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Design of jet-grouting for tunnel waterproofing

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ABSTRACT: A case history in Barcelona is described where a tunnel was excavated by traditional methods below an active railway line through a formation with lenses of water-bearing granular material. To avoid the possibility of sudden collapses a massive jet-grout treatment was applied. The treatment took several forms. Subvertical double and triple-fluid injection was applied whenever possible. Sub-horizontal monofluid canopies and slabs executed from within the tunnel were however required in zones where no vertical access was possible. This communication focuses on the later type of treatments and gives an overview of the design tools that were applied. These comprised the execution of several large trial fields and the systematic application of a probabilistic framework for design.

1 INTRODUCTION

It is a common paradox that surface transport infrastructures are usually more needed where there is less space available for them. Underground developments are then inevitable, be that by means of tunnels and excavations, or through refurbishment and adaptation of existing infrastructures, requiring new foundations and/or extensions. Wherever urban infrastructure construction activities take place there is a large likelihood that one form or another of ground improvement would be required. Ground improvement allows construction to proceed where otherwise it would be impossible, because some relevant required soil property (strength, stiffness, permeability) is missing.

Jet grouting is a technique where a high-pressure injection of mortar, with or without other accompanying fluids (water, air), impacts the ground in a borehole. In most cases the original ground is thus eroded, mixed with the mortar and, in fluid form, partly evacuated to the surface (resulting on what is called "spoil"). The remaining soil-cement mixture sets "in situ", resulting on a stiffer, stronger, more impermeable and less ductile material than the original soil. The injection equipment is displaced along the borehole, thus creating a body of treated soil of columnar shape. Several such injections are combined to create the desired shape of treated soil: slabs, arches and walls are common examples.

The basic reasons for jet-grout success are clear: of all the means of ground injection, jet is not only the fastest procedure, but is also the only one suitable for all improvement purposes (strengthening, stiffening, impermeabilization). In tunnelling operations jet grouting has the added attractive of access versatility: treatments can be executed from within the tunnel, from the surface of the ground or, eventually, from a side shaft.

This paper describes a case history in Barcelona where a variety of jet-grout treatments were performed to help drive a tunnel through water-bearing sediments under an active railway line. After giving the geotechnical background of the problem as well as a brief account of the construction procedures applied, the paper focuses on the design tools employed. Several results from large-scale "in situ" tests are described; afterwards an example of a probabilistic approach employed in design is outlined. Finally a brief summary of the observed treatment outcomes is also given.

2 PROJECT DESCRIPTION

2.1 *Location*

The new High Speed Railway Link between Madrid and Barcelona was open to traffic in February 2007. The Southern entry of the new line into the city of Barcelona follows the trace of the historical railway entrance, heading towards Sants railway station (Fig. 1). This railway entrance crosses the city of Hospitalet de Llobregat, just south of Barcelona. Hospitalet is currently involved in a major urban redesign operation in which the suppression of the railway barrier plays a major role. For this reason

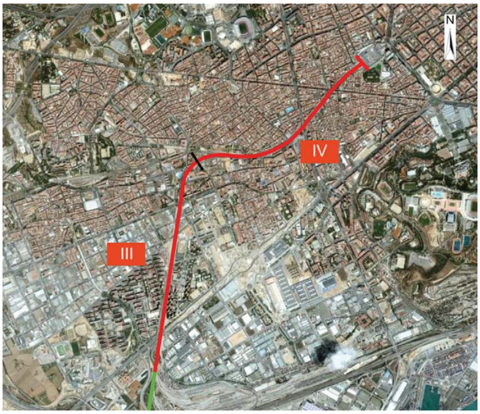

Figure 1. Plan view of the High Speed Railway Link south entrance to Barcelona.

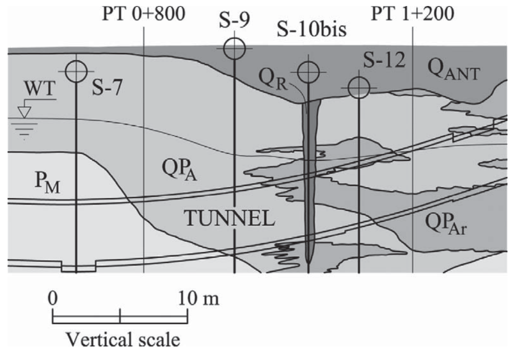

Figure 2. Geotechnical profile alongside the central part of the tunnel. P_M stiff tertiary marl; QP_A medium stiff clay-dominated quaternary deposits; QP_{Ar} sand-dominated quaternary deposits; Q_R gravel-dominated quaternary deposits; Q_{ANT} made ground. Spacing amongst vertical reference lines 20 m; spacing amongst horizontal reference lines 2 m.

the design of the new high speed link located it below ground well before its arrival at Sants.

