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Design and construction of a sewer tunnel in difficult site conditions

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ABSTRACT: The paper deals with a sewer tunnel, recently excavated in the city of Palermo. The tunnel layout is close to existing urban facilities and residential buildings. The geotechnical profile is quite variable and the soil cover ranges between 7 and 9 m. The groundwater level is located at mid height between the invert arch and the vault of the tunnel. Excavation support was granted by means of the canopy technique, employing micropiles and jet grouting. A detailed monitoring program was also organized and tunnel construction was followed step by step allowing for proper design changes, according to the subsoil conditions met during excavation. The case history is reported by describing the tunnel layout, the geotechnical profile and the monitoring plan. Recorded settlements of the existing structures are analyzed, considering the influence of the geotechnical properties of the subsoil as well as the peculiar construction sequence of tunnelling.

1 INTRODUCTION

Tunnel construction in urban areas is generally a very difficult task for several typical reasons that may be listed as follows:

i. the presence of weak soils (including made land), characterized by low shear strength, which may compromise the stability of the underground excavation
ii. the occurrence of seeping water, coming from local aquifers or water lines leakage, which may induce soil piping along the cavity
iii. the excavation process which generates soil deformations that may induce excessive settlements of adjacent or overlying buildings
iv. the crossing of other underground facilities which may interfere with digging activities.

It is well known that tunneling through weak soils may result in face and vault instability, unless proper soil support is provided during excavation. This goal can be reached either by using a TBM or by installing a set of reinforcing elements, forming a sort of canopy along the cavity contour and behind its face.

However, the use of a TBM becomes technically and economically feasible only for long tunnels with ample curvature radii, circular cross section and constant diameter.

On the contrary, the canopy technique can be readily used for short tunnels, non circular cross sections and abrupt changes of direction. The canopy technique can also be customized in order to account for variable soil properties, by making proper use of the different soil reinforcing techniques. Design features can also be modified in real time to meet unforeseen situations, such as unexpected soil conditions, undetected underground facilities or excessive surface settlements, as it frequently happens in urban areas. In such cases, it may be appropriate to apply the observational method (Nicholson et al., 1999; Patel et al., 2007; Croce, 2010) in which the design is reviewed during construction as allowed by the Eurocode EC7.

In the following, the typical features of the canopy technique are first recalled, considering the available soil reinforcing methods and their implementation in the tunneling procedure. This technique was successfully employed in order to excavate a sewer tunnel, built in difficult site conditions in the city of Palermo.

The case history is then reported in some detail by describing the tunnel layout, the geotechnical profile, the design features and the monitoring plan. Finally, recorded settlements of the existing structures are analyzed, considering the influence of the geotechnical properties of the subsoil as well as the peculiar construction sequence of tunneling.

2 CANOPY TECHNIQUE

The canopy technique is based on the use of soil reinforcing methods which provide stability of the tunnel contour during excavation. Such canopy is obtained by means of steel micropiles and/or jet grouting columns which are installed in advance with respect to excavation. Moreover, for very weak soils, the stability of the tunnel face may be granted by diffused reinforcement of the soil behind the face, which can be pursued either by jet grouting treatments or by the insertion of fibreglass elements (bars or tubes).

During excavation, the face reinforcing components are removed together with the soil, while the contour elements are progressively supported by setting up steel ribs and shotcrete in order to complete the provisional lining of the tunnel. The final lining, made of reinforced concrete, can then be installed at a later and more convenient time.

In principle, each reinforcement technique is best suited for particular soil types and seepage conditions. Jet grouting treatments are generally preferred for sandy soils, providing larger columns which can be set aside forming a sort of supporting arch (Fig. 1A). The latter can provide also waterproofing, when properly dimensioned, if pore water pressures are not too high.

However, for fine grained materials jet grouting is not very effective and so for clayey soils it is usually more convenient to choose micropiles (Fig. 1B). In particular, steel micropiles are used for the tunnel contour while fibreglass bars or tubes are employed for the face reinforcement, since they can be easily truncated during excavation. Finally, in difficult cases or doubtful soil conditions, it may be useful to combine jet grouting and micropiles in order to form a sort of reinforced jet column.

