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Milan Metro Line 4: The Dateo underground station

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ABSTRACT: The Metro Line 4 is the new underground line under construction in Milan, connecting the east to the west of the city centre. The underground Dateo station is one of the boldest along the whole line. Its depth is the maximum never reached by the subway lines built in Milan and it is determined by the exigency to pass under the existing underground Bovisa-Dateo Railway Link. The excavation is 31 m deep, 20 m below the water table, 60 m long and 23 m wide, having a distance from adjacent buildings less than 3 m. Retaining structures consists of 50 m high concrete diaphragms, 1.20m thick, with a 20 m injected bottom plug. It was excavated using a combined bottom-up/top-down method. The paper focuses on two key aspects of the design: settlement control of the nearby existing buildings and construction countermeasures to avoid drainage and counteract high underground water pressure.

1 INTRODUCTION

The Line 4 - also called Blue Line - is a new subway under construction in Milan, financed by M4 S.p.A, a holding formed by a public shareholder - the Municipality of Milan - and some other private ones, among which Salini Impregilo SpA and Astaldi SpA, which are also the main Contractors for the whole infrastructure.

Once completed, the line will connect the western and eastern sides of the City – from Linate airport up to the border with Buccinasco Municipality – becoming a strategic infrastructure of the mass-rapid transport system in Milan. The 15 km of the new Line will connect 21 stations and 30 intermediate shafts; the Line 4 will be the second driverless line of Milan. Underground stations are excavated in loose sandy soils below the water table, alongside the existing buildings and infrastructures in a densely populated urban area. The metro stations are built using "bottom-up" methods below the water table or "combined bottom-up/top-down" methods for deeper excavations. The level of tracks is placed in a range between 15 and 28 m below the existing ground level: the maximum depth is reached in correspondence of the Dateo Station. This depth is the maximum never reached by the subway lines built in

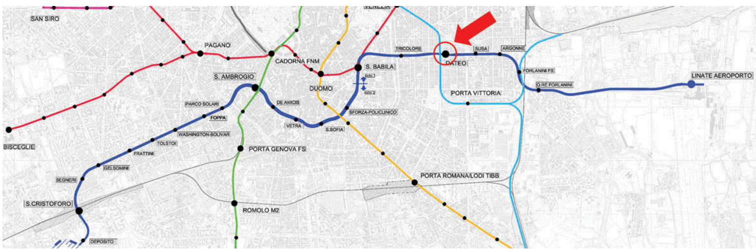

Figure 1. Alignment of the new Milan Metro Line 4 – Dateo Station in the highlighted circle.

Milan and it is determined by the exigency to pass under the existing underground Bovisa-Dateo Railway Link. The excavation depth, the high hydraulic head due to the water table and the presence of a great number of masonry buildings, some of which date back to the early 1900s, required an accurate analysis of the construction methodology, followed by a detailed design of the construction choices to fulfil the severe safety requirements and tough construction schedule fixed by the Municipality.

2 STATION LAY-OUT: SHALLOW VERSUS DEEP SOLUTION

The existing Dateo FS station, the railway tunnel, together with the sewage culvert called "Nosedo" nearby the ground surface were the key elements to be faced at the beginning of the design process: as a matter of fact, the existing underground railway intersects the new Metro 4 alignment almost orthogonally at a depth of 17 m, preventing the possibility to built the new Metro Line and the Dateo Station. Two different solutions were studied to overcome this problem: the first ("A" solution hereinafter) envisaged to cross over the existing railway (Fava & Galvanin, 2012), while the second ("B" solution) entailed to pass below the existing railway by means of twin tunnels excavated with two TBM machines. In the "A" solution, the Dateo station and the whole stretch of the Line passing over the existing railaway was conceived as "cut and cover" tunnel, executed with a "bottom-up" procedure, inside concrete diaphragms: existing roads should have been diverted on alternative routes, to gain the working areas along the Corso Plebisciti, creating great impacts on the traffic in the city center. This solution was very simple to be built; nevertheless, it would have created some problems to the altimetric alignment of the new line, forced to pass below the Nosedo and over the near existing tunnel with a partial demolition of the concrete revetment, as shown in Figure 3. Moreover, the "A" solution would have been not suitable for the excavation of Line 4 tunnels using the TBMs: the machines should have been dismantled before the Dateo station and remounted in the next one, with a remarkable loss of time and not negligible extra-costs. The "B" solution was developed to supersede the limits of the previous one, achieving a consistent reduction of

