Ground improvement for the foundation of a high-rise building

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Abstract

For the construction of a new office for the Lombardy region administration in Milan, Italy, a 161.3m high concrete tower with 39 floors and two basement levels, a ground improvement solution was proposed and implemented. This building constitutes the new headquarters for the personnel of the Lombardy region local authority, who were originally in different locations.

The objectives of this intervention were to increase bearing capacity and reduce settlement under the high rise building load. In a geotechnical context consisting of sand and gravel with water and a 3m silty layer, with low Young's modulus, at a depth of 30m from the surface, jet-grouting technology was chosen. After a preliminary phase of insitu and laboratory tests for the definition of the geotechnical model, columns were created using different sets of operational parameters and then examined to establish the most appropriate working procedure.

At the same time, the response of the ground, in terms of settlement under the building load, was simulated using FEM models. This paper describes the development of the design, from the initial definition of the geotechnical model, based on geotechnical tests, to the best value construction parameters of the grouted columns; and from FEM numerical simulation to predict settlement to the actual results measured after construction.

1. Introduction

This paper looks at the steps taken to develop the solution chosen for the ground improvement of the new office building in Lombardy.

The vertical structures of the building consist of a square concrete core, providing great rigidity from bottom to top, surrounded by a system of concrete columns. A 2m thick slab was chosen for the foundation, increasing to 3m in the core zone. The solution adopted for the ground improvement consisted of a system of jet-grouted columns on which the foundation slab of the building in reinforced concrete is set. After structural analyses it was clear that the vertical stresses under the slab would have been too high. In particular, maximum values of 700kPa and average values of 500kPa were deduced and judged excessive for the foundation system without improvement.

The studies were completed in terms of absolute and differential vertical settlement; theoretical results showed, in the worst conditions, maximum vertical translation of 40mm and differential vertical settlement of 20mm on a horizontal basis of 25m, which were judged unacceptable for the building.

A jet-grouting system was chosen because the technology for this kind of application is well developed in the area; it was preferred to concrete injections for timing and cost reasons. Before general jet-grouting operations began, a test bed was designed to achieve the following:

- identify the operating parameters for the construction of the columns, including the composition of the stabilizing mixture, the volumes and flow rate of the fluid introduced, the number and diameter of the nozzles, the pressure applied, the speed of withdrawal and rotation of the rods;
- verify the results of the treatment in terms of the geometry of the columns, diameter and continuity, and the strength of the improved ground.

The test bed was located in the area in which the future building was to be constructed and consequently the zero level, corresponding to the ground level, was assumed to be at elevation 125.07m.

The ground improvement operations were defined on the basis of the field tests and completed using finite element numerical modeling to predict and estimate expected ground settlement. The results of some of the calculations are presented here and compared with the monitoring data acquired during construction.

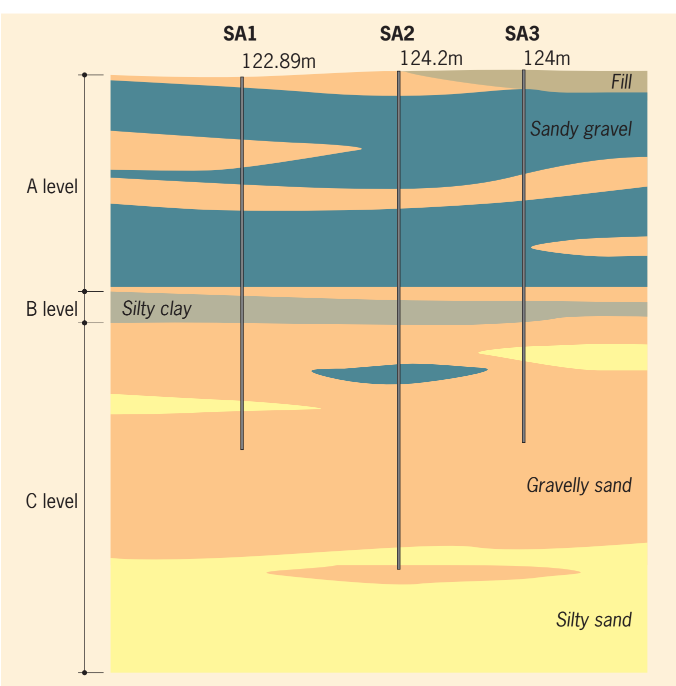

Figure 1 – Design stratigraphy

Table 1: Natural characteristics of the gravelly-sandy levels (A and C)