Whatever method of soil reinforcement is chosen, the canopy technique is characterized by a construction sequence which proceeds by subsequent spans, as depicted in Figure 2. The span length is usually comprised between 6 and 10 m. For each span

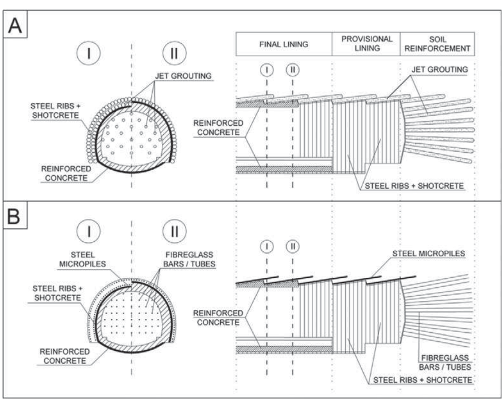


Figure 1. Schematic drawings of the canopy technique: A—reinforcement by jet grouting columns, B—reinforcement by steel micropiles and fibreglass bars or tubes.

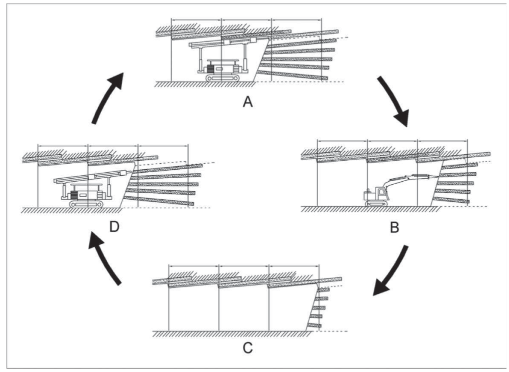


Figure 2. Tunnelling sequence by the canopy technique: A—reinforcement of tunnel contour by jet grouting and/or steel micropiles, B—excavation, C—excavation completed, D—face reinforcement by jet grouting and/or fibreglass bars or tubes (optional).

there are two main construction phases: treatment along the tunnel contour (Fig. 2A) and soil digging (Fig. 2B). After the excavation is completed (Fig. 2C) it is possible to consolidate the tunnel face (Fig. 2D) when this treatment is needed.

3 TUNNEL DESIGN

The sewer tunnel, which is still under construction in the city of Palermo, is almost 5 km long and is characterized by frequent changes of cross section and abrupt variations of direction, in order to meet peculiar hydraulic requirements and to conform to the urban features of the city. The tunnel stretch considered in the present paper is about 350 m long and is located downtown Palermo under a busy street, named “Corso Re Ruggero” (Fig. 3). This street is bordered on one side by a subway line which was previously built by cut and cover.

Therefore, on this side, the tunnel runs along a sheet pile wall (Fig. 4). On the opposite side of the tunnel, three different conditions are met. Along the first half of the tunnel stretch, there are some residential buildings (Fig. 4a) of various dimensions and characteristics, such as masonry and reinforced concrete. They are all placed on shallow foundations, with the exception of one single edifice founded on reinforced concrete piles.

In the second half of the tunnel stretch the street is bordered by an old masonry retaining wall, standing over a city park (Fig. 4b). Another relevant structure is the subway ticket office, which is connected to the subway line through a pedestrian underpass (Fig. 4c). This particular situation is met in the middle point of the tunnel stretch.

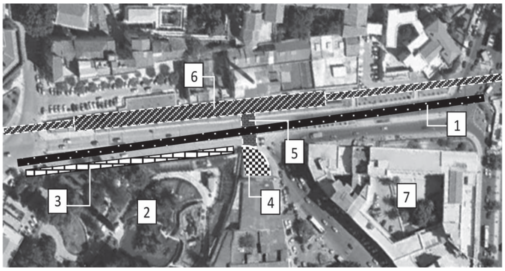


Figure 3. Plan view. 1) Tunnel layout 2) Park 3) Retaining wall 4) Subway Ticket Office 5) Pedestrian underpass 6) Subway line 7) Buildings.

