Arch shaped cut-and-cover tunnels on the new "Pedemontana Lombarda" motorway

G. Lunardi, M. Gatti
Rocksoil Spa, Milan, Italy

ABSTRACT: An innovative prefabrication method has been successfully employed for the construction of arch-shaped cut and cover tunnels on the new "Pedelombarda" motorway. The tunnels consist of prefabricated components resting on a cast-in-place invert. This solution has been employed with span widths varying from 14 m to 19 m and overburdens from a minimum of 1 m up to 8-9 m. Particular attention was paid to the backfilling stages, in order to define the geotechnical properties of the soil used for lateral and top backfilling. The high percentage of prefabrication in the solution adopted allowed high production rates to be maintained and also the use of high visual quality prefabricated concrete. After a detailed description of the construction system, this paper looks at the analyses performed in the design stage and the results of monitoring conducted during the testing of the works compared to design predictions.

1 INTRODUCTION

The new "Pedemontana Lombarda" motorway is a key system for the mobility of its territory, which will serve almost 4 million people. The first step towards this important transportation network is "Section A", the works of which were contracted to the General Contractor "Pedelombarda" S.c.p.A, made up by: Impregilo S.p.A, Astaldi S.p.A., Impresa Pizzarotti & C. S.p.A and A.C.I. S.c.p.A.. Section A is 14.5 km long, and will interest the provinces of Varese and Como, linking the existing A8 "Milan-Varese" Motorway with the A9 "Milan-Chiasso" Motorway, through interconnections placed in Cassano Magnano and Lomazzo (Figure 1). The route passes through a partially urbanized territory with important surrounding constraints such as roadways and tracks. Cut and cover tunnels were widely used (even with overburden up to 8-9 m) in order to limit the impact on the territory itself. Likewise, in order to speed up the tunnels' construction (and therefore limit the disturbance for the local inhabitants), an industrialized prefabricated tunnel system was employed. An innovative typology of arch shaped pre-fabricated linings was adopted to cope with major heights of backfill cover, thanks both to the structural shape-effect and ground arch-effect mobilization. Rocksoil S.p.A. carried out the final design, and defined the construction method for the pre-fabricated elements; Codelfa S.p.A. and Impresa Pizzarotti e C. S.p.A. constructed the works; Simete S.r.l worked on the detail design.

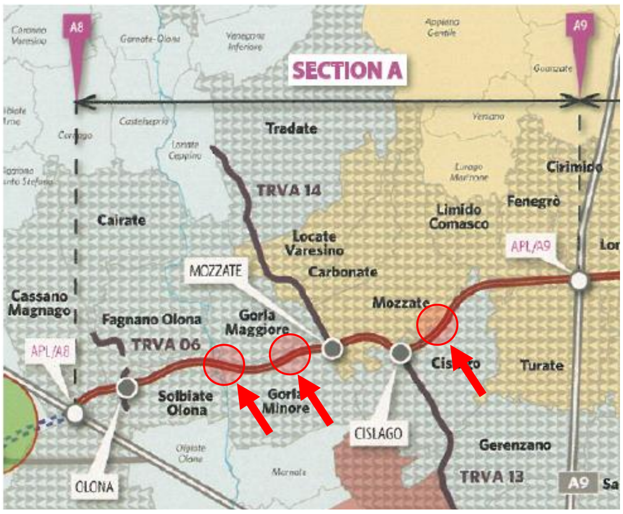


Figure 1. "Pedemontana Lombarda" Motorway Section A

This paper (after a brief overview of the works and the geological-geotechnical context) aims to illustrate in detail the construction system used and the executive phases followed. Furthermore, it shall highlight the importance that backfilling methods have on the proper static performance of cut-and-cover tunnels (especially regarding the geotechnical properties of the backfilling soil), as hypothesized by the static analyses, and proven by the load tests and deformation measurements.