In plan, the new link follows closely the current tracks, so much that the final 2 km stretch (marked IV in Fig. 1), had to be excavated just below them. The railway line south of Sants is the main medium and long-distance line out of Barcelona, doubling also as a very busy commuter line. Therefore only limited railway traffic restrictions were possible and the new entry had to be built below the still active old lines.

2.2 *Ground conditions*

The tunnel is located on the alluvial plain of Barcelona. The western limit of this geomorphological unit is given by the Collserola mountain range, whereas the southern and northern limits are given by the deltaic systems of two river mouths (Llobregat and Besos, respectively). The alluvial plain is formed by Quaternary deposits overlying a Tertiary substratum. The main unit of the Quaternary deposits in the tunnel area is a brown red clay that includes some carbonated levels and thin sand layers (QP_A in Fig. 2). This unit is crossed by a network of paleochannels, fossilized remains of a network draining the nearby Collserola range into the Mediterranean. These paleochannels are formed by sands and gravels with a variable clay matrix (QP_{Ar}). Younger, recently active brook deposits are also crossed by the tunnel trace (Q_R). The Tertiary deposits, mostly from Pliocene and Miocene age, are formed by some ochre sands and clays (PA) and blue-grey marine marly clays (P_M). The water table in the area is generally located between 10 and 15 m depth.

2.3 *General tunneling concept*

The main tunnel section had nearly 110 m^2. The longitudinal profile was established to pass below several underground road passages and a tube line that marked its lowest point. The tunnel cover varied between a minimum of 6 m and a maximum of 25 m. In the situation just described the main construction priority was to avoid any risk of sudden ground collapse, however minor. On the other hand, and thanks to an active ballast maintenance program, slowly induced excavation settlement could be easily compensated.

EPB tunnelling is generally the favoured urban tunnelling choice when soft soils are present on a tunnel trace. However, for relatively short tunnels or when access shafts for an EPB are not easily located, traditionally mined excavation might offer an interesting and robust alternative. These two circumstances were present at the tunnel here described. A partitioned section procedure, known as the traditional Madrid or Belgian Method, was then selected as the basic excavation procedure for the tunnel. It is worth mentioning, however, that this method is relatively slow, with median advance rates of 30 m/month per excavation front (Sacyr 2008). To comply with a tight construction schedule this required the execution of 2 intermediate deep access ramps and 3 further deep shafts to open intermediate excavation fronts.

The first 800 m of the tunnel were located either above the water table or within the impermeable, homogeneous PM layer. There, the Belgian method, with occasional forepoling help, was successfully applied (for details, see Deu et al. 2007). The last 600 m of the tunnel were again located mostly above the water table, and a similar construction procedure was applied.

However, in the approximately 400 m corresponding to the central and deepest part of the tunnel, the presence of erratic channels of quaternary deposits with a large granular fraction (QP_{Ar}, Q_R) was coincident with a water table well above the tunnel crown (Fig. 2). There were also close precedents showing that running ground conditions at the tunnel face were a distinct possibility. This was a major cause for concern, and thus there was some consensus that even the small partitioned and braced excavation fronts allowed by the Belgian method did not offer enough guarantee against that kind of failure. Therefore in that part of the tunnel a different construction procedure, relying on a systematic use of jet-grout, was envisaged.

2.4 *Jet-grouting treatments*

Jet-grout was selected as the soil improvement tool that was best suited to avoid the possibility of ground flows towards the excavation. The main function of the treatment was then one of impermeabilization around the tunnel; the natural soil had more than adequate resistance and stiffness to span the tunnel section without excessive deformations (Table 1).