For each span the cross section of the excavation ranges between the following values: width 5.10–5.40 m, height 5.40–6.00 m. The soil cover, taken as the distance between ground level and tunnel crown, is comprised between 7 m and 9 m.

The tunnel intersects different geologic formations. At the bottom, just below the invert arch, there is a continuous thick stratum of dense silty sands. On top of this stratum there are several layers of calcarenitic rocks, characterized by a very variable degree of cementation. The overall thickness of the calcarenitic layers ranges between 3 and 11 m. Moreover, the continuity of the calcarenitic formation is interrupted by alluvial sediments deposited by the ancient river Kemonia which bordered the city of Palermo many centuries ago. The grain size distribution of these alluvial soils is reported on Figure.5.

Finally, on top of the natural soils, there are relevant layers of made land i.e. materials of anthropic origin (bricks, pottery, fragments, transported soil, etc.). The groundwater level is located at the depth of 10 m, practically at mid height between the invert arch and the vault of the tunnel.

Several continuous borings were performed, at the design stage, providing the elements for drawing the geotechnical profile along the tunnel. However, due to the remarkable lithological variability of the subsoil, it was not possible to detect the contact between the alluvial materials and the calcarenitic formation.

Moreover only few undisturbed samples of calcarenite were retrieved, since these materials are very fragile and are thus subjected to extensive breakage during mechanical boring. However, it was well known that the degree of cementation of the calcarenites is very variable, generally low, and some times negligible due to their peculiar petrographical features (Canzoneri et al., 2002).

Design was thus accomplished by dimensioning three types of canopies, and each type of canopy was associated to a typical stratigraphical sequence (Fig. 6). In particular, steel micropiles

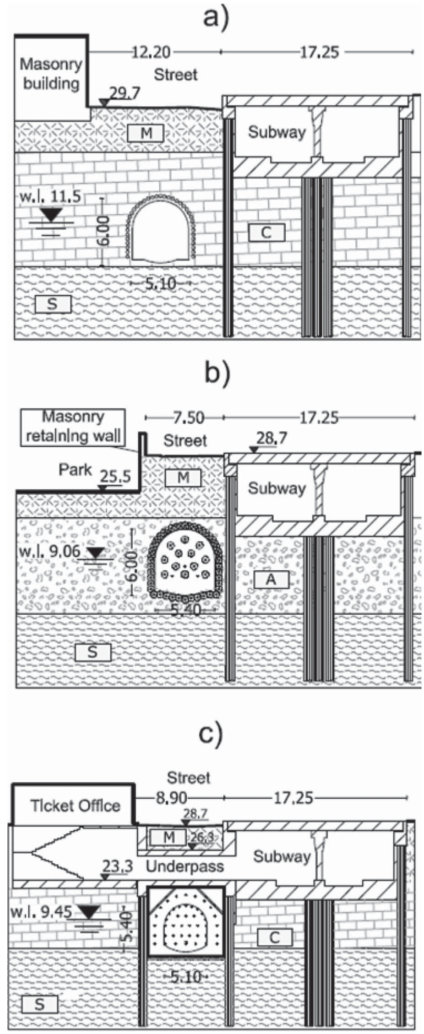


Figure 4. Typical cross sections: a) Micropiles Canopy b) Jet grouting Canopy c) Steel frame. (M made land; C calcarenites; A alluvial soils; S silty sands; W.L. water level).