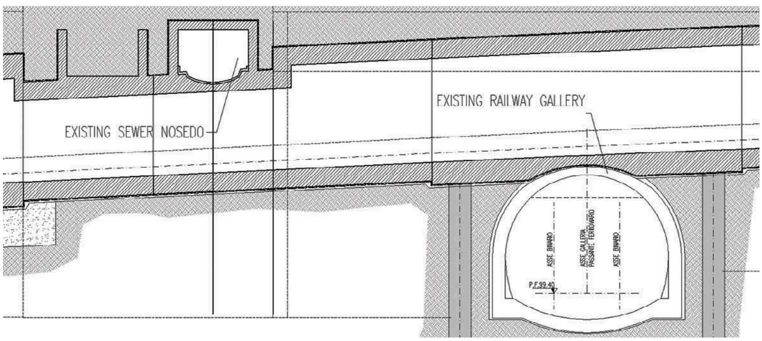

Figure 2. Metro Line 4: longitudinal profile solution "A". The Nosedo sewage culvert is on the left, while the existing railway tunnel is inside the diaphragms on the right.

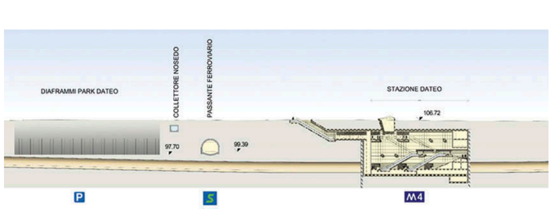

Figure 3. Metro Line 4: longitudinal profile solution "B".

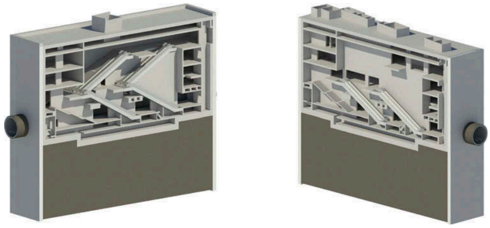

Figure 4. Dateo Station -solution “B”: 3D Model of the internal lay-out.

the impacts on traffic, a significant saving of the working areas and an improvement in the construction time schedule for tunnel excavation by means of TBMs.

In the “B” solution, the Dateo Station is placed underneath Corso Plebisciti, on the East side of the existing Dateo Railway Station, but the axis of the new station is rotated roughly 8° with respect to Corso Plebisciti axis road to avoid the interference between some deep foundation piles of the existing railway which would have obstacle the TBMs.

The Dateo Station lay-out in solution B is divided into two parts:

– a shallow portion (11.70 m from the ground level) which links the existing railway station to the main part of the new Metro 4 station;
– a deep portion, having planimetric dimensions 60 x 23 m and a maximum depth from the existing ground level equal to 32 m.

Due to the depth of the railway tracks, the Dateo functional lay-out is quite different from the standard one used for the majority of the Metro 4 shallow stations. The main portion of the station (see Figure 4) is hugely characterized by architectural choices studied to create a wide and open internal volume, without columns or intermediate slabs, useful to place the internal access stairs and escalators, which present a remarkable length, due to the vertical distance between the existing ground and the platform level. The architectural choices, as well as the above mentioned phisical constraints, required a carefull preliminary analisys of the construction methods, followed by an extensive structural analysis, as described in the following paragraphs.

3 DESIGN SOLUTIONS FOR THE DEEP STATION

3.1 *Geological and geotechnical setting*

The Dateo station, located in the eastern part of the Milan historical city center, interferes - from a geological point of view - with the so called recent “Diluvium”, formed by loose silty-sandy granular soils having high permeability (10-5/10-6 m/s) and characterized by a random variability of their granulometric composition, from silt to cobbles. Some more frankly silty clayey strata can be present, 2-3 meters thick. The geotechnical surveys executed before starting the design stage and deepened up to 60-70 m from the ground surface – well below the depth usually investigated for standard buildings in Milan - confirmed the first hints from technical literature and some typical characteristics of the Milan underground soil, for depths greater than 30-35 m (Cancelli, 1967).

It is worth to note that the deeper surveys highlighted the presence of two silty clayey layers, the first at 37-38 m from the ground surface - having a thickness of 3.50 m - and the second, with a similar thickness, at a depth of 57 m from the surface, with a higher content of silt and sand.