Core bore	Sample	Elevation (m)	w (%)	Gravel (%)	Sand (%)	Silt (%)	Clay (%)
	1	109.2	14.6	30	45	21	4
	2	103.7	9.3	44	40	14	2
	3	97.2	9.2	36	42	18	4
	4	85.7	15.1	2	70	26	2
SA1	5	79.5	11.9	10	60	26	4
	1	107.4	6.7	56	28	14	2
	2	102.1	10.5	42	43	13	2
	3	94.4	14.0	22	57	19	2
	4	85.6	16.0	9	79	11	1
SA2	5	77.9	14.0	22	65	12	1
	1	108.5	9.4	29	41	26	4
	2	102.9	10.4	40	40	18	2
	3	98.2	12.0	22	53	21	4
	4	88.5	12.5	22	51	23	4
SA3	5	80.8	12.7	20	63	15	2

Table 2: Natural characteristics of the cohesive level (B)

Core bore	Sample	Elevation	g_{nat}	g_{dry}	w	w_L	w_P	I_P	Gravel	Sand	Silt	Clay
		(m)	(kN/m³)	(kN/m³)	(%)	(%)	(%)	(%)	(%)	(%)	(%)	(%)
SA1	A	93.0	–	–	22.3	36	20	16	0	6	71	23
SA2	A	90.9	18.89	14.31	32.0	59	28	31	0	6	63	31

Table 3: Summary of elastic modulus (values in MPa)

LEVEL	E_d	E_{txCU}	E_{txCD}	E_{press}	$E_{0\ SPT}$	E_{din}
A				40	140-220	800-950
B	10-20	60-100	70-150	35-45		660-760
C					240-250	900-1,100

Table 4: Summary of values for cohesion and angle of friction

LEVEL	Direct shear		Triax CU		Triax CD		SPT
	c' (kPa)	f' (°)	c' (kPa)	f' (°)	c' (kPa)	f' (°)	f' (°)
A							40-43
B	20-35	22-27	25-45	21-25	20-45	20-26	
C							40-41

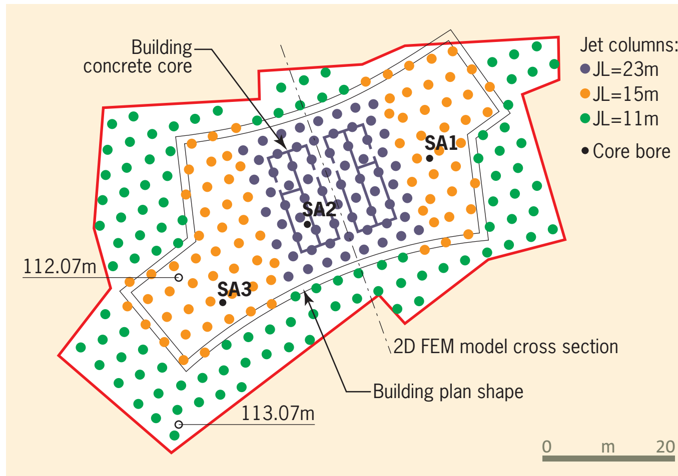

Figure 2: Plan view of the project

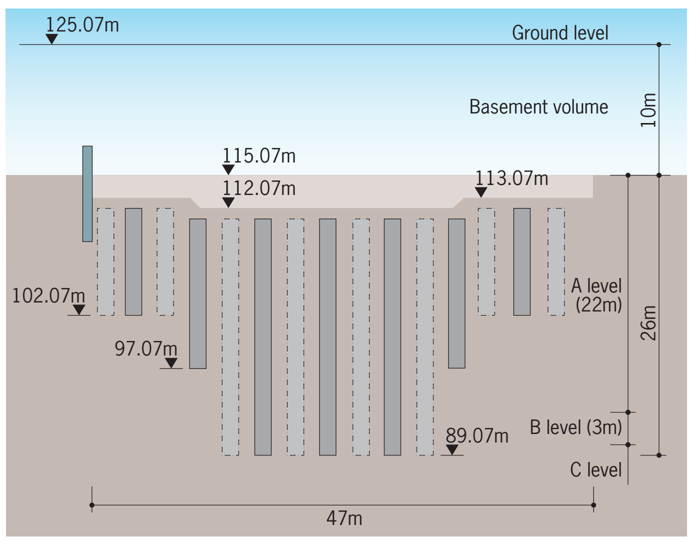

Figure 3: Cross section of the project (through the minimum width of the slab)