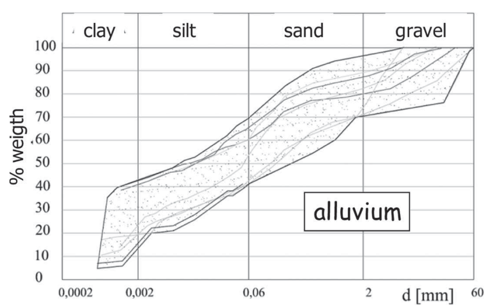
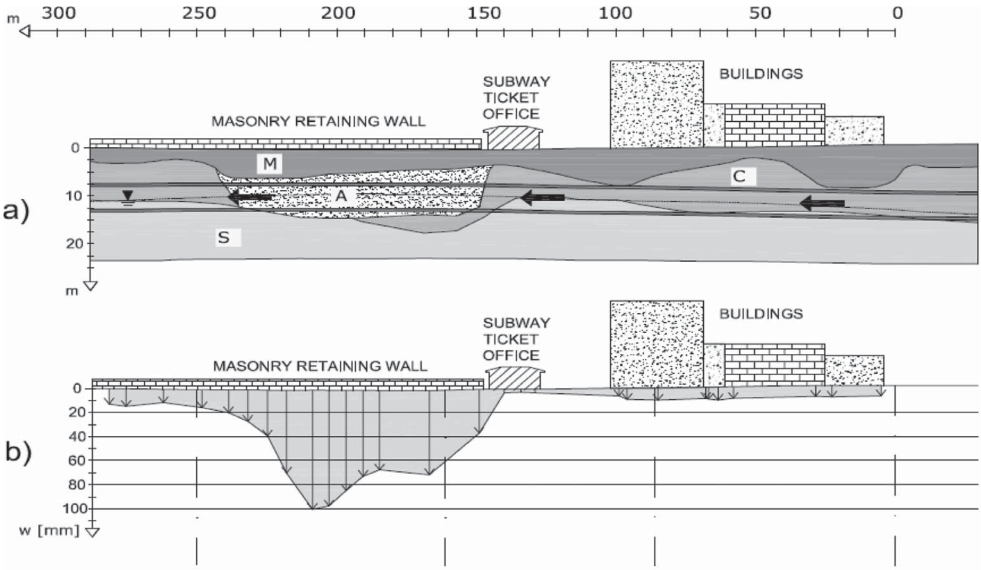


Figure 5. Grain size distribution of the alluvial soils.



c)

1.a	2	1.b	1.a	CANOPY TYPE
22 STEEL MICROPILES (d=0,06m)	23 REINFORCED JET COLUMNS (d=0,6m)	22 STEEL MICROPILES (d=0,06m)	22 STEEL MICROPILES (d=0,06m)	VAULT
———	12 REINFORCED JET COLUMNS (d=0,6m)	10 STEEL MICROPILES (d=0,06m)	———	PILLARS
———	15 REINFORCED JET COLUMNS (d=0,6m)	———	———	INVERT
———	12 REINFORCED JET COLUMNS (d=0,6m)	22 FIBREGLASS MICROPILES (d=0,06m)	———	FACE
3 HORIZONTAL DRAINS (d=0,09m)	3 HORIZONTAL DRAINS (d=0,09m)	3 HORIZONTAL DRAINS (d=0,09m)	3 HORIZONTAL DRAINS (d=0,09m)	DRAINAGE

Figure 6. Tunnel Construction: a) Geotechnical Profile; b) Final settlements; c) Canopy Types (M made land; C calcarenites; A alluvial soils; S silty sands).

were prescribed for supporting the vault in the calcarenitic formation, considering that the cover was mainly composed by made land (canopy type 1.a). The micropiles were extended down along the tunnel pillars, where it was expected to intercept the dense silty sands (canopy type 1.b). Jet grouting was prescribed instead for the alluvial soils of river Kemonia (canopy type 2). Horizontal drains were prescribed for all the tunnel spans. A special steel frame was devised for crossing the pedestrian underpass, where excavation was essentially supported by the pre-existing reinforced concrete slab and piles (see Fig. 4c).

4 CONSTRUCTION AND MONITORING

Tunnel excavation was recently completed, under "Corso Re Ruggero", but the final concrete lining has not yet been cast in place at the time of writing. During excavation a detailed monitoring program was carried on and the tunnelling process was followed step by step for each construction span, having an average length of 9 m.