The presence of a clayey layer at a depth next to 40 m is a characteristic “marker” of the underground soil in Milan, as evidenced in the past by several geological studies (i.e. Desio ‘53, Niccolai et al. ’67). This layer, almost continuos beneath the whole city center, marks certainly a inter-glacial phase during which glaciers withdrew up to Alps. This layer marks also a transition from mainly gravelly-sandy soils in the first 30-35 m, to silty-sandy soils in which the gravel component is negligible. This granulometrcic variation, well highlighted by

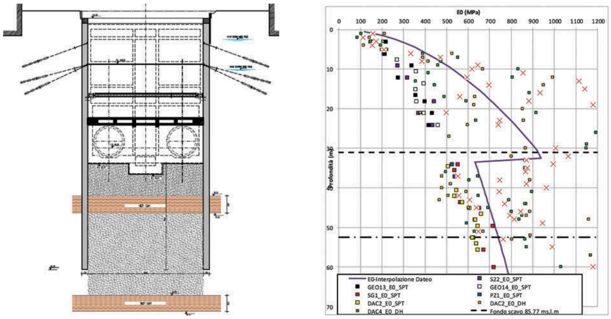

Figure 5. Dateo Station: simplified stratigraphy (brown strata: silty clay) and distribution of small strain E_0 modulus with depth.

geotechnical surveys, impacts also on geotechnical properties of soils as well shown in Figure 5: as a matter of fact, soil below 35 m is less stiff than the upper portion and the v_s shear velocities, as well as the small strain E_0 modulus, exhibit a significant discontinuity: all this information, above all the E modulus values for displacement and settlement calculation, was taken into account in the geotechnical and structural analysis and in the study of injections to be used for excavation waterproofing.

3.2 *Choice of the construction method*

Once the station lay-out was defined according to the planimetric and altimetric alignment of the Line as for "B" solution, the design focused on the choice of the most suitable construction procedures: station lay-out, its position and depth, the presence of a powerful aquifer and of numerous buildings at the ground surface as well as the construction time-schedule deeply affected the design choices: after several analysis and different proposals, a combined *bottom-up/top-down* excavation strategy was considered the most suitable in order to:

– guarantee the maximum excavation safety, limiting the settlements underneath the existing buildings placed in a range from 3 m to 10 m from the perimetric retaining walls of the station;
– comply with the construction time schedule, allowing the breakthrough of the TBMs used for the tunnel excavation without interferences with the construction stages of the station: in other words, a complete separation of the two working processes (one for the station and the other for tunnels) was possible thanks to the chosen solution, thus creating a remarkable time saving in the construction schedule;
– execute the ground injection for excavation waterproofing as fast as possible.

Different solutions of combined *bottom-up/top-down* excavation sequences (Peri et al., 2011) were analysed to achieve the above goals:

– the "B1" solution envisaged two intermediate reinforced concrete slabs, having a thickness 50/60 cm, using the top down procedure: these two slabs should have been withstood by vertical columns, executed in advance from the ground level by means of deep drilling, as shown in Figure 6 on the left;
– the "B2" solution entailed a different approach, based on a system of temporary steel struts in lieu of the upper slab of the "B1" solution, plus a unique intermediate slab. This slab, 140 cm thick, is conceived to counteract the perimetric wall thrust working as a one-way slab, having 23 m span, hinged on the diaphragms, without any kind of intermediate support. To achieve this result the slab was designed as a latticework, lightened with polystyrene (Figure 6, on the right).

The "B2" solution was preferred to the "B1" because it allowed to eliminate the support columns, removing any kind of internal constraint for the TBMs breakthrough, without waste

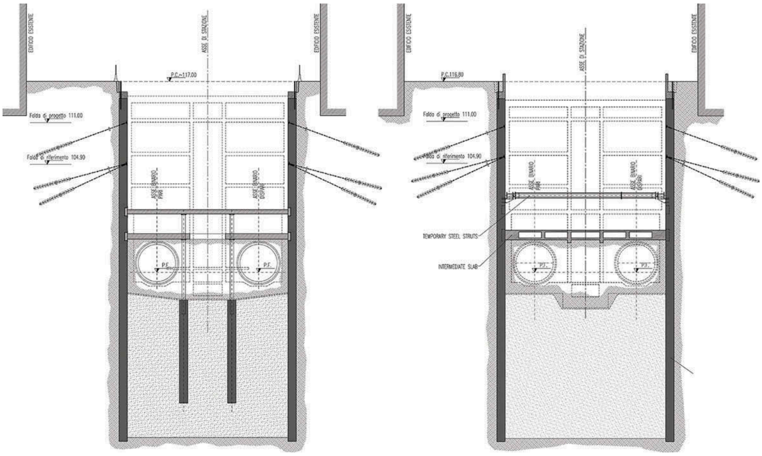

Figure 6. Dateo Station – Excavation sections and retaining structures – Solution “B1” (left) vs. Solution “B2” (right).

