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INJECTION OF A MICROFINE CEMENT GROUT FOR THE TUNNELING OF METEOR

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ABSTRACT

The fourteenth line of the Paris metro (France), called Météor, is currently extended towards Saint-Lazare railway station in a very busy and historic district of Paris. The Transport Company (RATP), as a contracting authority, is then confronted with a complex environment. The constraints have determined works planning and construction methods. In particular, the alluvial deposits were reinforced by injection of microfine cement grout whose fluidity allows to accurately permeate fine soils. It was preferred to more traditional and potentially polluting grouts. After a description of the geotechnical conditions of the site, the different steps of underground constructions are briefly showed. Many control measurements have been integrated to the project to ensure the integrity of the surrounding structures. Grout pressure was thus continually fitted to the heaving of the foundations of sensitive buildings. *In situ* pressuremeter tests were also carried out before and after grout injection in order to verify the quality of soil improvement. Likewise, triaxial tests were performed on uncemented alluvial deposits and on alluvial deposits grouted in the laboratory. These tests show that the mechanical properties of the alluvial deposits were clearly improved by grouting.

METEOR: A FULLY AUTOMATED SUBWAY LINE

Many offices gather round the Saint-Lazare railway station in the 8^{th} district, in the heart of Paris (France). The increase of the pedestrian and automobile traffics revealed the necessity for an underground transportation infrastructure that could finely serve this busy and historical district of the city. Therefore it has been decided to extend the 14^{th} subway line Meteor. This line has linked the French National Library and the Madeleine station (Phase 1 of the project) since 1999, relieving the RER A commuter line of saturation at rush hours (Selosse 1992). The extension of the 14^{th} subway line between the Madeleine station and the Saint-Lazare railway station (Phase 2) began in July 1998. This underground part 500 meters in length is spread over five years.

TECHNICAL AND GEOLOGICAL CONTEXT

For the second phase of the project, the underground structure comprises three parts (Fig. 1):

- the tunnel before the station;
- the station close to the Saint-Lazare railway station;
- the tunnel after the station.

In Fig. 2, the main underground structures and the buildings at the surface are superimposed. The tunnel before the station, 130 meters long, starts beneath Haussmann Boulevard from the Anjou shaft and ends once level with Pepiniere Street. It splits up into two levels separated by an intermediate slab (Fig. 2a): the lower and the upper parts respectively constitute the subway traffic lane and the connection with the 9^{th} subway line. Because of the congestion of the subsoil, the tunnel had to come in between the 9^{th} subway line and the RER A line below Haussmann Boulevard. It had also to skim the foundations of a bank (Fig. 2b). For this important building, the instructions as for the settlements caused by the excavation or the upheavals caused by the grouting process were particularly strict. This part of the tunnel extension is now almost completed.

The station was built simultaneously with the tunnel before the station. This structure constitutes the most outstanding part of the project. Indeed, from the mezzanine at the surface or from the 3^{rd}, 12^{th} and 13^{th} subway lines, the passengers will be able to approach the station through a vast complex with a neat architecture. The underground platforms, 120 meters in length and 6 meters in width were being built from an open cut shaft protected from collapsing by Berlin type walls and located in front of the Saint-Lazare railway station. The inner span of the station is 17.48 meters. Also note that the work schedule was relatively difficult to establish because the running of the other subway lines had not to be stopped.

Finally, the tunnel after the station will allow to turn round the trains or to immobilize them. This part will be 258 meters in length.

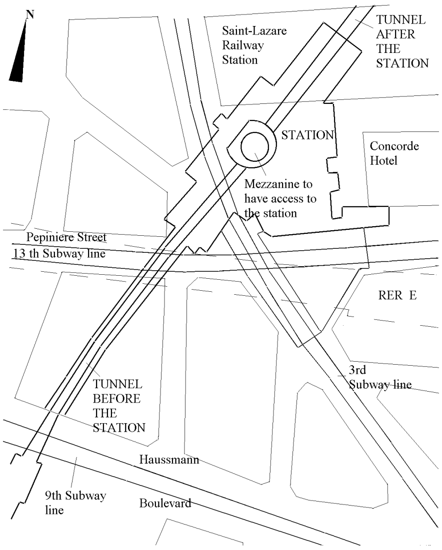


Fig. 1. Extension of METEOR.