2 BRIEF OVERVIEW OF THE WORKS

Three of the artificial tunnels along the motorway were constructed with the method presented in this document: the "Solbiate Olona" (670 m long), "Gorla" (1,280 m), and "Cisalgo" (474 m) tunnels, named according to localities. Each of these tunnels is twin bore, one for each traffic direction; the road is made up of three lanes for each direction, two of which are for traffic (each 3.75 m wide), and the third for emergencies (3.00 m wide, with a side platform on the left 0.70 m wide). In order to guarantee visibility on bends for the design speed of 80-140 km/h (in presence of planimetric radii of curvature varying from 1,260 m to 1,350 m, and of transversal inclinations equal to up to 5.8%), the road presents widening up to 19 m. Table 1 reports the width of the carriageway for each tunnel, towards East and towards West, as well as the overburden on the crown (maximum value 8-9 m). The Solbiate Olona tunnel presents a section with top backfilling made up of light weight material (equivalent cover 8.30 m). The internal profile of the tunnels guarantees heights equal to 5.50 m from the road, due to the presence of signage on the crown (min. net gabarit: 5.0 m).

Table 1. Tunnels' geometry.

Tunnel	Carriageway East (m)	Carriageway West (m)	Overburden (m)
Solbiate	13.70-16.75	13.70-19.00	2.00-8.30*
Gorla	14.75-17.85	13.45-16.45	1.00-8.40
Cislago	14.50-16.25	15.50-16.25	1.00-3.50

* Equivalent height with light-weight backfilling

Each tunnel includes emergency exits every 500 m, using lateral shafts equipped with emergency staircases. Emergency refuges are also positioned every 600 m, and constructed by widening the road 3.0 m on the right, thus reaching net internal widths up to 19.00 m.

Considering the different types of roads used, the tunnels were constructed of variable diameter between 14 m and 19 m; with invert intrados/crown extrados between 12.75 m and 13.45 m.

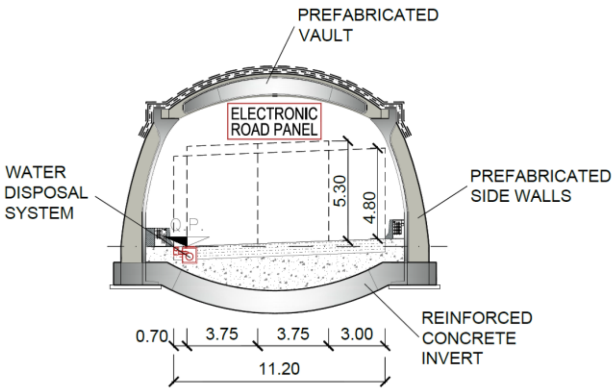


Figure 2. Arch-shaped tunnel -Typical section

Tunnels geometry also had to take into account the constraints brought upon the invert by the water-disposal system, as detailed in Figure 2.

3 GEOLOGICAL AND GEOTECHNICAL CONTEXT

The here-described tunnels fall within an almost completely flat morphological context. The surfacing formations are fluvial and glacial deposits belonging to the "*Besnate allogroup*" (middle-late pleistocene), consists of gravels and graded/stratified sands. The Solbiate Olona tunnel was almost exclusively constructed in deposits of gravels and silty sands, belonging to the lithological–geotechnical unit of "prevalent gravels"; while the Gorla tunnels passed through deposits of sand with silty gravels belonging to the lithic-technical unit of "prevalent sands". Furthermore, "*Olona Ceppo*" lenses and outcrops were faced during excavation; these were made up by clastic support conglomerates with variable levels of cementation – ranging from good to bad. Finally, the Cislago tunnel passed through the "*Besnate allogroup*" in the west part, and through the fluvial and glacial deposits of the "*Binago allogroup*" (middle pleistocene) in the east part: from a lithological point of view, the tunnel developed in sandy grounds with silty gravels, belonging to the lithological–geotechnical unit of "prevalent sands"; silty-clayey levels occurred also during the excavation phase. The Solbiate Olona and the Gorla tunnels didn't encounter any criticalities from the hydrogeological point of view, as they didn't interfere with the watertable, whose depth is greater than 35 m.