of time for the waterproofing and final revetment of the temporary support columns. In a nutshell, the “B2” solution envisaged the following constructions stages:

1. concrete diaphragms execution, 120 cm thick and 50 m long, using a clamshell bucket;
2. excavation up to the ground water table: the concrete walls are restrained by two prestressed anchor levels, the first placed roughly 5 m below the ground level, inclined to avoid interferences with the existing foundations, and the second placed half a meter above the water table, 10 meters below the ground surface;
3. once reached the working level for the second anchors level, the injection of the soil below the foundation slab is carried out;
4. after the injection completion, the excavation is deepened up to 18 m below the existing surface and the steel trusses are installed: this system is composed by waling beams (2 HEM 900) and circular hollow steel struts (diameter 1100 mm, 20/26 mm thick), spaced with an interval in the range from 4.5 m to 6 m;
5. the excavation is deepened up to the level of the intermediate slab, casted in situ on the bottom of the excavation, roughly 20 m below the existing ground surface: this slab marks the limit between the bottom up and the top down procedure: underneath the slab, the excavation continues as in a top-down mode; some hatches were provided in the slab to allow the excavation of the remaining portion of the station, up to the foundation level, and the lifting and lowering of the construction materials,
6. excavation is carried on below the slab executed in the previous stage and the foundation slab is completed;
7. after the TBMs breakthrough, the platform level is casted, while the bottom-up construction of the upper part of the station continues, dismantling the temporary steel struts, without interferences with the tunnelling process.

The combined procedure *bottom-up* and *top-down* proved to be the right choice for the following reasons:

– the use of steel struts and - above all - the intermediate slab casted before reaching the foundation level were able to reduce the horizontal displacements of the diaphragms, limiting the settlements of the adjacent buildings, improving the safety of the excavation;
– the removal of anchors below the water table entailed to reduce construction time and to eliminate the problems related to drilling operations below the water table in loose soils (which usually require preventers and accurate procedures to avoid dangerous water leakages inside the excavation);

– the complete separation between the station construction procedures and the tunnel excavation process avoided interferences with TBMs break in and out: also in this case the time saving was remarkable and the twin tunnels were completed within the scheduled time.

3.3 *The excavation retaining walls*

The deep excavation inside the perimetric walls was executed using reinforced concrete diaphragms having a thickness of 1.20 m and a length equal to 50 m, considered in the structural design like permanent retaining structures.

Two different construction options were taken into account before making the final choice: the first one envisaged the use of an hydromill, while the second the use of a clamshell bucket. Hydromills are normally used for diaphragms having a length grater than 30-35 m and they guarantee optimal performances in terms of verticality tolerances, execution speediness and joint-water tightness. Notwithstanding the aforesaid advantages, they usually require, while operating in loose soils and if compared to other systems, a lower density bentonite slurry to maximize the performance of the excavation tools. Moreover, they require a quite huge working area, larger than the one available in Dateo: due to these constraints the daily production could have been reduced below the optimum standard.

Having in mind the aforesaid limitations, a second option was examined based on a heavy (20 ton) clamshell bucket, with an hydraulic closure system type “BAYA”, in the version stretched by TecSystem for this kind of applications: the bucket verticality is controlled by an electronic device to assure the compliance with the vertical design tolerances (0.5%). The satisfying experiences with this kind of equipment - gained during construction of Line C in Rome and High Speed Railways in Florence, encouraged to propose this solution as the final one: as then experienced on site, the bucket was able to guarantee the same performances of the hydromill in terms of diaphragms quality, continuity and verticality: the good final results can be appreciated also looking at Figure 8, where the absence of water leakages through diaphragm joints and from the bottom of the excavation is well remarkable.