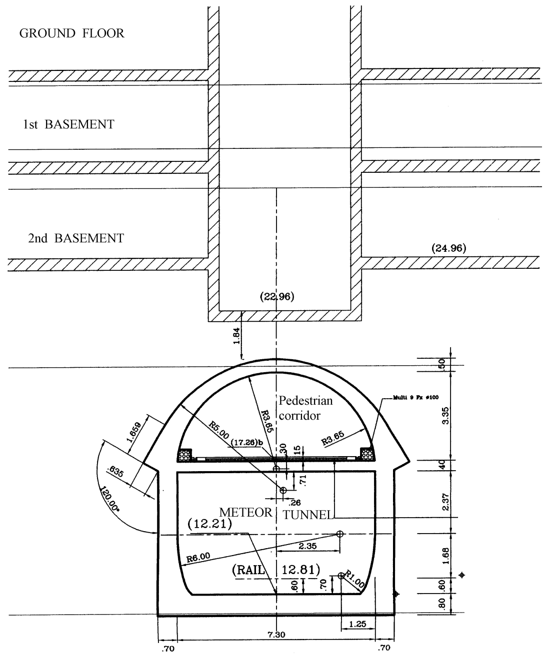
<u>Local geology</u>

The geology of the Paris basin is well known by the great experience acquired in the past during the construction of many underground structures (subway lines, sewers, pipes, …) and by the soil investigations carried out for the Meteor project. In the zones corresponding to the station and the tunnel before the station, the stratification, from the surface to the substratum, was found to be a succession of:

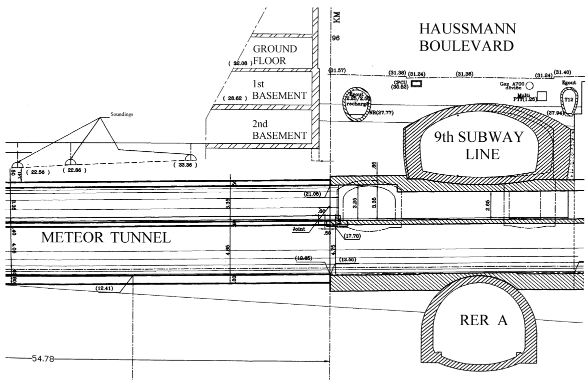
- fills, 3 meters thick;
- soft old and recent alluvial deposits, 7 meters thick;
- marls about 10 meters thick;
- coarse limestones;
- Cuisien sands.

The alluvial deposits are sandy-gravelly siliceous and calcareous soils without cohesion. Their coefficient of permeability is between 10^{-5} and 10^{-3} m/s.

In the zones corresponding to the tunnel after the station, site investigations also showed the presence of a Beauchamp sand layer between the fills and the marls, instead of alluvial soils. More, it was established that the marls had undergone a weathering that mainly appeared in the form of disturbed weak zones and Beauchamp sand inclusions that had migrated from the upper layers because of the decomposition of gypsum inclusions. We finally noted the presence of two water tables: the Lutetien table whose surface is 10 meters below the ground level and the Cuisien table.



(a)



(b)

Fig. 2. The tunnel before the station.

TUNNELING METHODS

Layout of the tunnel

The layout complied with the constraints of insertion of the tunnel between the existing underground structures and the positioning of the tunnel invert in the calcareous substratum. The vault was mainly set up in the marl layer but sometimes outcropped in the soft alluvial deposits as in the tunnel before the station and in the station for instance.

Ground improvement

Before excavating, the alluvial soils underwent a suitable improvement by means of a grout injection whose aim was to reinforce the soil and to prevent high inflows of water. The injection was performed in two steps:

- filling of the major voids by injection of bentonite-cement grout (subsequently noted BC), in particular at the interface between the marls and the alluvia;
- filling of the residual voids by injection of a microfine cement grout (subsequently noted MC) preferred to traditional organic grouts for environmental considerations.

The treatment was associated with a pumping from open sumps in order to lower the level of the Lutetien water table.

In the case of the tunnel before the station, grouting ahead of the tunnel face was carried out by passes of 22 meters long (Fig. 3).

