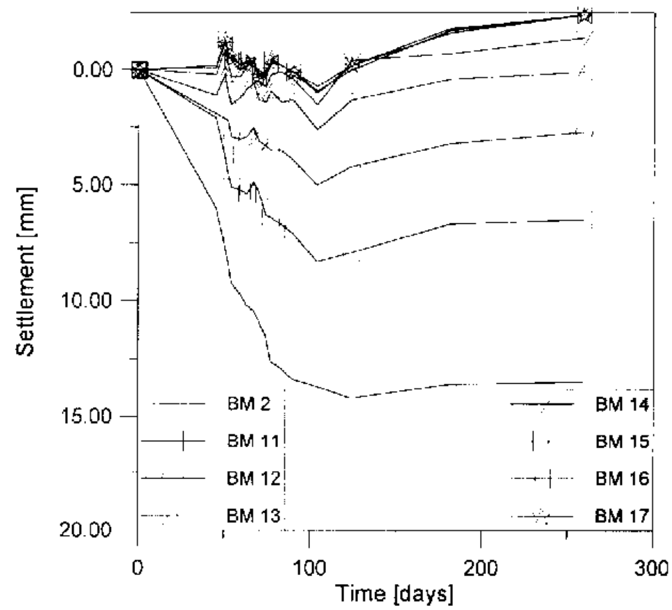

Fig. 9 Settlement vs time in Blocks A, B and C from beginning of execution of micropiles; east side.

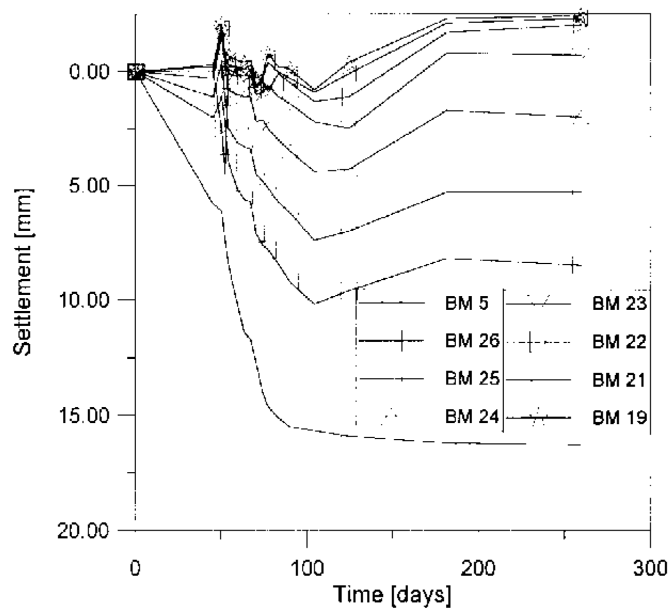

Fig. 10 Settlement vs time in Blocks A, B and C from beginning of execution of micropiles; west side.

micropiles were taken into account, simulating the areas underpinned with micropiles by means of the equivalent pier method.
A general purpose program (ABAQUS V.5.6) was adopted in which various constitutive laws are implemented to simulate soil behavior. In the analyses, the Drucker-Prager elasto-plastic law for cohesionless soils and Modified Cam Clay law for clayey soils were used. The slab was simulated by means of tridimensional elastic elements, whereas the external parts of the mesh were schematized by means of infinite elements following a linear elastic law. 4760 elements and 18427 nodes were used in the analyses.
Modified Cam Clay model parameters and the friction angle value for cohesionless soils were mainly derived from the

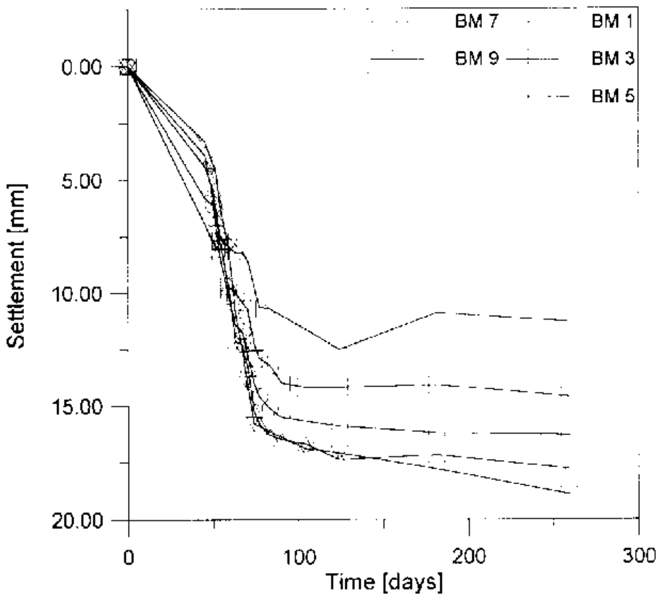

Fig. 11 Settlement vs time of bench marks along Rio Novo canal, from beginning of execution of micropiles.