This observational procedure allowed for proper design implementation of the design canopies, according to the subsoil conditions met during excavation. In particular, the following monitoring activities were carried on:

i. lithological observation of the excavation face and evaluation of water drainage
ii. sub-horizontal borings to check subsoil conditions for subsequent spans
iii. deformation measurements of the provisional lining by means of convergence bolts
iv. deformation measurements of the steel ribs by strain gauges
v. direct stress measurements on the steel ribs by means of load cells
vi. topographical observations of the ground surface and of the existing buildings
vii. inclinometric, assestimetric and piezometric measurements from vertical borings.

All the above measurements provided useful contribution for checking the design and for prescribing the most appropriate solutions for each excavation span, according to the principles of the Observational Method. However, in practice, lithological and topographical observations proved to be the most valuable means for construction control (Leta et al., 2007). A typical stratigraphical condition observed on site is shown by a picture taken at the tunnel face (Fig. 7).

Topographical data were retrieved and processed in real time, allowing for continuous checking of surface settlements. However, measurements of bench marks placed on the road surface were affected by traffic disturbance which induced excessive scattering. On the contrary, continuous settlement data were recorded from the bench marks placed on the existing buildings and on the masonry retaining wall located along the park. These data provided the most useful mean for monitoring the near by structures.

The final settlements, measured after tunnelling completion, reached quite different values (Fig. 5b), corresponding to the various subsoil conditions (Fig. 5a) and canopy types (Fig. 5c).

Buildings settlements were always less than 10 mm and did not cause any problem to the existing structures. This behaviour was related to the over all stiffness of the calcarenitic layers. On the other hand, the settlements of the retaining wall reached a maximum value of 100 mm causing relevant tilting towards the park. Such larger settlements were associated to the lower stiffness of the alluvial soils of river Kemonia and to a higher percentage of ground volume loss.

The same data were later analysed in order to investigate the effects of tunnelling in more detail. For such purpose the settlements recorded at the bench marks have been plotted versus distance y from the tunnel face at the time of recording. Such distance was taken along the horizontal, with reference to the orthogonal projection of the bench mark on the tunnel axis.

Figure 8a shows the settlements measured by the bench mark which experienced the maximum final settlement. This bench mark, labelled by number 8.3, was placed on the previously mentioned masonry retaining wall located between the street and the park. The plot points out that settlement is produced not only by excavation of the span located directly under the check point but also by excavation of the preceding and of the following spans. It can also be appreciated that, when excavation is approaching the bench mark (i.e. negative distance on plot) the settlement proceeds not only during excavation but also when the tunnel face is immobile (dotted lines between excavation spans). A similar behaviour was observed for all bench marks.

However, if these discontinuities are disregarded, the measured settlements can still be interpolated by a Gauss cumulate probability curve (Fig. 8b) as first suggested by Peck (1969) and later confirmed by several authors (e.g. O'Really and New, 1982; Nyren et al. 2001). Moreover, if the settlements S recorded at each bench mark are scaled

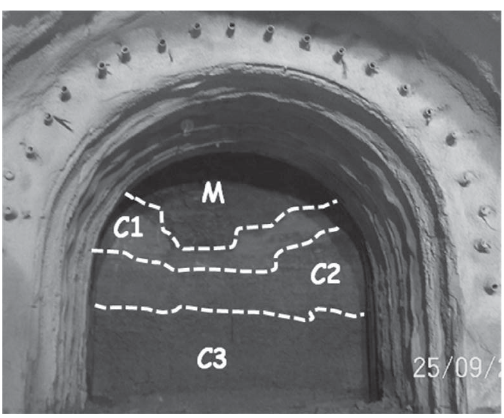


Figure 7. Lithological observation of excavation face: M made land; C1 slightly cemented calcarenite; C2 weakly cemented calcarenite; C3 cemented calcarenite.