The weathertightness of joints between diaphragms was studied during the design stage: the joint executed on site is a male-female type and was created in the diaphragms using “stop-ends” sheetpiles. These joints can be shaped while casting the concrete diaphragms, executing at first the primary panels, installing on its two lateral faces the trapezoidal sheetpiles, having an appropriate length if compared to the depth of the excavation (36 m for the Dateo Station).

Once completed the primary panels, the secondary ones are excavated and, as soon as the excavation is completed, the sheetpiles are removed by means of a vibrator. The extraction time is normally 2-5 mins for each sheetpile without damaging the adjacent panels. For Dateo station the male-female joint water tightness was improved using two different water stop: the

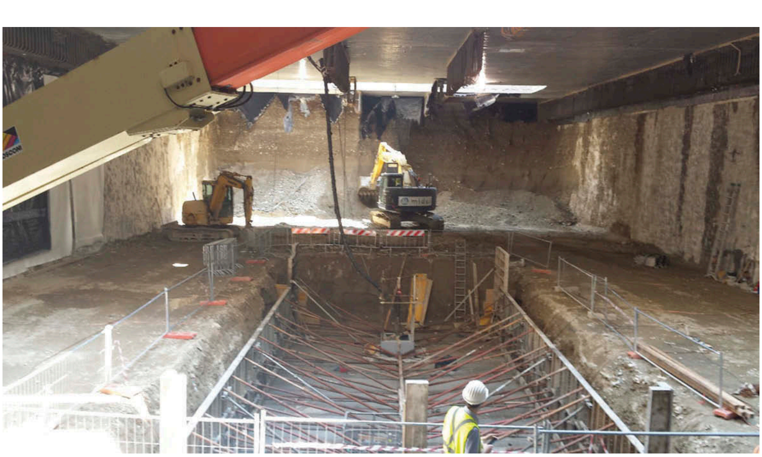

Figure 7. Top down excavation stage.

first is the standard PVC profile, while the second is a continuous and injectable pipe to be used in case of failure of the PVC profiles, allowing the injection of polyurethane resins.

3.4 *Soil injection for the waterproofing of the excavation*

The presence of granular and permeable soils below a high water-table and the impossibility to execute a dewatering of the excavation by means of pumps, to avoid unacceptable settlements of the adjacent buildings, imposed to inject the soil below the bottom of the foundation slab, before removing the soil below the water table.

During the design stage, three different options were evaluated. The first solution ("J1" on the left in Figure 8) envisaged a massive injection of the soil for a total height equal to 20 m below the foundation slab (Balossi Restelli, 1981). This injection was executed using a TAM methodology (tubes à manchettes injections) with cement and silicates controlled by volume and pressure. In this solution the length of the diaphragms is optimized since the hydraulic pressure acting on the bottom part of the excavation is counteracted by the self-weight of the injected soil and by the "arch effect" of the massive treated soil. The "J2" solution (in the centre of Figure 8) entailed the execution of a thin waterproof layer placed at a depth where the self-weight of the soil above the waterproof layer equals the water pressure. The waterproof treated soil, in this case, is composed by a first layer of cement injected soil, thick 0.50 m, a second layer 1 m thick, injected with high quantities of sodium aluminate and a final capping layer injected as the lower one. At the bottom of the excavation a layer injected with micro-fine cements was added to improve the soil passive thrust acting on the diaphragms, to reduce the horizontal displacements of the perimetric walls. This solution would have been able to optimize the total injected volume and also the execution time; conversely it would have required a greater length of the perimetric walls (up to the depth where the self weight of the treated soil equals the water pressure) and a greater accuracy in drilling procedures to assure the correct verticality, having a length more than 40 m.

The "J3" solution, (on the right in Figure 8) was based on a TAM injection of cements and silicates as in J1, but the treated soil was designed as a "sandwich" to optimize the total injected volume. The diaphragms - as explained for the J2 solution - should have been stretched up to the pressures equilibrium point.

The three different solutions were object of careful cost-benefit analysis, at the end of which it was decided to operate with the J1 solution described above: the key points in the final choice turned out to be:

- the verticality check during drilling phases - much less stringent in solution J1 tanks to the massive soil treatment;
- the impossibility - due to technological constraints - to guarantee the continuity of the male-female joint below a depth of 36 m (maximum length of the sheet piles), with possible water leakages from joints in the diaphragms, which could have had compromised the stability of J2 and J3 solutions.