2. Geotechnical conditions and stratigraphy

The soil in the construction area consists mainly of gravelly-sandy deposits and in a stratum of silt and clay, with very different geotechnical characteristics compared to the surrounding ground.

Examination of the stratigraphy from the boreholes revealed the following:

- a heterogeneous surface level consisting of backfill and reworked soils with thicknesses ranging from 1.5m to 3.0m;
- a first succession of gravelly-sandy layers with subordinated pebbles and silt with elevation from 125m to 93m defined as LEVEL A (its bottom surface were then at about 32m below ground level);
- a silty-clayey level with an average thickness of between 2m and 3m, ranging from solid to very solid and continuous throughout the area in question, defined as LEVEL B;
- a final level consisting almost entirely of sandy soils, defined as LEVEL C, with more gravelly subordinate levels. This was discovered below the cohesive level, down to the maximum depth surveyed, set at 50m below ground level.

The surface of the water table in the area examined was found to be between 105.09 and 105.90m and therefore at a depth of between 19.17m and 19.98m below ground level, with a direction of flow from north-northwest to south-southeast. For design purposes the altitude of the water table was assumed to be 106.07m and therefore 19m below ground level.

Table 5: Design parameters for cohesion and angle of friction

LEVEL	c' (kPa)	f' (°)	E (MPa)
A	0	37	100
B	20	22	30
C	0	37	150

3. Geomechanical characterisation of the site

The geomechanical characterisation of the site was performed by means of a series of geological survey and laboratory tests; a summary of the results is given in Figure 1. In terms of natural characteristics Tables 1 and 2 summarise the parameters obtained from the laboratory tests.

3.2 Elastic Modulus

Numerous in situ tests such as SPT, pressure meter and seismic cross-hole were carried out to determine the design values for the elastic moduli, which are very important for settlement analysis. Laboratory oedometer and triaxial tests were also carried on gravelly-sandy samples.

Table 3 gives a summary of the values obtained in MPa where

- E_d = Elastic modulus after seismic test
- E_{txCU} = Elastic modulus after consolidated undrained triaxial test
- E_{txCD} = Elastic modulus after consolidated drained triaxial test
- E_{press} = Elastic modulus after pressiometric test
- $E_{0\ SPT}$ = Elastic modulus after Berardi e Lancellotta (1992)
- E_{din} = Elastic modulus after cross-hole test

3.3 Strength parameters

SPT insitu and direct shear and triaxial laboratory tests were performed to determine the design parameters for cohesion *c'* and angle of friction *f'*. Table 4 gives a summary of the values obtained. For gravelly and sandy soils, A and C levels, very common in the chosen area, SPT blow counts were considered sufficient to mechanically describe materials.

Table 5 gives design parameters which were assessed on the basis of the data reported; those adopted were statistically based on the results of testing and observation, as well as experience in underground work in the Milan area and similar conditions. For granular soils Young's modulus design values were mainly based on SPT tests; for silty-clay the definition base was substantially »

Table 6: Test column data

Column	Construction date	Column length (m)	Column head elevation (m)
C1	2007, January 2	35	125
C2			
C3			
C4	2006, December 15		
C5			
C6			
JB38	2007, February 26	23	112
JA2	2007, March 1		

Figure 4: Photograph of jet-grouted column

Figure 5: JB38 core, elevation 99m-94m

« constituted by triaxial tests. In both cases a safety factor was applied to minimum values.

4. Description of ground improvement operations

The design of the tower foundations involved ground treatment consisting of "bi-fluid" jet-grouting columns. This solution was chosen to control building settlement during the construction and service stages. The ground treatments, columns with a nominal diameter of 1,500mm, were performed in a quincuncial pattern with sides of 3-3.5m and length varying from 11m at the edges of the slab to 23m under the central core of the building, as described in Figures 2 and 3.