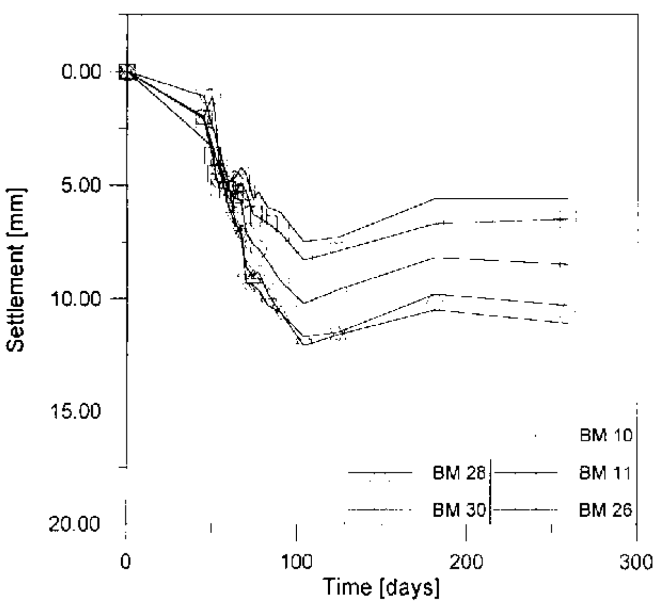

Fig. 12 Settlement vs time of inner bench marks of Block A, from beginning of execution of micropiles.

results of laboratory tests. The elastic moduli of sandy or silty-sandy soils were estimated using an empirical correlation linking them to the N_{SPT} values.

The equivalent pier method was used in the analyses in which the presence of micropiles was considered. Poulos (1993) proposed elastic behavior for the areas directly influenced by the group of micropiles. The equivalent modulus was determined by means of the following equation:

$$E_e = E_p \frac{A_p}{A_G} + E_s \left(1 - \frac{A_p}{A_G}\right) \quad (1)$$

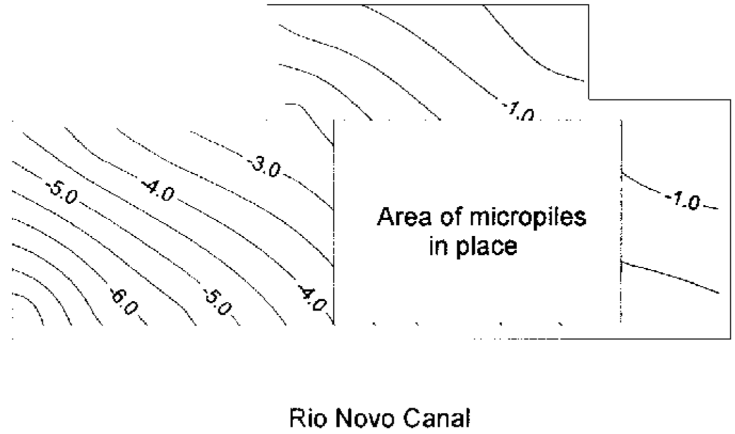

Fig. 13 Contour of Block A settlement in first stage (mm).

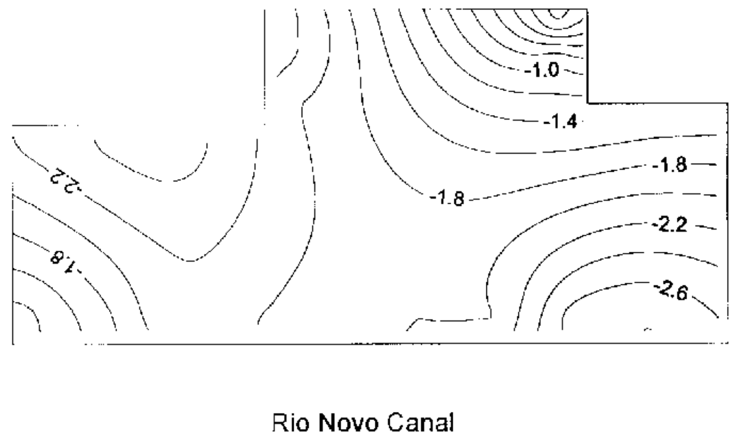

Fig. 14 Contour of Block A settlement in second stage (mm).

where E_p is Young's modulus of piles, E_s Young's average modulus of soil within the area influenced by the group of micropiles, A_p the total cross-sectional area of the group of micropiles, and A_G the plan area of the micropile group.