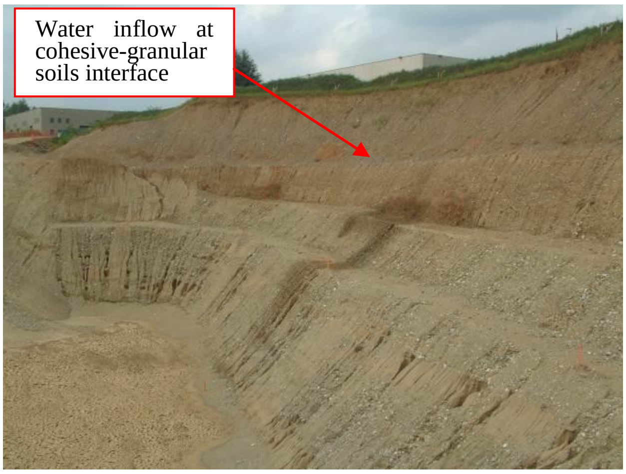


Figure 3. Excavation of a road cutting (Cislago Tunnel)

On the other hand, the excavation of the Cislago tunnel encountered local perched watertable (which had already been foreseen during the design phase), probably formed by accumulation over the impervious silty-clayey layers. In any case, the amount of water was modest. Table 2 reports the parameters of strenght and of deformability of the excavated grounds, in function of their depth. For the numerical evaluations, it was also important to define the value of the vertical Winkler constant k_w, estimated in the range 2-4 kg/cm^3.

Table 2. Geotechnical properties of natural soils (average values)

Stratigraphy	Depth m	γ kN/m^3	c' kPa	φ' kPa	E MPa
Backfill	0-1	18,0-19,0	0	28-30	10-20
Sand with silty gravel	<15	18,5-19,5	0-5	32-36	20-50
Sand with silty gravel	> 15	18,5-19,5	0-5	32-36	50-100
Gravel and silty sand	lents	19,0-20,0	0	34-38	20-50

Finally, for the seismic site conditions, the peak bedrock horizontal accelerations a_g are equal to 0.054g as regards the Life-Safety Ultimate Limit State, and 0.031g as regards the Serviceability Limit State, taking into account a nominal life V_N of 100 years with a class of use coefficient cu = 2.0 (period of reference V_R = 200 years).

4 CONSTRUCTION SYSTEM

The innovative system used for construction consists of an arch-type structure made up of three pre-fabricated elements: a top vault and two sidewalls, installed on an invert cast-in-place during the works. The shape effect given by the curved geometry of the construction elements increases the static capacities of the structure, allowing it to support heavy loads (i.e. significant overburdens), while continuing to adopt the usual prefabricated cross sections in reinforced concrete. The geometries employed, and the construction details are described below.

4.1 *General layout*

Two solutions were employed, according to the layout of the two carriageways. Wherever the inner distance between the two carriageways required maintaining a significant diaphragm between them (at least around 3 m), a "twin tunnels" solution was used; creating two separate tunnels, as shown in Figure 4a. On the other hand, when the distance was limited, an "adjacent tunnels" system with a shared central pillar was implemented (as shown in Figure 4b).