Two types of treatments were applied. In locations where access above the treated zone was possible, vertical or subvertical columns were employed. For this type of treatments powerful two- and three-fluid means of jet injection could be applied, since the spoil might be evacuated from the surface and the geometry of a newly executed column was not unstable. The typical design column diameter for the vertical columns was 2.5 m. The layout included full face sections, where the whole tunnel face was covered by the treatment (Fig. 3a) and sections where the columns where only surrounding the tunnel (Fig. 3b). The purpose of the full face sections was to create longitudinally isolated excavation zones, so as to minimize the extension of hypothetic failures.

Table 1. Geotechnical parameters; average values in parentheses.

Material	Fines %	w %	N_{SPT}	S_u kPa	E MPa
Q_{ANT}	80-9 (50)	(15.1)	(5)	–	7.5
Q_R	86-14 (43)	(16.1)	(48)	–	22.5
QP_A	99-50 (80)	(18.4)	(24)	(100)	22.5
QP_{Ar}	80-5 (30)	(14.5)	(31)	–	40
P_M	23-99 (95)	(20.8)	(34)	(150)	42

However, there were some zones within the treated tunnel section where surface access above the tunnel was impossible. In these zones the excavation was partitioned between heading and bench (Fig. 4). Treatment of the heading created a soil chamber by covering the full section perimeter and closing the full tunnel face ahead of the excavation using sub-horizontal columns. From within the excavated heading followed a full treatment of the bench with subvertical columns. Executing the injection from within the tunnel itself only allowed for single-fluid injection and relatively small diameter columns (around 0.5 to 1 m, see below for details).

These zones executed from within the tunnel were the most critical, both because the treatment execution was particularly risky and because the required number of columns made the procedure very slow. A number of special measures were then taken to guarantee a successful outcome, both at the design stage and at the execution stage. While this paper focuses on design, it is worth mentioning, amongst the execution-related aspects, the

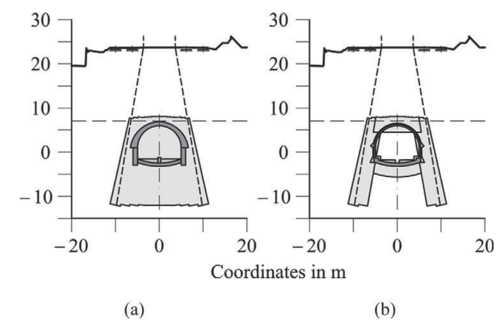

Figure 3. Typical treatment configurations where surface access was available.

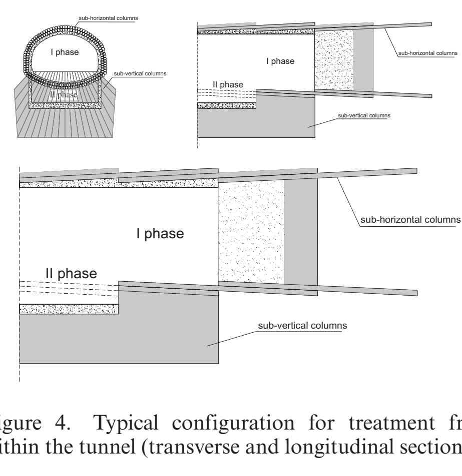

Figure 4. Typical configuration for treatment from within the tunnel (transverse and longitudinal sections).

systematic application of a manually controlled blow-out preventer (Guatteri et al. 2009), a tool that helped to avoid instabilities with the sub-horizontal injections.

3 JET-GROUT DESIGN TOOLS

3.1 *General approach*

The development of jet-grout has been technologically driven, with the relevant science lagging behind. As a result the design, implementation and control of jet-grout treatments has developed on a heuristic basis, lead by practitioners and real-life experience applications. Although this approach has been successful in most cases, a number of high-profile construction problems can be traced to ineffective jet-grout treatments, for instance the Souterrain tram tunnel in The Hague (NL) (van Tol 2004), the Les Cretes tunnel of the Aosta highway in Italy (Croce et al. 2004) and the Kaoshiung Mass Rapid Transport tunnel, in Taiwan (Ishihara 2008).

The effectiveness of jet grouting treatments is dictated by the interaction between technological factors and natural soil properties, the role of which cannot yet be predicted with sufficient accuracy but needs to be evaluated by means of experimental investigations. To prove the efficiency of treatments, execution of preliminary field trials is in fact required by current standards (e.g. ENV 12716 1997). In design cases where the treatment should provide watertightness, continuity of treatment is the most fundamental aspect. Field trials should then focus on the cross sectional dimensions and on the required spacing of contiguous columns supposed to overlap.