Table 1 lists the values of the parameters used in the analyses regarding both various soil layers and soil modified by the presence of micropiles. Different parameters were used for layer 2, according to whether the situation at the end of construction or immediately before underpinning was considered.

Table 1: Soil foundation parameters of F.E.M. analysis.

Layer	κ	λ	e_0	M/φ	E [kPa]	OCR
1					20000	
2	0.026	0.174	1.25	1.07		1.3
3				38°	75000	
4	0.015	0.091	0.8	1.07		1.8
5				34°	30000	
6	0.016	0.096	0.88	1.07		1.5
7				36°	70000	
micropiles					700000	

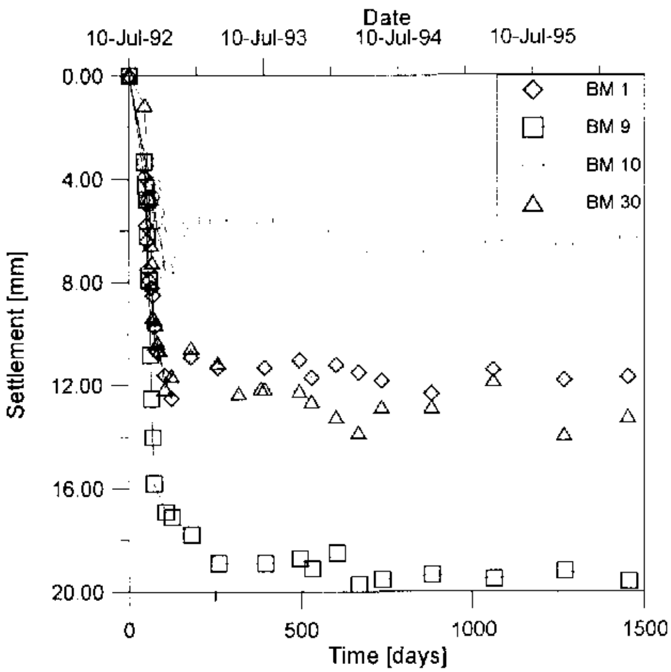

Fig. 15 Settlement vs time of four outermost bench marks, from beginning of works until the present.

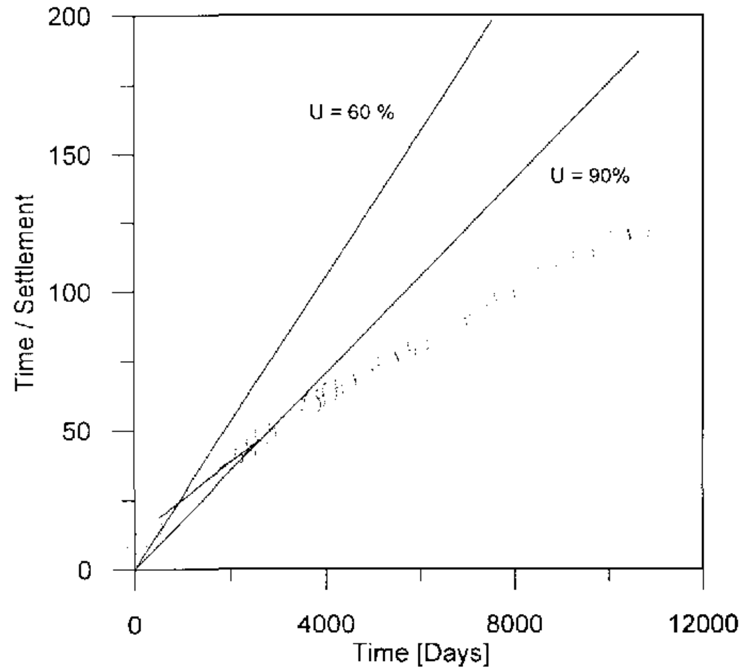

Fig. 16 Hyperbolic plot of settlements of bench mark 9.