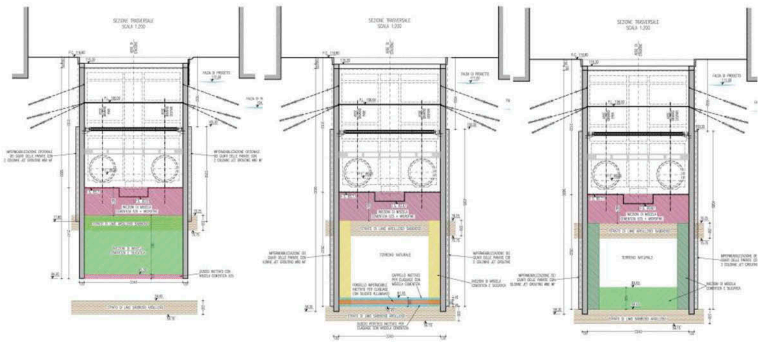

Figure 8. Different injection solutions studied for the waterproofing of the bottom of the excavation.

4 DESIGN CRITERIA AND ANALYSIS OF THE RETAINING STRUCTURES

4.1 *Assessment of settlements induced by the excavation*

While excavating, the loosening of the ground behind a retaining wall creates a subsidence basin, at the ground surface, potentially able to induce unacceptable settlements of buildings next to the excavated area and their potential damaging: therefore, it is necessary to carry out an estimate of the possible damages caused by settlements to verify their admissibility. Analyses of potential damages, caused to buildings or other facilities next to an excavation, is usually divided into the following phases:

1. assessment of vertical subsidence and consequent maximum values of vertical settlements and distortions potentially induced by the excavation on the buildings;
2. evaluation of category of the potential damages on buildings to be compared with the allowable ones.

The starting key point for each estimate of expected potential damages is the definition of the foundation settlements induced by the excavation. Considering the importance of the project and the proximity of existing buildings to the excavation, 2D FDM numerical analyses were carried out, assuming plane strain conditions, following an excavation procedure and a gradual staged construction, to have a realistic prediction of settlements and damages. The calculation code used was FLAC 2D (Fast Lagrangian Analysis of Continuous), developed by ITASCA Consulting Group. The material behaviour in the models was described by means of linear elastic constitutive laws for structures and elasto-plastic laws, based on the so called "CY-Soil" model implemented within the calculation code, for granular soils. The "CYSoil" model was used to take into account the different elastic properties of soils along unload-reload conditions. In particular, the compressibility modules were evaluated using the equation:

$$E_s = m\,Pa\;(\sigma_h/P_a)^n \tag{1}$$

where P_a = reference pressure = 1 atm; m=1000; n=0.5 and assuming a coefficient equal to 3 within the model "CYSoil" to simulate unload and reload conditions avoiding an overestimate of the induced settlements.

The numerical models were able to highlight some elements useful for excavation monitoring during construction:

- the maximum horizontal displacement of the concrete perimetric walls is equal to roughly 15 mm at a depth 20 m from the ground surface;
- maximum vertical settlements observed on the buildings are in the range 8-10 mm; in the numerical analysis a building was simulated at a medium distance equal to 5 m from the diaphragms;

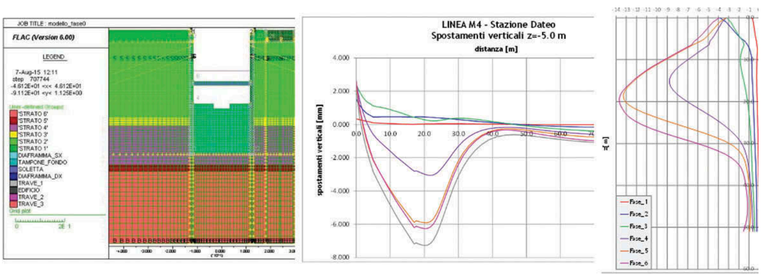

Figure 9. FDM 2D model, maximum vertical displacements beneath building foundation and wall horizontal displacements for different excavation steps (mm).

– examining the subsidence curves, the vertical displacements induced at the ground surface are pretty greater than those induced immediately below the buildings foundation level. This effect is due to the foundation stiffness and to the stress-release in the soil, caused by the excavation;
– the subsidence basin is quite huge, due to the excavation depth (30-40 m), so buildings inside this basin are potentially involved by the excavation in the so called “sagging” or “hogging” zones;
– the shear strains in the soil behind the concrete walls are approximately 1‰; thus they are in good accordance with the choice of the operative E_{op} modulus used in the numerical analysis, and assumed equal to 1/3 of the small strain modulus E_0 shown in previous § 3.1.