Concerning the former point it is worth reminding that, for a given set of injection parameters, column diameters are strongly influenced by natural soil properties and that inhomogeneous subsoil conditions turn into irregularities of column shape and discontinuity of the waterproofing barriers (e.g. Croce & Modoni 2005). With regard to the second aspect, even if a direction is specified for columns axes, deviations are possible depending on how accurately the position and inclination of injection tools is controlled. Generally, even when low tolerances are prescribed, unpredictable misalignments occur, particularly for longer columns, due to the self weight of perforation bars or to inaccurate driving of the supporting arms.

Bearing in mind these goals, different field trials were specifically devised in the presented work, some to find relations between column diameters and treatment parameters, others to quantify the effects of imperfect drilling operations.

All the information collected in the trials was then fed into probabilistic design models to explicitly take into account the inherent variability of the method and deal economically with the uncertainty of the treatment outcomes.

3.2 *Vertical column trials*

A first field trial was located within one of the intermediate deep access ramps to the tunnel. The ramp itself was roughly at the middle of the stretch were the treatment was required, however, specific stratigraphy at the ramp site was dominated by the more clayey quaternary levels (QP_A).

Two parallel rows of ten vertical columns each (identified as A and B in Fig. 5) were injected for a length of about 15 m starting at 2 m below the surface. Four columns of each group (A1, A8, A9, A10 and B1, B8, B9, B10) were positioned with spacing larger than the expected diameter (0.5 m) to provide data in diameter variability and material properties. The other six were intended to overlap being injected with variable axes span (0.30 m for A2, A3, A4 and B2, B3, B4 columns, 0.45 m for A5, A6, A7 and B5, B6, B7).

All columns were executed with a single-fluid system, like the one to be applied later within the tunnel. Columns A and B shared some injection parameters like nozzle diameter (3 mm) and rotation velocity (12 rpm). However, they did differ in several parameters (Table 2): withdrawal

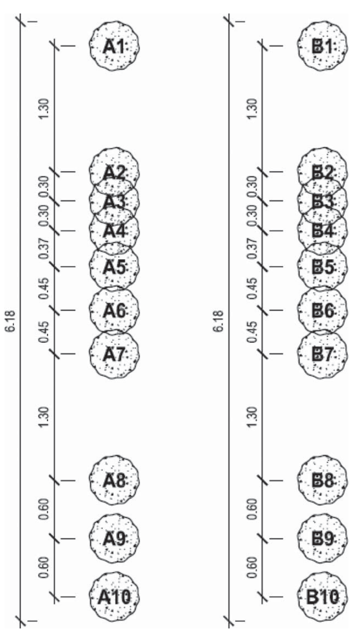

Figure 5. Plan view: vertical column trials.

Table 2. Injection parameters of vertical trial columns.

Column type	v (cm/min)	p (bar)	V_0 (m/s)	ρ (t/m³)	W/C
A	37	350	200	1.59	1/1.2
B	35	400	212	1.52	1/1

Table 3. Measured treatment outcomes on vertical trial columns.

Column type	$\bar{D}$ (m)	COV (D)	q_u (MPa)	COV (q_u)	E_{50}/q_u
A	0.38	0.14	11.6	0.26	573
B	0.48	0.12	7.9	0.38	576

speed, v; injection pressure, p (and, consequently, nozzle velocity, V_0); water/cement ratio, W/C (and, consequently, grout density, ρ).

The treatment parameters for both columns had been initially proposed by the contractor, whose experience-based average diameter estimate for both types of treatment was equal to 50 cm.

It seemed clear, however, that other predictive approaches to estimate column diameter were also worth trying. For instance, the treatment input energy per unit length could be computed following Croce & Flora (2000) as

$$E_n = \frac{8\rho V_0^3}{\pi v d^2} \tag{1}$$

In the case of the vertical trial columns type A columns had $E_n = 7.36$ MJ/m, whereas type B columns had $E_n = 9.06$ MJ/m. It was then expected that type B columns would result in higher column diameters and, because of their higher W/C ratio, in smaller strengths.