ANALYSIS OF RESULTS

The rectangular hyperbola method provides a simple approach to predict the magnitude of primary compression, using early field settlement data (Sridharan and Rao 1981, Sridharan *et al.* 1987, Tan and Chew 1996), with an error of about 10% between predicted and observed values. Plotting field settlement data concerning measurements between 1960 and 1992 in the form time/settlement versus time, a linear segment between the 60% and 90% consolidation state was recorded. Figure 16 shows the data relating to bench mark 9. A 90% consolidation settlement was reached after about 2600 days (about 7 years). Thereafter, there was a stage of secondary compression for about 1200 days (about three years), after which external action modified the ratio significantly. Considering the slope of the first linear segment, the final primary settlement was about 65 mm. Tri-dimensional analysis carried out without the presence of micropiles and using the data of table 1 gave a settlement value for bench mark 9 of 67.3 mm. In the cases of bench marks 1 and 10, with the hyperbolic method settlement values of 37.5 and 16.0 mm were obtained, whereas numerical analyses gave 58.0 and 19.0 mm respectively. The difference between measured and calculated settlement was probably due to dishomogeneous soil layers and/or to a small difference in the real distribution of loads with respect to that hypothesized. However, the results show that the settlement of the structure without external action could be considered acceptable.

The behavior of Block A during restoration was also simulated with numerical analyses. Along the side of the canal, measured settlements were 4.0÷5.0 mm after sand filling and 14÷16 mm, with a maximum value of 19.3 mm, at the end of work.

At first, in the numerical analysis only the sand fill (200 m^3) along the retaining wall was considered and simulated by means of a pressure of 10 kPa distributed uniformly on a strip 60.0 m long and 3.0 m wide. Later, also pressure Q of 76.6 kPa (inclusive of the load of the five water tanks and buoyancy) was taken into account. This pressure is transmitted from the underpinning micropiles to the deep clay layer (-16.0 to -26.0 from ground level), taking into account the restraint on Block B.

Settlement calculated with finite element analyses due to sandfill was about 2.1 mm, whereas that measured was 4.0÷5.0 mm.

Total settlements along Rio Novo were about 30.0 mm, greater than those measured (Fig. 17a-d). This probably occurred because load diffusion through the micropiles, in relation to their real arrangement, was excessively limited and/or the live load of the building was overestimated. However, tri-dimensional analysis simulates the overall behavior of the building as regards tilting along its longitudinal and transversal axes. In fact, the trend of differential settlements can be detected to a sufficiently precise degree, especially along the sides parallel to the canal.

CONCLUSIONS

The behavior of an office building, monitored for more than thirty years and subjected to considerable total and differential settlements, such as to require underpinning with micropiles, is examined.

Settlement data before restoration show larger deformation of the structure than that which could be anticipated, due to degradation of the retaining wall along Rio Novo.

Although underpinning caused non-negligible settlements (10 ÷ 20 mm) during work, these stopped the movements.
Finite element analyses, carried out in various situations, simulated quite well the tridimensional behavior of the building, but evaluated settlement values approximately.

ACKNOWLEDGEMENTS

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REFERENCES

Colombo, P. [1987]. Aspetti geotecnici ed idraulici nella laguna di Venezia. Proc. of 6th National Congress of Geologist, Fondazione Cini, Venezia, pp. 227-238 (in Italian).

Hibbit, Karlsson & Sorensen [1996]. ABAQUS Version 5.6.

Koreck, H.-W. [1978]. Small diameter bored injection piles. Ground Engineering, May, pp. 14-20.

Makarchian, M. and H.G. Poulos [1994]. Underpinning by piles: A numerical study. Proc. of 13th Inter. Conf. on Soil Mechanics and Foundation Engineering, New Delhi, pp. 1467-1470.

Poulos, H.G. [1993]. Settlement prediction for bored pile groups. Deep Foundations on Bored and Auger Piles. Van Impe (ed.), Balkema, Rotterdam, pp. 103-117.

Ricceri, G. and P. Previatello [1971]. Caratteristiche del sottosuolo della laguna Veneta. *Atti Accademia Patavina Scienze Lettere Arti*, Vol.84 (in Italian).

Sridharan A. and A.S. Rao [1981]. Rectangular Hyperbola Fitting Method for One Dimensional Consolidation. Geotechnical Testing Journal, Vol. 4, No. 4, pp. 161-168.