The lengths were calculated according to the distribution of the loads bearing on the base of the slab in an attempt to make settlement uniform and minimise differential values.

The design parameters employed for the dimensions of the jet-grouting operations were as follows:

- Nominal diameter in the gravelly and sandy formations: 1,500mm
- Nominal diameter in the clayey formation: 1,200mm
- Ultimate compressive strength: 8-10MPa
- Elastic modulus in the gravelly and sandy formations: 8,000MPa
- Elastic modulus in the clayey formation 1,500MPa

Values of elastic modulus appear to be different for gravelly/sandy and clayey formations because jet-grouting columns are constituted by mixing in situ materials and concrete. Formations involved in treatment strongly influence then the results in terms of mechanical features of improved soil.

5. Description of the test bed

5.1 Location and geometry

A test bed was employed for the design of the ground improvement columns consisting of two stages. The first stage of tests was performed in December and January 2007 with the creation of two sets of three columns labelled C1-C2-C3 and C4-C5-C6, with a length of 35m running from ground level, 125m approximately. Following initial test surveys, the second phase investigated the success of the ground treatment in the deepest layers, within and below the clayey formation.

In March and April 2007 two columns were constructed, labelled JB38 and JA2, running from approximately 112m, with a length of 23m, enough to completely cross LEVEL B. Table 6 summarises the programme.

5.2 Operational parameters adopted

The operational parameters adopted for the construction of the columns using a "bi-fluid" jet-grouting system, where in addition to the injection of a binary water-concrete mixture air is injected with an annular jet, are given in Table 7; following experiences in a similar context this technology was considered optimal to obtain good results. The delivery pressure of the mixture was assumed to be 45MPa (450bar). The "bi-fluid" type mixture injected involved the use of cement type 325, with a water to cement ratio of 1 and a cement dosage of around $6kN/m^3$.

5.3 Summary of the investigations carried out

After an appropriate period of at least 10 days to allow the cement to harden, a number of investigations were carried out to assess the effectiveness of the treatments:

- the tops of the columns were uncovered for visual examination and measurement of the geometry;
- continuous core drilling was performed along the central axis of the columns, with a description of the stratigraphy and photographs;
- samples were taken, monoaxial compression tests were performed and exact weight was measured;
- seismic surveys were performed.

6 results of the investigations performed

6.1 Visual examination of the ground improvement

Visual and geometrical examination of the ground improvement was performed by uncovering the tops of C1, C2, C3, C4, C5 and C6 columns to a depth of approximately 4m from ground level (Figure 4).

Measurement of the circumferences and the diameters are given in Table 8. With the exception of column C1, which broke during excavation, the results showed columns with a diameter well in excess of the nominal design dimension of 1,500mm. The geometries were regular, especially for the set C4-C5-C6, for which the treatment appeared more homogeneous, evidence that the mixing action occurred correctly.

Columns C1-C2-C3 were less regular, probably because of the presence in the ground of larger chunks of material, which allowed the injected grout to take specific paths. However, these too showed satisfactory amounts of improved ground. Visual examination of the heads of the columns led to the selection of columns C5 and C6 as the most representative and suitable for specific testing; core drilling was then performed along their central axes.

6.2 Core samples and geophysical investigations

The core drilling on columns C5 and C6, with a diameter of 131mm, was performed with hydraulic rotation; it showed good performance down to a depth of approximately 24-26m from ground level (el. 101-99m); at greater depths the treatment had performed poorly and in some sections ground improvement was virtually non-existent. In detail, column C5 had good ground improvement down to 24m from its head, with average RQD values of around 60-70% and some more fractured zones (RQD of 20-30%). C6 quality was found to be excellent down to a depth of 25m, with markedly higher RQD values, consistently greater than 70-80%.