Several treatment outcomes were measured in the field trial. On the one hand, continuous rotary coring of four columns provided samples at every meter on which to measure strength (unconfined compression, q_u), stiffness (on-sample deformation measurements, E_{50}) and density of the treated material. On the other hand the top four meters of all the trial columns were examined while excavating the station access ramp and cross sectional dimensions were taken by measuring the length of cemented soils samples cored by horizontal drilling. A summary of the results obtained through these measurements is included in Table 3.

The average diameter of columns B was higher than the average diameter of columns A, in agreement with the computed energy input (1). The strength of columns B was less than that of columns A, again as expected from the W/C relation. Strength variability was higher than geometric variability. The latter was also easily quantified by means of the distributions of column diameters (Fig. 6).

It was made clear that, while detrimental for the mechanical properties of the treated soils, a larger amount of water in the injected mix enhanced the erosive action of the jet and resulted in larger

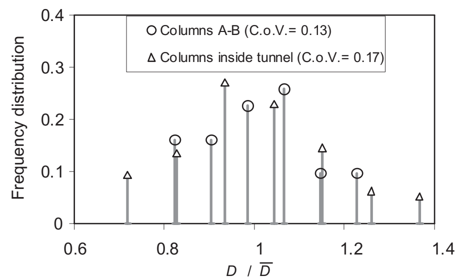

Figure 6. Distributions of column diameters measured at the vertical field trials.

column diameters. For the case here contemplated the reduction in strength was immaterial, whereas the increase in diameter was clearly beneficial.

A second consequence of this first field trial was to show the benefits that might follow from a rational approach to diameter prediction. A more elaborated predictive formula, following from Modoni et al. (2006), was then proposed and validated with a later round of single-column field trials. It is not possible here to give a detailed account of this latter round of single-column trials. However, it is worth mentioning that the sub-horizontal columns that were finally adopted as a basis for the treatment execution were injected with larger nozzle diameter (3.5 millimeters), with higher water-cement ratio (1.5/1), and with a lower lifting speed (20 cm/min) than those originally proposed.

3.3 *Full-scale heading trial*

A second, more ambitious, field trial took place to test the execution of the treatment conceived for the heading section of the tunnel. An almost full-scale (80%) heading section was built with a similar geometry and procedure as that intended in the tunnel (Fig. 4) from a dedicated 15 m deep excavation.

For reasons of space availability the trial took place at some 8 km to the south of the tunnel location. This meant that instead of on the Barcelona alluvial plain the trial was fully within the Llobregat delta plain. The geotechnics of the Llobregat delta are rather different from that of the alluvial plain, with a medium dense sand layer overlaying a soft sandy and clayey silt and water table close to the surface (e.g. Gens et al. 2011). The trial heading section was mostly performed within the silt layers and had 9 m soil cover above the crown, mostly in sands. The geotechnical conditions for the jet-grout execution at the trial site were then clearly unfavorable when compared to those prevailing at the tunnel site. However, it was felt execution under such conditions will clearly prove the potential and/or show the limitations of the proposed construction technique.

The tunnel contour canopy consisted of two concentric rows of 151 diverging columns, 12 m long and with 0.50 m expected diameters, injected after initial perforation of a previously created thick jet grouting supporting wall. A 3 m thick plug was created at the deeper end of the tunnel, made of 174 columns of 0.80 m diameter, located on six concentric circles with 0.50 increasingly larger radii.

A net of vibrating wire piezometers and settlement plates was installed above and around the trial tunnel for monitoring purposes. Treatment execution parameters were continuously recorded and, in several instances, measurements of column axis inclination were also performed.

Here only a few results from this trial will be commented; some results can also be found in Guattieri et al. (2007). During jet injection asymmetrical settlements and heave were recorded at ground level (about 20 millimetres maximum settlement on one side and 10 mm heave on the other side, Fig. 7). These movements were likely related to a pressure build-up induced by injection. However, water pressure measurements were almost constant in all the piezometers.

While this outcome was hardly welcome, it was not very discouraging. On the one hand, settlement issues were of limited concern (although the possibility of brusque jet-induced motion limited the temporal window for treatment in certain cases). On the other hand, the construction of the railway tunnel had to be performed at considerably larger depths below a thicker soil cover. Even in the absence of a detailed model, it was clear that the movements that were induced at the trial were well above those to be expected on site.