4.2 *Structural analysis*

Design and verification of the whole structure, partly “top down” and partly “bottom up”, were developed starting from the identification of its characteristics in terms of functionality, construction requirements, expected performances and applied loads to form the database to be used in structural analysis.

The various structural typologies used for construction were modelled by means of different FEM codes (Midas Gen and Sap 2000), to specialize, in the most rigorous and efficient way, the necessary checks, taking into account the differences in the structural typologies and construction procedures. The numerous construction phases and static schemes, together with the need to anticipate the execution of the intermediate slab, suggested to develop different models, with increasing degrees of complexity, as listed below:

– 3D models for the system of temporary steel struts, counteracting the concrete walls;
– 2D shell models for the dimensioning and verification of the intermediate concrete slab;
– plane frame models, for analysis of the effects caused by possible differential settlements of the base slab and by the staged construction and loading;
– a full three-dimensional model to check the final station lay-out and to carry out detailed analysis of the foundation structures and intermediate floors.

4.3 *Monitoring strategy*

Before starting the excavation of the Dateo station, a monitoring plan was prepared considering the proximity of the excavation to the existing masonry buildings, some of which date back to the early 1900s, as usual in the central areas of the city centre.

The installed instrumentation allowed to carry out surveys on buildings while excavating, according to construction sequences, to detect in advance the extent of possible excessive settlements or rotations of buildings produced by the excavation. The following quantities were subjected to periodic monitoring:

– vertical displacements of buildings;
– rotations of buildings;
– movement of existing structural joints on buildings;
– horizontal displacements of the diaphragms at various depths from the ground surface;
– stress and strains inside the steel struts and the intermediate concrete as well as temperature variation;
– width of pre-existing on new cracks present on buildings.

The numerical analyses carried out during the design stage allowed to set the attention and alarm thresholds for each measured value, together with the countermeasures to be implemented, if the thresholds were reached or exceeded: monitoring data made it possible to ascertain the correspondence between the behaviour of diaphragms and buildings predicted by numerical models and the observed one during the construction phase.

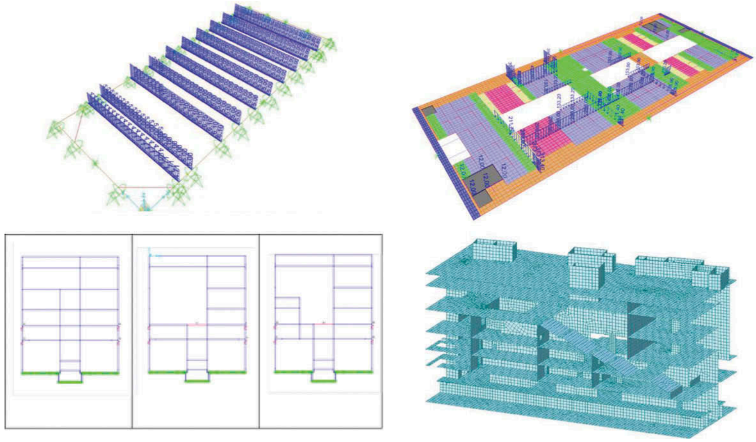

Figure 10. Different structural models used for station design. From left, clockwise: 3D models for the steel truss behaviour analysis, 2D model for the top-down slab, 2D simplified models for analysis of effects induced by construction stages, 3D complete model.

5 CONCLUSIONS

Solutions provided for construction of Dateo station, described in the present paper, are the result of a design process which required a careful analysis and comparison of different solutions, to optimize the whole lay-out and construction process of this strategic station, placed in a complex urban context. Many recent experiences in the construction of similar structures show that design choices are strongly affected by conditions present on site, construction time-schedule and specific exigencies of the stakeholders involved in the project. The desired optimizations of the construction process can be attained only through an appropriate preliminary analysis and a coherent design development, which usually require an appropriate time frame to be fully detailed: this time frame should be taken into account during the drafting phase of the construction program. If these analyses and structural modelling are adequately detailed, they allow a real improvement of the construction process, minimizing the "adjustments on site" to solve contingent and unforeseen problems.

The experience gained in the construction of Dateo station could be a useful reference for future further refinement of combined top-down and bottom-up excavation procedures for deep under-ground structures in urban areas, to achieve even more faster and reliable construction processes.

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