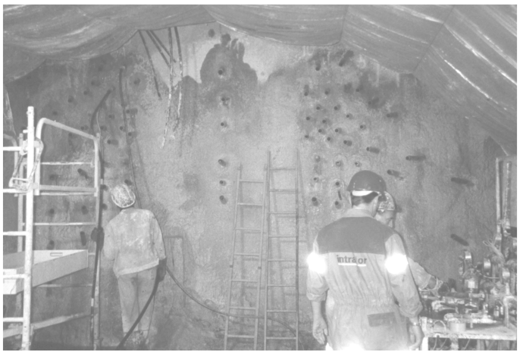


Fig. 3. Tunnel face with the injection process.

For the station, a longitudinal gallery with a radius of 3.70 meters was bored in the tunnel alignment from the shaft located at Rome Place, in front of the Saint-Lazare railway station. Ground improvement was performed from the gallery face by a 4 meters large radial grout injection over the permanent extrados. Sleeve grout pipes or "*tubes à manchettes*" in French (Fig. 4) were systematically used.

The sleeve grout pipes are PVC pipes that are introduced in boreholes. They are cased in the soil with a casing grout that prevents the grout to rise along the tube. Two packers define the section of the ground that has to be improved. The flowable grout under pressure reaches this section, then inflates the rubber sleeves, breaks down the casing grout and finally impregnates the soil. Improved mechanical properties are obtained with the setting of the cement. It is therefore possible to excavate the soil after 28 days. In the case of Meteor, the injection process represented about 25 km of drilling holes and 8000 m^3 of grout.

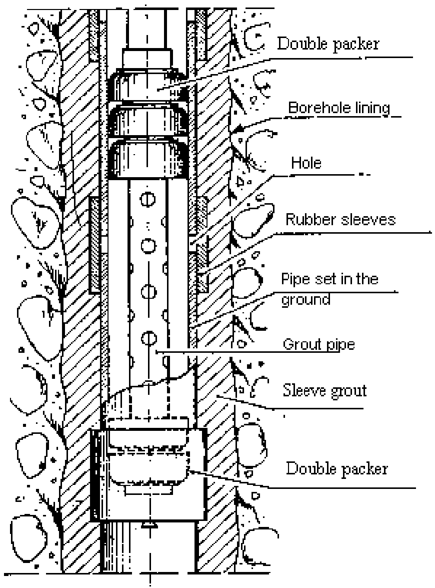


Fig. 4. Sleeve grout pipes device (after Cambefort 1992).

Excavation methods

A slurry pressurized shield was used for the first phase of the project, between two shafts called Arsenal (after the east sub-fluvial crossing of the Seine River) and Anjou. The quite small length of the second phase did not justify the use of such an expensive device. Thus the excavation works were and are still carried out with a conventional mechanized technique by means of boomer machines, hydraulic wedges and drag shovels.

After the grout injection in the tunnel before the station, the upper half section was excavated with a full section method. The supporting structure was made of arches lost in shotcrete.

The vault and the intermediate slab were then concreted by 6 meters long passes with a metallic form. The lower half section was underpinned in two times on its full height. During the excavation stages, the structure was supported by two concrete abutments that leant on the soil, as shown in Fig. 2a.

For the station, the side drifts with a width between 4.20 and 5.90 m and a height between 5.10 and 6.30 m were excavated in two times on their full height on either side of the main shaft. They were then concreted. The vault of the station was built by 1.50 m long passes. HEB arches and shotcrete were also implemented. A waterproof membrane was also previously set above the vault to prevent any subsequent water rush.

CONTROL OF THE GROUT INJECTION PROCESS

Displacement survey

In order to avoid any upheaval of the bank foundations above the tunnel before the station, strict instructions had been imposed. Thus, the maximal upward displacement was 10 millimeters beneath the buildings and 15 mm beneath the roadways. These constraints caused to use a grout easily penetrating the alluvial deposits without excessive grout pressures.

After a first treatment with a bentonite / cement grout (noted BC in Table 1), a patented microfine cement grout (noted MC in Table 1) was injected, from the tunnel face, with a rate of discharge of about 7 l/mn. The cement contents were 150 kg/m^3. The grout volume was 30 % of the volume of the zone to be treated. The pressure whose maximal value was set to 0.8 MPa could be regulated according to the displacements monitored from settlement hubs.