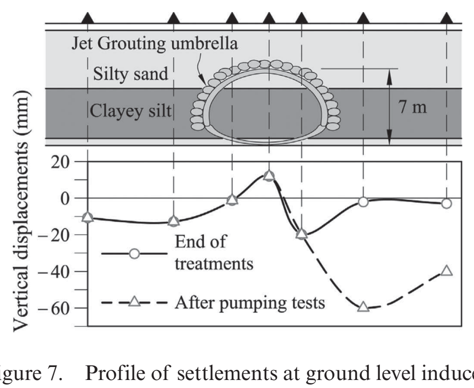

Figure 7. Profile of settlements at ground level induced by execution of treatments and extraction of water from the tunnel chamber.

After conclusion of treatments, a test on the waterproofing capacity of the jet grouted soil was performed, by inserting a number of pipes at the front of the tunnel and by extracting, with a vacuum pump, the water present in the inner chamber. Pumping lasted two days and the amount of water extracted was well below the estimated soil water content (less than 5%). The poor response of the pipes was likely due to clogging of the pipe protective filters by silt particles. During this test, a sudden drop of water head was measured in a piezometer located on the right side of the tunnel. Moreover, this was immediately followed by a funnel shaped collapse that emerged at the ground surface in a position close to the contact of the canopy and the extreme plug of the tunnel (see settlement profile of Fig. 6).

Explanation for this result can be found in the consolidation of soil inside the tunnel chamber activated by the extraction of water and by the lack of bending capacity of jet grouting. It is worth mentioning that jet grouting columns were not reinforced and that collapse occurred at the attachment of columns to the end plug, i.e. where the flexural moment reaches its maximum values. The amount of consolidation settlement to be expected at the tunnel site was much less and therefore it was concluded that passive (i.e. without pumping) dewatering of the soil chamber could be applied safely at the tunnel.

Also particularly interesting are the measurements of column axes inclination, which are rather infrequently performed in jet grouting treatments within tunnels. These were measured using one inclinometer mounted on the injection mast (Jean Lutz 2006). The two components Δy and Δz of deviation from the theoretical axis direction, plotted for five different columns in Figure 8, show that there is a common systematic downward trend. Deviations increase significantly with column length, and their absolute values are non-negligible.

The deviations could be explained as a result of the bending of perforation bars due to their self weight (which, for the measured columns, was also increased by the inclinometer). Partial compensation can be provided by an initial upward inclination of the perforation axis, as shown by column S01-15E.

Again, and despite is large magnitude, this systematic deviation was not a major cause

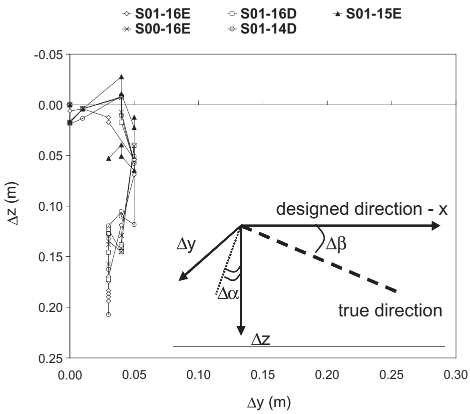

Figure 8. Deviation of column axes from their theoretical position (deviation projected on a plane orthogonal to column axis).

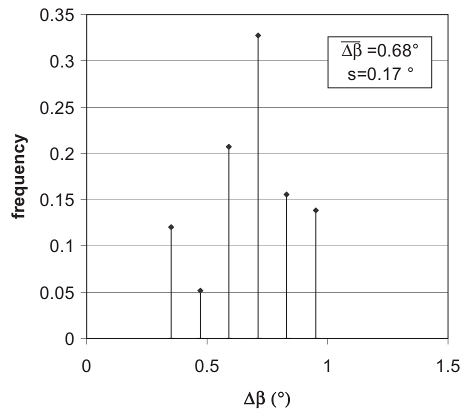

Figure 9. Frequency distribution of the inclination deviation angles Δβ.

for concern. The continuity of a jet grouting canopy is not particularly sensitive to systematic deviations. On the other hand a random scattering of column directions would be very detrimental for the same objective. The deviation data were analyzed to quantify the random scatter in direction. The average direction of columns was calculated and the two angles (azimuth Δα and divergence Δβ) expressing the deviation of columns axes from this mean trend were evaluated. The plot in Figure 9 shows that a bell shaped frequency distribution can be clearly inferred for the divergence angle Δβ, while Figure 10 shows that the distribution of azimuth angle Δα is approximately uniform.