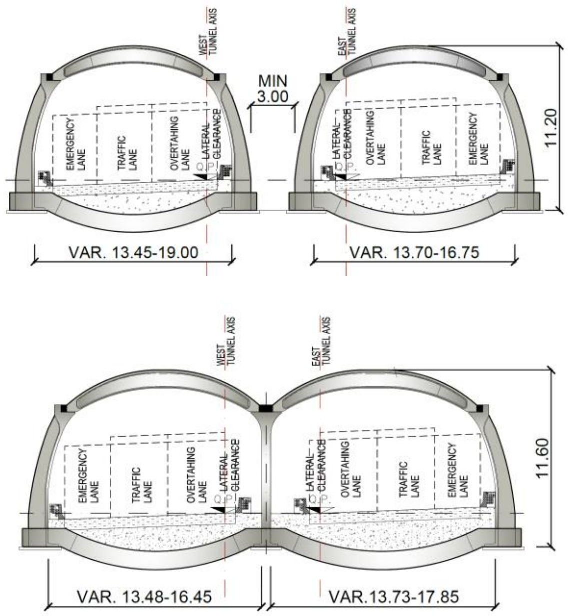


Figure 4. Separate/Adjacent twin tunnels – typical layout

The latter solution was preferred whenever possible, as it limited the motorway encumbrance in the territory; this system was adopted for almost the entire extension of the Gorla and Cislago tunnels, and for around the first 190 m of the Solbiate Olona tunnel.

The "twin tunnels" solution was used in the section of the Solbiate Olona tunnel - 490 m in length - approaching the conventional tunnel, where a significant diaphragm needed to be set between the two tunnels in order to safely start underground excavation (see Figure 5); this solution was also used for the initial section of the Gorla tunnel (120 m in length).

Figure 5. Solbiate Olona Tunnel –Final stretch of cut-and-cover tunnel close to bored tunnels portal.

4.2 *The prefabricated elements*

For the top vault precast-element a double T-section curved frame was used having a higher and lower wing 124 cm wide, and variable in thickness – from 8 to 15 cm (higher wing), and from 5 to 8 cm (lower wing). The height, at crown top, where the bending moment is maximized, was determined in function of the overburdens according to following three types: 100, 120 and 150 cm, with a 15 cm web. The height of the element also varies along the main axis, being reduced towards the supports, where the section presents heights between 94.70 (minimum) and 112.10 cm (maximum) according to the three structure types and the span-widhts involved. A full rectangular section is applied for the last 2.0 m close to sidewalls connection, in order to hold the maximum shear stress. The axis of the element is curved, with a radius between 8.68 and 10.56 m (Figure 6).

A "pi-greek" section geometry was chosen for sidewalls elements, 249 cm in length and 130 cm in height; the element contains two webs – from 23 to 27 cm thick – attached to a wing in the intrados of the tunnel, 16 cm thick.

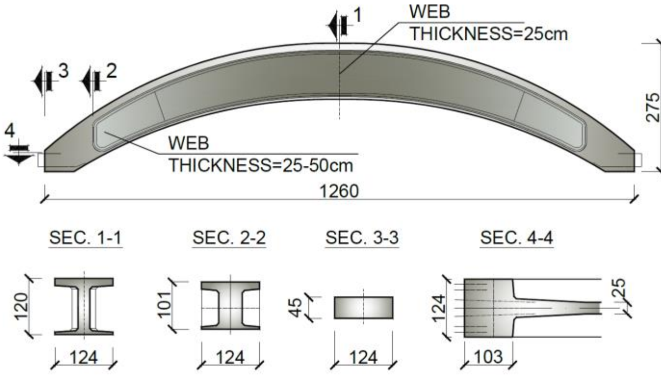


Figure 6. Prefabricated vault-element (H 120 cm).

The sidewall is also curved, with a radius equal to 3.47 m (Figure 7). Both prefabricated elements are in concrete, class C45/55, with ordinary steel reinforcement; therefore not requiring pre-compression. Main steel reinforcements, Ø22-26 mm bars, were adopted at the base of the web, coupled with secondary-reinforcement Ø16 bars, both in B450C steel.