Case of the bank

The excavation works were carried out with a furtherance by sections that successively and independently underwent an injection of bentonite cement, an injection of microfine cement and were finally excavated. Two sections, namely S1 and S2, were more especially followed. Displacements of 4 significant points (8r and 11 for the section S1, 4 and 7r for the section S2) were monitored. In all, more than 40 points were surveyed at a rate of one measure per day in the critical stages of the works and one measure per week out of these periods. The schedule for the underground works is reported in Table 1.

Figure 5 shows the displacements recorded for the 4 points. In this figure, each tick in the abscissa axis represents 20 days. Positive values mean an upheaval, negative ones a settlement of the ground. The vertical continuous lines depict the successive stages of the works: (1) end of BC injection, beginning of injection of Intra-J, (2) excavation of the section S1, (3) injection of BC and IJ in the section S2, (4) excavation of the section S2, (5) end of excavation of the 2 sections.

Table 1. Schedule for the works beneath the bank.

Section	Period		Operation
	Outset	End	
S1 / S2	30/09/98		Level references
S1	15/10/98	30/10/98	BC injection
S1	13/11/98	03/12/98	MC injection
S1	03/12/98	24/03/99	Excavation
S2	24/03/99	02/04/99	BC injection
S2	02/04/99	28/04/99	MC injection
S2	28/04/99	25/05/99	Excavation

The injection process induces an upheaval of the foundations of the bank mainly due to the microfine grout whereas the excavation almost resets them at their initial position. Besides, the treatment of the section S1 causes some displacements of the structures in the second section. This means that the microfine grout penetrated quite far away in the soil.

Finally, some points present an upheaval that exceed the admitted displacement. Thus 3 points out of 11 in the section S1, as the point 11, and none in the section S2, were beyond 10 millimeters. The maximal value was 11.1 millimeters. These upheavals were mainly due to the microfine grout that had to fill the residual smallest voids. For instance, if we consider the point 11, 3.6 mm and 5.8 mm could be directly and respectively attributed to the bentonite cement grout and microfine cement grout injections. Nevertheless, a global upheaval of the foundations is not usually prejudicial to the structures, contrary to the differential displacements. In the first section, the longitudinal differential displacement between the 2 points 8r and 11 was equal to $+0.29\ ^{o}/_{oo}$ after the BC injection, $+0.42\ ^{o}/_{oo}$ after the IJ injection. and $-1.67\ ^{o}/_{oo}$ during excavation. Likewise, in the second section, the transversal differential displacement (the point 4 is close to the sidewalls of the tunnel whereas the point 7r is on the alignment) was equal to $-0.07\ ^{o}/_{oo}$ after the BC injection,

+0.02 $^{o}/_{oo}$ after the IJ injection. and +0.30 $^{o}/_{oo}$ during excavation.

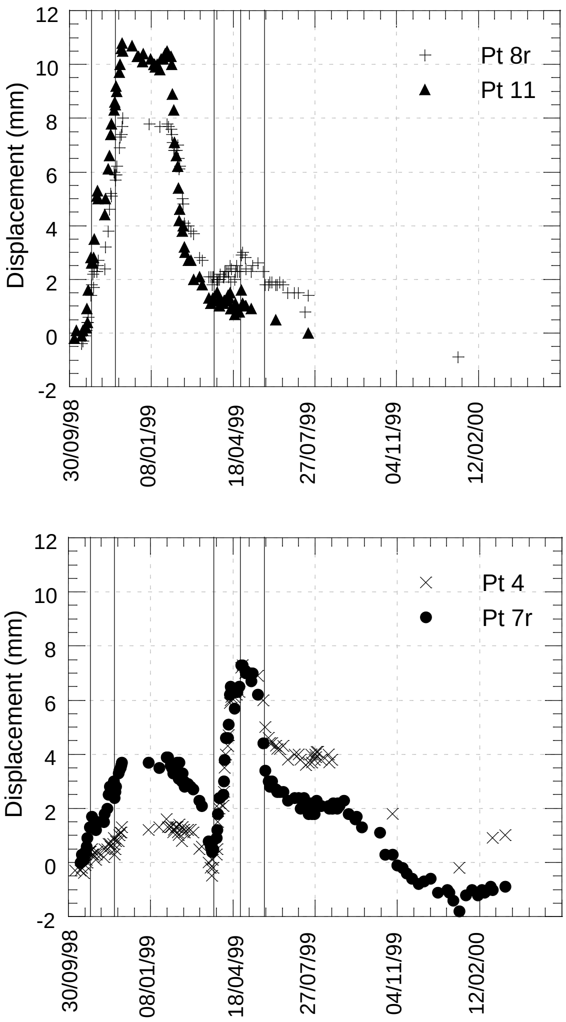


Fig. 5. Survey of the displacements due to grout injection.