Sridharan A., N.S. Murthy and K. Prakash [1987]. Rectangular hyperbola method of consolidation analysis. Geotechnique, Vol. 37, No. 3, pp. 355-368.

Tan S.-A. and S.-H. Chew [1996]. Comparison of the hyperbolic and Asaoka observational method of monitoring consolidation with vertical drains. Soils and Foundations, Vol. 36, No. 3, pp. 31-42.

Weltman, A. [1981]. A review of micropile types. Ground Engineering, May, pp. 43-49.

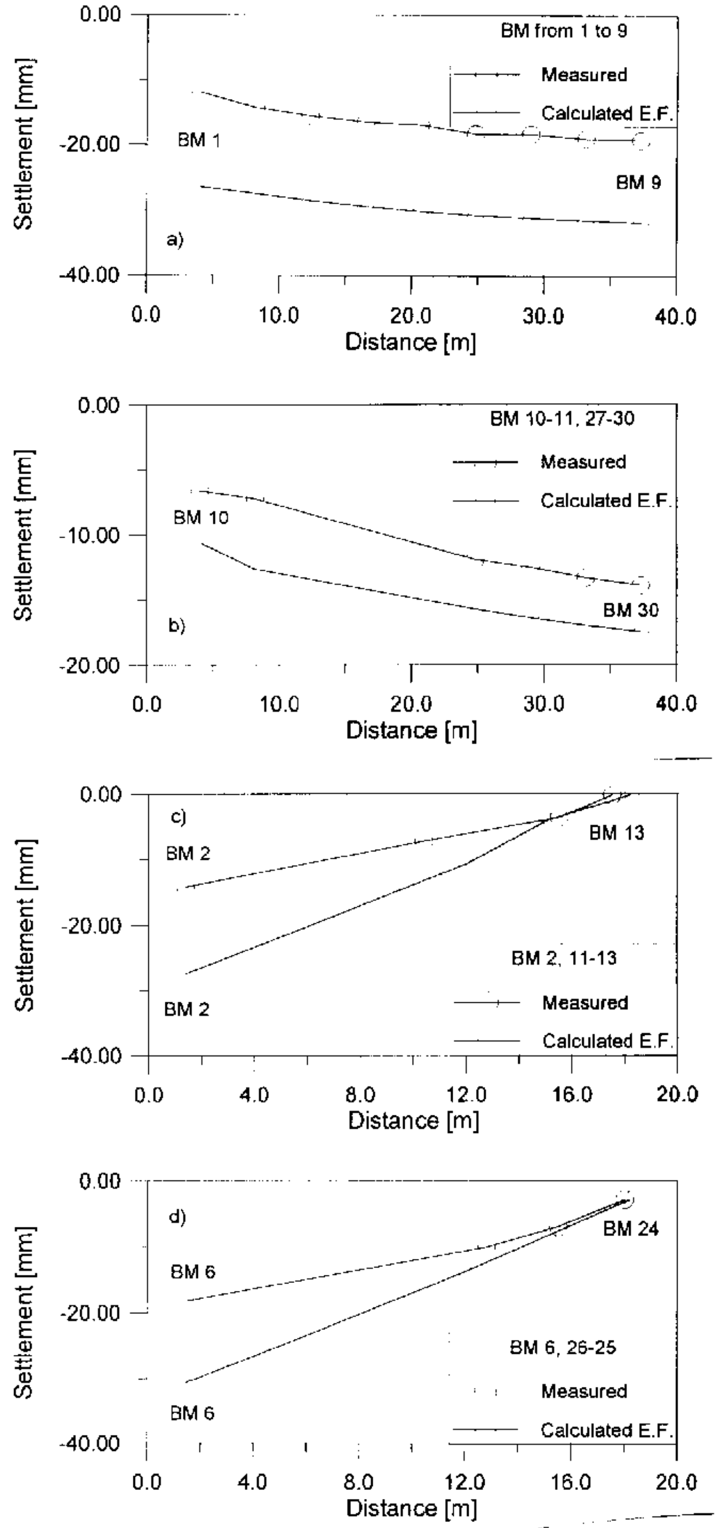

Fig. 17a-d Comparisons between measured and calculated (F.E.M.) settlements.

Wrench, B.P., W. Heinz and G. Salerno [1989]. Underpinning a multi-storey building using micropiles. Proc. of 12th Intern. Conf. on Soil Mechanics and Foundation Engineering, Rio de Janeiro, Balkema, Rotterdam, pp. 1043-1047.