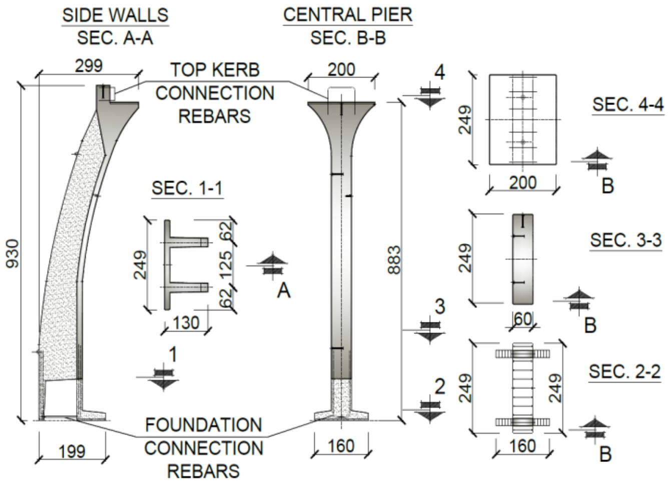


Figure 7. Sidewalls/central pier prefabricated elements.

The structural coupling requires each sidewall-element to support two prefabricated vault elements, so that the wings of the sidewalls and of the vault elements make up the uninterrupted intrados of the artificial tunnel. Special correction pieces were also employed in order to follow the planimetric continuity in curve of the layout; closing ribs were also used and cast during the works, in order to connect tunnel sections having different geometries. Finally, the structural outline requires an invert, cast-in place with class C25/30 reinforced concrete, 120 cm in width and with a radius of curvature varying between 13.55 and 15.79 m.

4.3 *Construction details*

The design stage of the technical details for the connections between invert and sidewalls-base and vault-elements and sidewalls-top was of great importance. Special solutions were developed to speed-up the placement of the prefabricated elements during the works, guaranteeing reliable static efficiency as well. It was decided to guarantee the structural continuity between sidewalls base and the invert (joint constraint), thus providing an overall structural stiffness. The base of the sidewalls was set with two frontal supporting seats – guaranteeing the stability of the elements during the positioning phase – and by leaving the sole steel reinforcements as a web; this allows them to join with the steel reinforcements of the invert, which is later cast during construction at the section linked to the sidewall (Figure 8.).

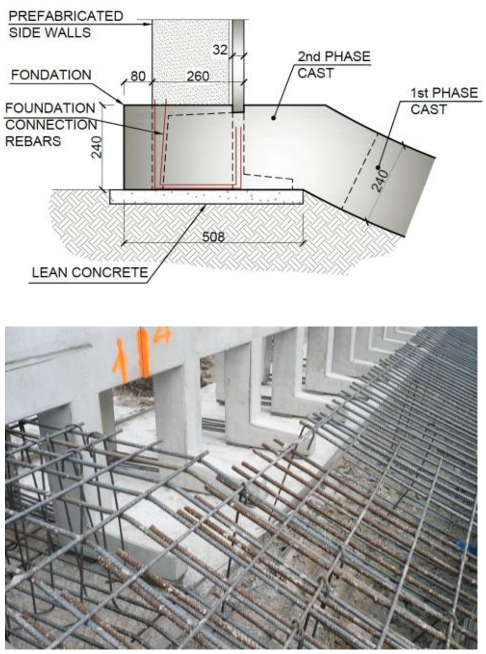


Figure 8. Detail of sidewalls-foundation connection.

A pin constraint was chosen for the vault-sidewall node, capable of absorbing any rotation of the covering element at the support. Indeed, the sidewall requires a full-face section at the top, to grant an horizontal support surface of 86 cm, equipped with a 210 mm thick layer of reinforced neoprene (Figure 9.). The sidewalls and vault elements at the extremity are also equipped with reinforced stirrups that are external to the cast; in this way, once placed together, they can be joined with steel reinforcements lengthwise and further firmly connected by means of an integrative cast while work progresses. Thanks to this longitudinally continuous solution at the height of the sidewalls, alongside the structural continuity at the invert, the structure exhibits sufficient horizontal stiffness when facing seismic activity.