<u>Pressuremeter tests</u>

Menard pressumeter tests, almost all of them comprising an unload – reload cycle, were conducted in the site in order to assess the mechanical improvement of the soil:

- 4 tests in the alluvial deposits before grouting close to Haussmann Boulevard (tests referred Hauss);
- 4 tests in the untreated soil in Pasquier Street (tests referred Pasq);
- 6 tests (2 in the untreated soil and 4 in the grouted soil) carried out from the basement of the bank (tests referred Indo);
- 2 tests performed in the North tunnel gallery of the station in the grouted soil (tests referred GalN);
- 7 tests performed in the South tunnel gallery of the station in the untreated soil (tests referred GalS).

For the pressuremeter tests carried out in the two side drifts of the station, the pressuremeter probe was located above the Pressure-Volume Control system as shown in Fig. 6. The Menard pressuremeter test, that consists in expanding a probe in a borehole, was chosen because of the ease in conducting the test, of the ability to give information about the deformability and the strength of the soil and of the simple boundary conditions that allow to easily model the test.

<u>Conventional interpretation</u>. Field data were examined following the French Standard NF P 94-110-1 and NF P 94-110-2, in order to determine the Menard pressuremeter modulus E_{m1}, the pressuremeter modulus E_{m2} measured on the unload – reload cycle, the creep pressure P_f and the net limit pressure P_l*.

The results are reported in Table 2. The depth noted H was estimated from the NGF level in comparison with the reference level in Haussmann Boulevard. NG and G respectively stand for non grouted soils and grouted soils.

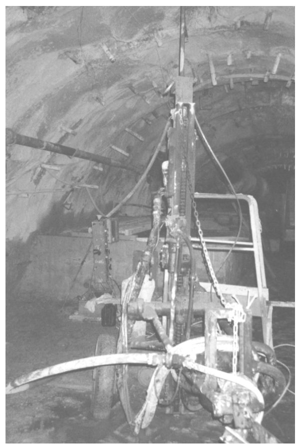


Fig. 6. Pressuremeter tests conducted in the vault.

Some tests were not representative of the alluvial deposits. Therefore, the following tests were not subsequently considered:

- the tests referred Hauss and Pasq because the ground had been previously grouted by a silicate gel at the time of the construction of the RER A line;
- the test noted Indo PR5 at a depth of 3.7 m because of the uncertainty about the nature of the soil;

- the tests referred GalSud PR8 because of the difficulty in the carrying out of the tests in a soft soil;
- the tests referred GalNor because of the uncertainty about the nature of the soil.

Table 2. Results of the pressuremeter tests.

Location		H	E_{m1}	E_{m2}	P_f	P_l*
		(m)	*(MPa)*	*(MPa)*	*(MPa)*	*(MPa)*
Hauss PR1	NG	6	28	56	1.9	3.04
Hauss PR1	NG	7	33	110	1.9	3.44
Hauss PR1	?	8	31	88	1.5	2.61
Hauss PR1	?	9	33	86	1.7	3.02
Pasq PR2	NG	6	33	144	2.0	3.40
Pasq PR2	NG	7	18.5	118	1.8	3.10
Pasq PR2	NG	8	29	114	2.2	4.10
Pasq PR2	NG	9	32	166	3.0	5.10
Indo PR3	G	8.3	44	129	2.0	3.35
Indo PR3	G	9.3	69	130	>3.9	>6.56
Indo PR4	G	8.5	34	73	>2.7	>4.60
Indo PR4	G	9.5	32	63	>3.2	>5.44
Indo PR5	NG	8.2	4.7	12.5	0.3	0.51
Indo PR5	NG	9.2	17.3	63	1.5	2.52
GalN PR6	?	6.8	32	96	1.9	3.20
GalN PR6	?	8.8	20	44	0.9	2.10
GalS PR7	NG	8	6.5		0.4	0.88
GalS PR7	NG	9	6.7	23	0.8	1.28
GalS PR7	NG	10	13.3		0.8	1.40
GalS PR8	NG	8	2.6		0.4	0.51
GalS PR8	NG	9	1.1		0.2	0.34
GalS PR8	NG	10	11.4		1.4	2.30
GalS PR8B	NG	10.9	5	24	0.3	0.54

The exploitable data clearly showed the efficiency of the grout injection on the pressuremeter parameters. An improvement ratio X* was defined as the ratio between the value of a parameter after injection on its value before injection. Only average values were considered because of the scattering of the field data.