At greater depths the quality was lower, especially in the cohesive stratum (LEVEL B). On the basis of these results two new test columns, labelled JB38 and JA2, were constructed with five-inch drill diameters. The new tests gave good ground improvement results for the

Table 7: Operational jet-grouting parameters

Columns	Number of nozzles	Nozzle Ø (mm)	Flow rate (L/min)	Speed of rotation (rpm)	Rate of withdrawal (cm/min)	Withdrawal time (sec/4.0 cm)	Specific energy (MJ/m)
C1	1	5.00	280	6	20	12.0	57
C2 and C3	1	5.50	340	7	23	10.5	60
C4, C5 and C6	1	4.50	230	5	15	16.0	61

Table 8: Results of the geometrical examination of the tops of the columns C1 - C6

Column	Column height (m)	Circumference (m)	Direct measurement of diameter (m)	Diameter deduced from circumference (m)
C1	1.70	1.75	0.45	0.55
C2	3.70	6.70	-	2.13
C3	3.55	6.85	-	2.18
C4	2.25	6.90	2.15	2.19
C5	0.80	6.95	2.10	2.21
C6	2.80	6.10	1.85	1.94

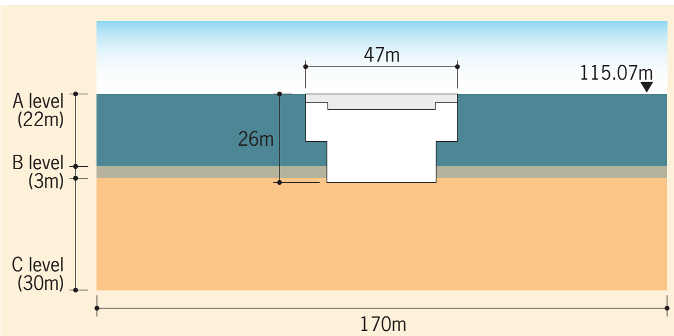

Figure 6: Calculation scheme

entire length of the columns (Figure 5), even in the cohesive stratum and in the granular deposits below.

In addition to the direct tests on the columns, the study included a geophysical investigation with cross-hole, topographic and mechanical admittance tests. In the sands and gravels under the water table the velocity of the shear waves without treatment generally increased according to depth and was between 300m/sec and 400m/sec. Under the same conditions the compression waves were between 1,200m/sec and 2,300m/sec. In the improved ground the investigations found an increase in velocity of between two to two and a half times those in the untreated ground; even greater increases were found locally in the more permeable strata. Finally, the tomographic and admittance tests confirmed the expected diameter reduction in the clayey level.

6.3 Laboratory tests

The following laboratory tests were performed on the core samples obtained from drilling:

- calculation of the specific weight;
- compressive strength by means of a monoaxial compression test, 28 days after the jet-grouting was performed.

The results of the laboratory tests relating to compressive strength gave average values higher than 8.5MPa. More specifically, the tests conducted on the samples from columns C5 and C6 gave values constantly higher than 8.5MPa, with an average value of 9.4MPa, excluding the samples taken from 97m where the ground treatment had not been successful.

The tests conducted on the samples from columns JB38 and JA2 also gave values generally higher than 7.0MPa, with an average value of approximately 8.6MPa, excluding the two anomalous results of 4.26MPa and 4.0MPa. The latter appeared to be associated with local test problems like the presence of pebbles or uncemented levels which affected the scale of the sample. The results of the laboratory tests are summarized in Table 9.

Table 9 - Laboratory test results for samples

Column	Sample	Elevation (m)	Compressive strength (MPa)	Specific weight (kN/m³)
C5	C5-1A	112.6	9.36	23.56
C5	C5-1B	112.3	11.36	22.75
C5	C5-2A	106.9	9.90	23.46
C5	C5-2B	106.6	9.22	22.15
C5	C5-3A	98.8	9.54	23.66
C5	C5-3B	98.7	9.88	23.36
C5	C5-4	97.9	8.88	22.35
C6	C6-1A	109.7	8.97	22.45
C6	C6-1B	109.3	9.16	22.65
C6	C6-2A	103.4	9.68	23.16
C6	C6-2B	103.3	8.77	22.89
C6	C6-3A	94.7	9.39	21.59
C6	C6-3B	94.5	8.66	21.89
C6	C6-4A	92.7	4.22	18.77
C6	C6-4B	92.5	2.85	19.17
JB38	JB38-1	98.6	9.00	-
JB38	JB38-2	92.5	4.26	-
JB38	JB38-3	90.4	4.00	-
JA2	JA2-1	109.0	7.00	-
JA2	JA2-2	98.3	9.60	-
JA2	JA2-3	96.3	8.60	-
JA2	JA2-4	92.6	8.40	-

7. FEM analysis

The study of the foundation on the improved ground was conducted using multiple finite elements models. Details are given from a two-dimensional section traced along the axis of the slab along the shorter side (Figure 6).