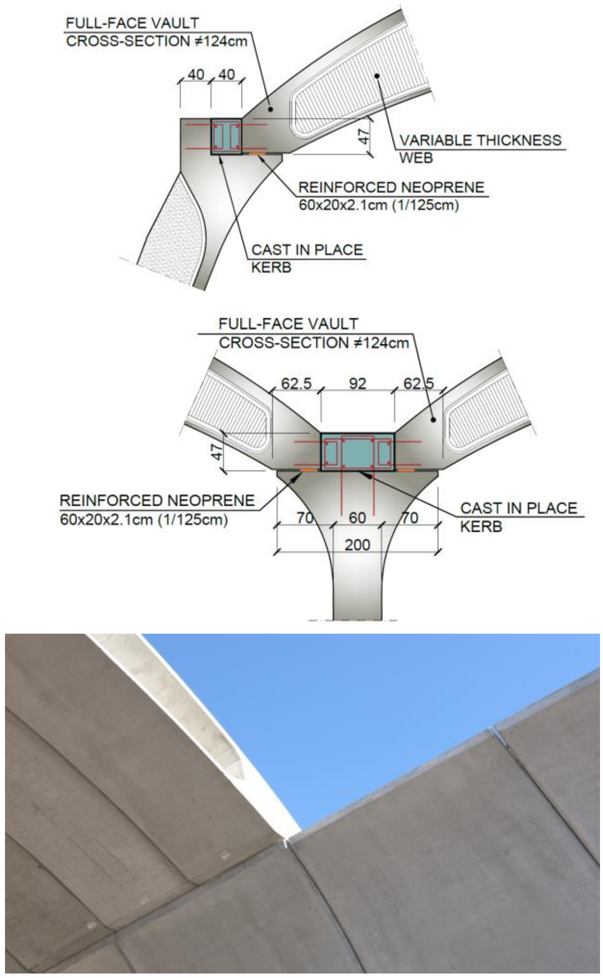


Figure 9. Details of Sidewalls/Piers-Vault elements connection and intrados-view of vault-sidewalls coupling.

4.4 *Waterproofing system*

Despite being above the watertable level, a waterproofing system was put into place in order to protect the tunnels against rainwater infiltration and natural humidity. Thanks to the high quality level of the pre-fabricated concrete walls, a hot-mix emulsion spray waterproofing could be used. This also allowed the use of a specific technical solution for the vertical joints between adjacent pre-fabricated elements, which are often a primary cause of water infiltration. A hot-mix elastomeric insulation was set on the joint surface, the hot-mix emulsion was sprayed, and an HDPE laminate was then placed for the final sealing. A sheet of

TNT 500 gr/sqm was then put down externally in order to protect the waterproofing.