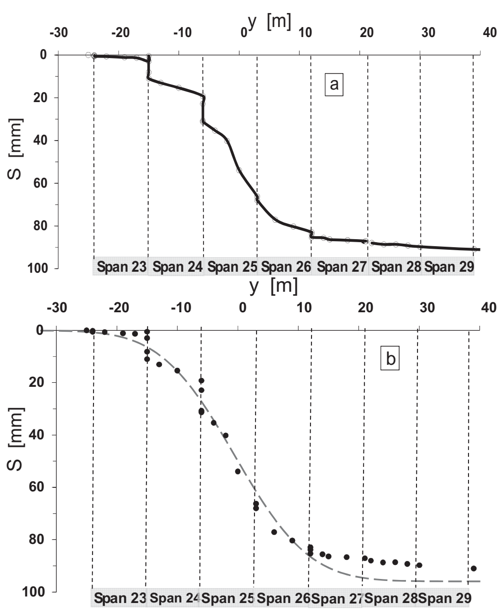


Figure 8. Surface Settlements (S) vs. Distance (y) from excavation face as recorded at bench mark 8.3 (under retaining wall). a) Settlement progress b) Data interpolation by cumulate probability curve.

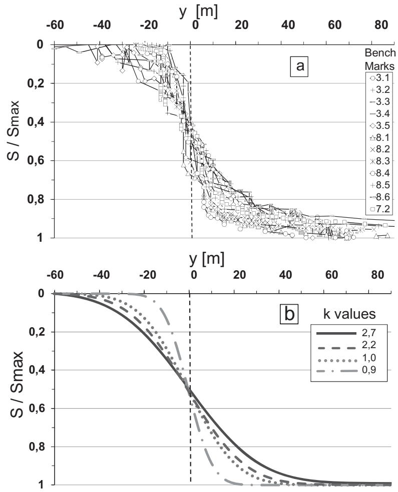


Figure 9. Settlements ratios for all bench marks. a) Recorded data b) Average settlement ratios (k = 2.7—building settlements on calcarenite; k = 2.2—retaining wall settlements on calcarenite; k = 1.0—under pass settlements on steel frame; k = 0.9—retaining wall settlements on alluvial soils).

with respect to their maximum value S_{max}, then all the data can be grouped in a single plot as reported on Figure 9a.

The parameter k was estimated for each bench mark according to the well known relation

$$\frac{S}{S_{max}} = \frac{1}{kH\sqrt{2\pi}} \int_{-\infty}^{y} e^{\frac{-y^2}{2(kH)^2}} dy \tag{1}$$

The k values, obtained by numerical best fitting of the recorded data, are reported on Table 1 together with the maximum settlement S_{max} and the depth H of the tunnel centre line.

Clearly, the results are scattered because of the variable conditions encountered along the tunnel stretch. However these data can be further analysed by considering four different combinations of structures and subsoil characteristics, listed as they were met during tunnelling:

1. buildings on calcarenite (canopy 1.a/1.b)
2. underpass on calcarenite (steel frame)
3. retaining wall on alluvial soils (canopy 2)
4. retaining wall on calcarenite (canopy 1.a).

Table 1. Settlements data.

Bench mark	S_{max} (mm)	H (m)	k	Soil type	Canopy type	Existing structures	k	S_{max} (mm)
1,2	3,5	11,7	4,3	Calcarenite	1.a	Buildings	2,7	7,7
2,1	3,5	11,7	3,9					
2,2	3,9	11,7	4,5					
2,3	9,4	11,2	2,9					
3,1	10,6	11,2	2,3					
3,2	9,9	11,2	2,2					
3,3	9,7	11,2	2,0					
3,4	9,9	10,6	1,4					
3,5	9,4	9,6	2,2		1.b			
4,1	6,9	9,6	1,3					
4,2	2,6	10,0	0,8		Steel frame	Underpass	1,0	2,5
4,3	2,2	9,9	1,2					
4,4	2,6	9,9	1,1					
7,2	70,3	9,7	1,1	Alluvium	2	Retaining wall	0,9	64,0
7,3	67,6	9,8	1,0					
8,1	71,3	9,7	0,9					
8,2	83,4	9,7	0,8					
8,3	95,9	9,7	0,8					
8,4	99,2	9,7	0,9					
8,5	68,2	9,8	0,8					
8,6	39,2	9,8	0,9					
8,7	26,3	9,8	0,8					
8,8	19,0	9,8	0,9					
8,9	15,5	9,9	2,8	Calc.	1.a		2,2	12,9
8,10	11,0	10,0	1,4					
8,11	13,0	10,2	1,0					
8,12	12,2	10,3	3,6					