This approach was chosen after deducing that the shape of the building, rectangular in plan, could generate a plane stress state at foundation level, especially when horizontal loads, wind in particular, were applied.

The calculation mesh employed possessed 2,624 triangular elements and 1,393 nodes. It runs for 170m on the horizontal plane and for 55m on the vertical, and is anchored vertically along the lower edge and horizontally on the left and right sides. The discretisation was kept more concentrated in the central upper zone where the stresses were of greater magnitude and interest.

The study was developed in stages as follows:

- Stage 1 – Geostatic condition
- Stage 2 – Implementation of jet-grouting and rc slab
- Stage 3 – Application of the design loads

Design loads are divided into dead loads and working loads, where wind contribution plays an important role. At Stage 3 Figures 7, 8 and 9 show results with only dead loads (PP) or with dead and working loads in worst conditions (SLE). Settlements at Stage 2 (Figure 7) are only related to concrete slab and jet-grouting weights greater than those of the ground.

In Figure 8 geostress is always included in the vertical stress. The cases analyzed, SLE and PP, show similar results, although the stresses and settlements are more uniform in the second case due to the low eccentricity. Figure 8 clearly shows the presence of maximum loads over the largest of the staircase and lift structures.

In terms of axis, v represents vertical displacement, s_z vertical stress and x the coordinate also shown in Figures 2, 3 and 6.

Ground improvement were introduced in FEM analyses considering materials with better mechanical features, in particular increasing elastic modulus and strength, with the values described in Chapter 4. The system of columns were transformed into homogeneous materials with average parameters, different in case of presence of gravelly/sandy or clayey formations. On the basis of the tests, these parameters were deduced considering the presence of treated material (column) inside the soil matrix, acting proportionally to the volumes.

This approach doesn't take into account any mechanical direct interaction between columns and untreated soil in which these elements are realized. From »

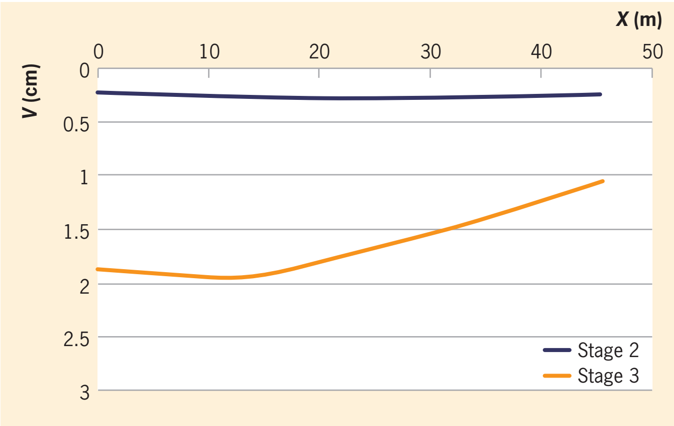

Figure 7: Results of the SLE calculation – vertical movements upper edge of the slab at Stages 2 and 3

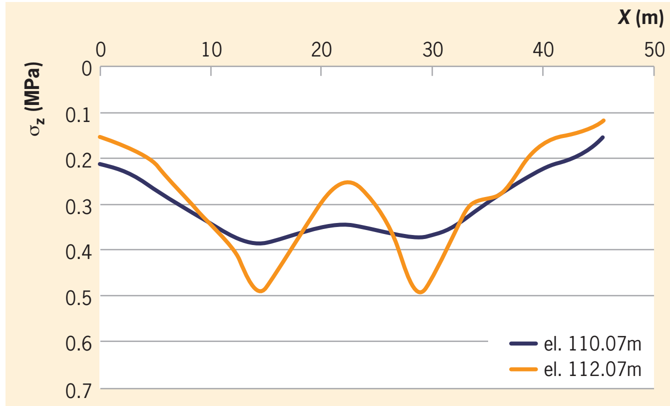

Figure 8: Results of the PP calculation – Stage 3 vertical stresses on the earth-slab contact interface (el. 112.07m) and 2m deeper (el. 110.07m)