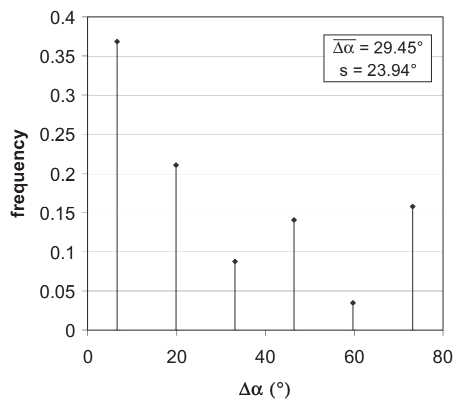

Figure 10. Frequency distribution of the azimuth deviation angles Δα.

3.4 *Probabilistic approach*

The continuity of different jet grouting structures (canopies, plugs and vertical walls) was routinely analyzed during the project by developing probabilistic methods specifically customized for each structure. An analysis developed for the perimetral canopies of a tunnel heading (Fig. 4) is here reported to illustrate these calculations.

The probabilistic analysis of this structure was performed selecting two sections, at the extremes of the overlap section between two consecutive canopies (see Fig. 11a). Based on the field trial results as well as on later quality assurance measurements, a probabilistic distribution was assigned to column diameters and column axes deviation. In particular, a Gaussian distribution has been considered for column diameters with a mean value equal to 0.75 m and a coefficient of variation equal to 0.17. This value of average diameter has been chosen as equal to the minimum value estimated with the adopted set of injection

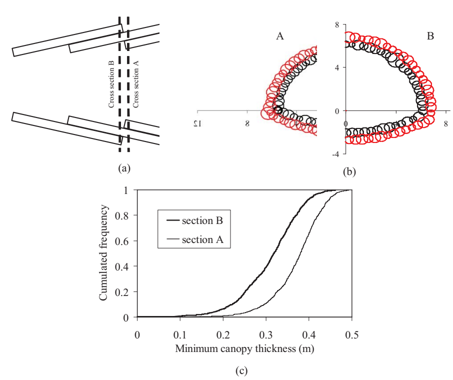

Figure 11. Probabilistic analysis of the continuity of tunnel canopy (a. longitudinal section; b. example of cross section simulation; c. cumulated frequency distributions of the minimum thickness of the canopy).

parameters for sandy soils, considering these latter as the most critical in case of piping.

Random divergence of column axes from their designed position has been modelled by considering angles $\Delta\alpha$ and $\Delta\beta$ respectively distributed with uniform and Gaussian laws. For the latter a nil average value and a standard deviation of 0.68° were assigned.

A Monte Carlo simulation technique was applied, generating one thousand cross sections consistently with the adopted probabilistic distributions (one example of simulation result is reported in Fig. 11b). The minimum canopy thickness was then calculated in each case to obtain a statistical sample of this simulation outcome. The cumulative frequency distribution of this variable (Fig. 11c) represents the discontinuity hazard of this particular structure.

4 JET GROUT PERFORMANCE

The field trials and probabilistic methods described above were useful to generate confidence on the construction team about the issue of one construction operation that was initially perceived as extremely risky. About 100 m of the main tunnel and access galleries were thus treated in full section or just at the bench from within the tunnel. The advance was performed without any incident.

However, and despite the optimization allowed by the probabilistic design, the complexity of the subterranean treatments still resulted in very slow excavation procedure, with median advance rates below 1 m/day. An effort was then made to allow even some partial surface access above most of the treated tunnel length and therefore vertically executed treatments were applied in most of the treated section. Lacking time to perform the same extensive trials that preceded the subhorizontal treatments, the uncertainty on the jet grouting outcomes for vertical treatments was higher, particularly for column diameter. Hence ancillary pumping to lower the water pressure on the treatment walls was introduced as an auxiliary measure to increase the safety of excavation. The combination of partial pumping and vertical jet-grout chambers was successful in that the tunnel was excavated without incidents at a median advance rate two to three times higher than that allowed by subhorizontal treatment.

5 CONCLUSION

The reported case history has proved that the design of jet grout structures can largely benefit from a rational approach that includes experimentally based uncertainty within a probabilistic framework. The measurements taken and observation made at the field trials in this project contain useful information for future designs in similar conditions.

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