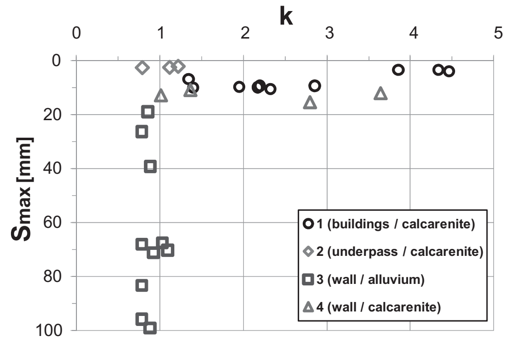


Figure 10. Maximum settlements and k values for different combinations of structures and subsoil characteristics.

For each of the above groups, average values of S_{max} and k are reported in Table 1 and average settlement trends are drawn in Figure 9b. Moreover S_{max} values are plotted versus k values in Figure 10, in order to provide a general picture of the observed settlements.

By comparing data groups 3 and 4, the influence of subsoil properties can be singled out. In fact, in these two cases, the influence of the masonry wall (see Fig. 4b) is equal and probably very limited due to its poor rigidity. It is thus confirmed that the amount of settlement is mostly related to the subsoil stiffness, higher for calcarenites and lower for alluvial soils, even though relevant data scattering is observed particularly for the alluvial soils. Some scattering is also observed for buildings settlements on calcarenite (data group 1) due to their variable structural and foundation features (Standing, 2006; van Tol, 2006). However, in the average, buildings settlements are smaller than those of the masonry wall on calcarenite. This difference can be attributed to their higher structural rigidity and also to their larger distance from the tunnel axis (see Fig. 4a). The underpass settlements are even smaller due to the very high rigidity of this piled structure (see Fig. 4c).

Finally from Figure 9b it can be seen that the settlement curves are steeper for the alluvial soils with respect to the calcarenitic formation. It also appears that the higher rigidity of the buildings, compared to that of the masonry wall, provides a more elongated shape to the length-settlement curves.

5 CONCLUSIONS

The project of a shallow sewer tunnel in difficult site conditions has been successfully accomplished by means of the canopy technique, making proper use of different soil improvement methods: steel and fibreglass micropiles, jet grouting, horizontal drains. In particular, design was carried on by dimensioning three different types of canopies, each one associated to a typical stratigraphical sequence.

During excavation a detailed monitoring program was organized and the tunnelling process was followed step by step for each construction span. This observational procedure allowed for proper implementation of the design canopies according to the subsoil conditions met during excavation.

Settlement recording was the most effective means for monitoring the near by structures in real time and provided also useful data for investigating the effects of tunnelling afterwards. It was found that the final settlements measured after tunnel completion reached very different values, corresponding to the various subsoil conditions and canopy types.

It was also observed that settlements are produced not only by excavation of the span located directly under the check point but also by excavation of the preceding and of the following spans. In addition it was appreciated that settlements proceed not only during excavation but also when the tunnel face is immobile. However, by careful processing of all the settlement data, it was observed that the relation between settlement ratio and distance from the tunnel face still follows, with acceptable approximation, the typical shape of a Gauss cumulate probability curve as repeatedly observed for TBM tunnelling.

Clearly, more observations are needed in order to gain a better understanding of movements induced by canopy technique tunnelling In fact only few cases on this topic have been published so far (e.g. Croce et al. 2004, Russo & Modoni, 2006). Some new settlement data gained by other case histories of canopy tunnelling are going to be reported in the near future (Croce et al., 2011).

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