The improvement ratios indicated in Table 3 are common values for a grout injection whose aim is to reinforce the soil. For a watertightness treatment, the values are rather close to 2.

Table 3. Improvement ratios for the pressuremeter parameters.

Parameter	E_{m1}	E_{m2}	P_l*
Before injection	7.2 MPa	19.8 MPa	0.92 MPa
After injection	44.7 MPa	96.2 MPa	5.00 MPa
Improvement ratio	6.2	4.9	5.4

LABORATORY TRIAXIAL TESTS

Alluvial deposits were also sampled in the site. Their gradation curves show that the natural alluvial deposits were composed of two types of materials subsequently named deposits 1 and coarser deposits 2. The natural gradation of the deposits was subsequently reduced to a particle size of 10 mm (Fig. 7). The coefficient of uniformity was respectively equal to 2.1 for the deposits 1 and 5.9 for the deposits 2. The mean grain size was respectively 0.41 mm for the deposits 1 and 1.30 mm for the deposits 2.

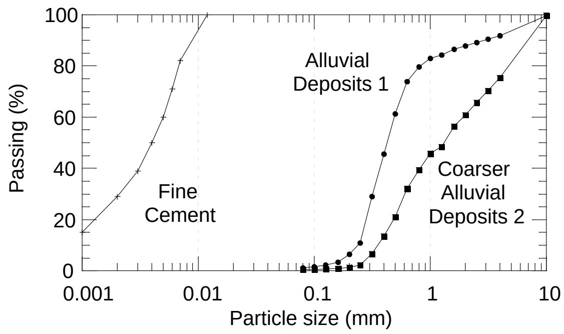


Fig. 7. Gradation curves of the alluvial deposits.

Drained triaxial tests were conducted on the 2 types of saturated and dense alluvial deposits. Results are presented in Table 4 for alluvial deposits 1 and in Table 5 for alluvial deposits 2. e_0 represents the void ratio at the beginning of the shearing, σ'_c the effective confining pressure, E_{sec} the secant modulus determined at a strain level equal to 0.1 %, ν the Poisson's ratio, ψ the dilation angle, φ' the effective friction angle and c' the effective cohesion. More information can be found in (Dano 2001).

The uncemented alluvial deposits, in a dense state, were grouted in the laboratory by the microfine cement grout used in the site. The cement-to-water ratio of the grout was 0.17. The experimental procedure was fully described in (Dano and Derache 2001). Samples of grouted sands have been stored in a temperature-controlled and humidity-controlled room for at least 3 months. This period is necessary for the setting of the cement.

Drained triaxial tests were carried out on these grouted samples, 200 mm in height and 100 mm in diameter. As previously done for uncemented deposits, a conventional interpretation of the test results was conducted to identify the parameters of the linear elastic perfectly plastic model assuming a Mohr-Coulomb yield function. The results are reported in Table 6 for the grouted alluvial deposits 1 and in Table 7 for the grouted alluvial deposits 2.

Table 4. Triaxial test results on unbound alluvial deposits 1.

Test	e_0	σ'_c	E_{sec}	ν	ψ	φ'	c'
		kPa	*MPa*		*deg.*	*deg.*	*kPa*
1	0.480	100	46	0.16	14.6		
2	0.493	200	71	0.12	9.8	36.6	0
3	0.469	400	139	0.22	8.9		

Table 5. Triaxial test results on unbound alluvial deposits 2.

Test	e_0	σ'_c	E_{sec}	ν	ψ	φ'	c'
		kPa	*MPa*		*deg.*	*deg.*	*kPa*
1	0.399	100	47	0.23	8.9		
2	0.382	200	68	0.20	8.3	37.8	0
3	0.368	400	110	0.22	7.8		

Table 6. Triaxial test results on grouted alluvial deposits 1.