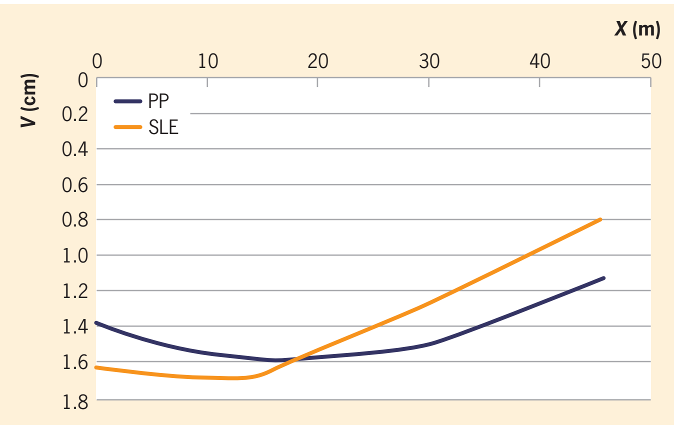

Figure 9: Comparison of the results of the PP and SLE calculations – vertical movements, upper edge of the slab in Stage 3 without the effects of the movements relating to the previous stages

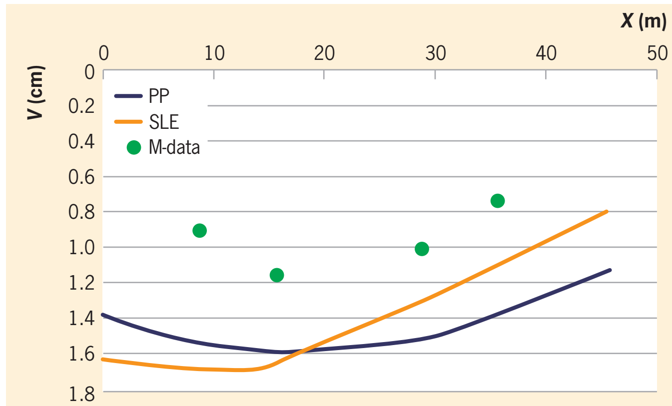

Figure 10: Results of the PP and SLE calculations – vertical movements, upper edge of the slab in Stage 3 without the effects of the movements relating to the previous stages. Comparison with monitoring data

an engineering point of view this aspect was neglected, because considered of secondary order and surely ameliorating in terms of strain reduction of the improved soil foundational system.

8. Monitoring results

The construction was monitored using a complex and co-coordinated system during the entire course of the building work. Particular attention was paid to the tower, observed using a horizontal grid of 50 points, 10 of which were on the perimeter of the central core. Figure 10 gives the readings for the vertical movement of four points located in the FEM calculation cross section, and the values given are for the completion of the work.

They therefore take account of the application of dead loads, permanent loads and a live load component in a theoretically intermediate condition between the simulated PP and SLE cases.

Examination of the results shows movements of a smaller size than predicted, with basic confirmation of the orders of magnitude and global behavior. These elements demonstrate how ground treatment had higher performance, in terms of strain control, than what estimated. The cited mechanical direct interaction between columns and untreated soil surely played a role in this better results.

9. Conclusions

This paper summarises the work performed on the design for the ground improvement for the foundations of an office building in Milan, Italy. The study, developed in stages, began with the characterisation of the site (definition of the geotechnical model) and with some preliminary dimensions.

The large size of the building, with a height of approximately 160 m and 39 floors, required ground improvement due to the loads and the need to control settlement. Indeed, preliminary studies showed a high degree of settlement related to weight and wind action. These levels were considered unacceptable and, after technologic and economic analyses, ground treatment in the form of jet-grouted columns was chosen.

Firstly, computer simulations were created that allowed us to choose geometry and features that would guarantee a sufficient reduction in building settlement. Work then began with the construction of a number of test columns, in order to define optimum operating parameters. Continuity, strength, shape and treated soil volumes were verified to optimize the intervention.

From a computational viewpoint, the stress-strain behaviour of the ground under the footprint of the tower was modelled using finite element software. The results obtained were compared with those found after monitoring activity had been performed during construction.

The settlement found matched expectations; it is this aspect that represents the real aim of this work and it clearly confirmed the validity of the process, from the design stage to the creation of the columns, and from displacement prediction to the effective performance of the system.

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