Test	σ'_c	E_{sec}	ν	ψ	φ'	c'
	kPa	*MPa*		*deg.*	*deg.*	*kPa*
1	100	208	0.10	28.9		
2	200	151	0.20	24.9	45.6	218
3	400	256	0.23	20.8		

Table 7. Triaxial test results on grouted alluvial deposits 2.

Test	σ'_c	E_{sec}	ν	ψ	φ'	c'
	kPa	*MPa*		*deg.*	*deg.*	*kPa*
1	100	206	0.21	30.4		
2	200	213	0.23	29.1	44.7	313
3	400	279	0.19	21.4		

The results presented in Tables 4, 5, 6 and 7 show that grouting involves an improvement of both stiffness and strength properties of the alluvial deposits. The value of the secant modulus at a strain level of 0.1 % is increased by a factor of 1.5 at least. This ratio decreases with the confining pressure. It can reach a value of 4 at low confining pressures. The values of the Poisson's ratio of the uncemented deposits or the grouted deposits are typical ($0.1 < \nu < 0.3$) but the effect of the grouting on this parameter is not important.

Grouting mainly induces a cohesion which value depends on the cement-to-water ratio of the grout (Biarez et al., 1998). A cohesion of 200 kPa at least was found here. This value has to be taken into account in structural design in order to optimize the costs of supports.

Finally, the values of the friction angle and of the dilation angle are increased. For the friction angle, it is probably due to a compaction of the soil mass during grout injection. For the dilation angle, high values of the volumetric strain rate in the plastic domain for grouted sands mean that the main mechanism is a damage of the cementitious bonds rather than slipping and rotation of sand grains as in the case of uncemented sands. Thus, for grouted sands, failure occurs with vertical cracks due to damage.

CONCLUSIONS

The injection of microfine cement grout clearly increases the mechanical properties of alluvial deposits and allows to excavate the soil ensuring the safety of the workmen, of the excavation machines and by avoiding damage to the buildings. However, the implementation of these recent high-performance grouts requires to even precisely control the rates of injection and the parameters of injection in order to carry out a treatment of high quality without generating disturbance of the surrounding structures.

The improvement of the mechanical properties is shown through the interpretation of pressuremeter tests carried out before and after grout injection and through the results of triaxial tests performed on laboratory grouted samples. Grouting involves a clear increase of the stiffness and a cohesion that should be taken into account in structural design.

REFERENCES

Biarez, J., David, H., Gouvenot, D., Hicher, P-Y., Tailliez, S. and Varjabédian, M. [1998]. "Comportement mécanique des sols granulaires traités par injection", Revue Française de Génie Civil, Vol. 2, No. 2, pp. 255-295.

Cambefort, H. [1992]. "Pression et débit d'injection avec le tube à manchettes", Annales de l'Institut Technique du Bâtiment et des Travaux Publics, No. 500, pp. 93-111.

Dano, C. [2001]. "Comportement mécanique des sols injectés". Ph.D. Thesis, Ecole Centrale Nantes & Université de Nantes (France).

Dano, C. and Derache, N. [2001]. "Grout injection in the laboratory", Landmarks in Earth Reinforcement, International Symposium on Earth Reinforcement, Fukuoka (Japan), pp. 21-26, Ochiai et al. (eds).

Dano, C., Hicher, P-Y. & Tailliez, S. [2003]. "Engineering properties of grouted sands". Accepted for publication in Journal of Geotechnical and Geoenvironmental Engineering.

Filliat, G. [1981]. "*La pratique des sols et fondations*". Le Moniteur (eds.).

Le Moniteur du BTP [1999]. "Meteor fait route vers Saint-Lazare", No. 4984, pp. 80-81.

Selosse, C. [1992]. "Meteor : une nouvelle ligne de métro entièrement automatique", Tunnels et Ouvrages Souterrains, No. 113, pp. 261-270.

Tailliez, S. [1998]. "Etude expérimentale du comportement mécanique des sols granulaires injectés". Ph.D. Thesis, Ecole Centrale de Paris (France).

Vergnaux, H. [2000]. "Meteor à Saint-Lazare", *Proc. AFGC Coll.*, INSA Lyon (France), pp. 19-26.

French Standard NF P 94-110-1 : Sols : Reconnaissance et essais – Essai pressiométrique Ménard – Essai sans cycle

French Standard NF P 94-110-2 : Sols : Reconnaissance et essais – Essai pressiométrique Ménard – Essai avec cycle