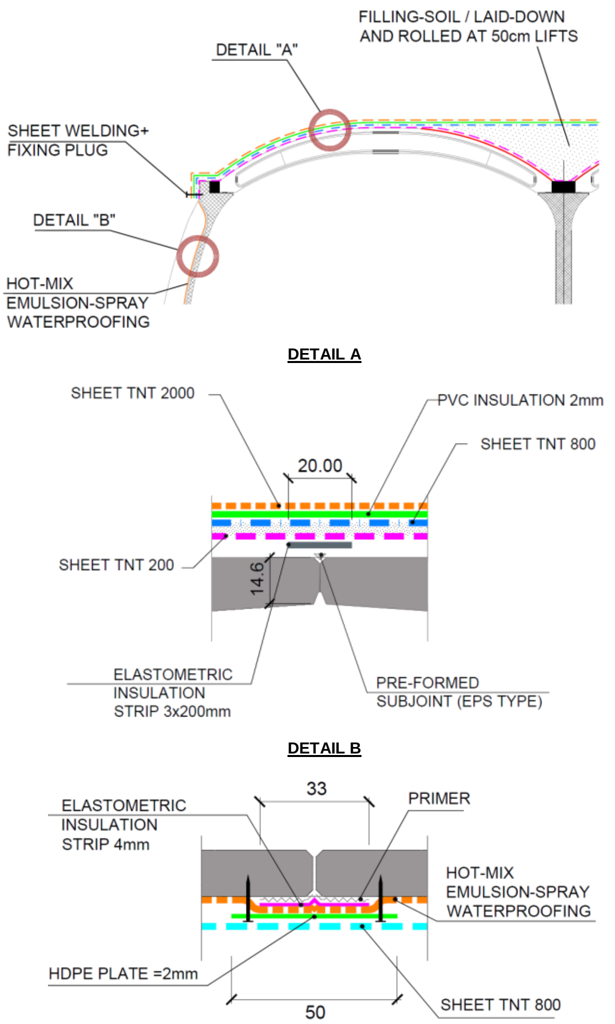


Figure 10. Vault-Sidewalls waterproofing system

A 2 mm PVC insulation was used to waterproof the spherical cap, interjected between sheets of TNT 800 gr/sqm and 2000 gr/sqm in extrados, towards the backfilling. Special care was taken at the joints between adjacent elements, where a pre-formed sub joint was put in place during the work, capable of bearing the backfill loads, as well as an elastomeric insulation strip.

5 CONSTRUCTION PHASES

The first construction phase consists of open-air excavation in order to meet the base levels of foundation inverts. Considering the geotechnical properties of the ground, the excavations mainly followed slopes 1 (vertical) on 1 (horizontal), with intermediate berms every 6 m in height. As it served as a foundation level, the bottom of the excavation after ground-movements completion was protected with a layer of shotcrete and the slopes provided with barbicans. In order to limit the impact of the works on already existing structures (mainly residential buildings and factory structures), temporary retaining structures were set up: tangent piles walls, 1500 mm in diameter, contrasted with tie-rods (Figure 11). The excavation was carried out sector by sector, in order to limit impact on the territory, and to manage local viability with short-term deviations.

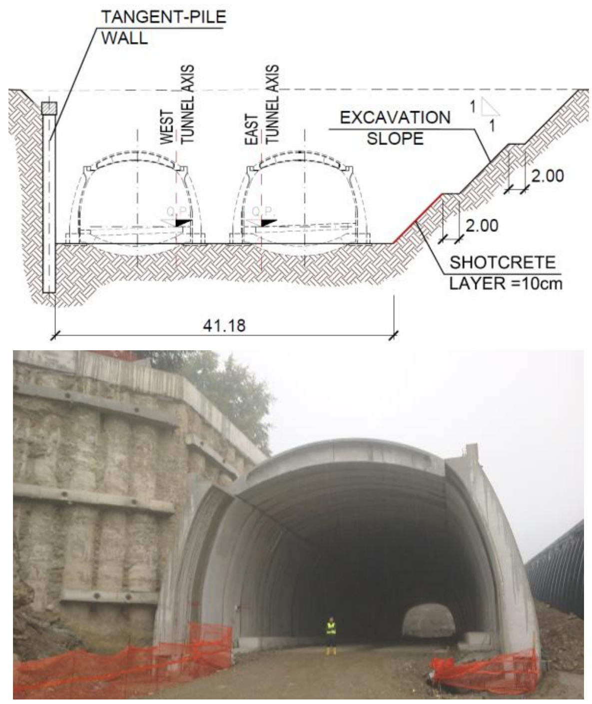


Figure 11. Road-cutting section type: temporary retaining structures (on the left) and excavation profile (on the right) – At the bottom, tangent pile walls

Once the excavation was completed, the structures were constructed according to the following construction phases:

1. Cast of the middle portions of the invert (first phase cast), in order to create a work track for prefabricated elements supply.
2. Positioning of the sidewalls-prefabricated elements, thanks to provisory support structures.
3. Positioning of the steel-reinforcement for continuity between the sidewalls-base and the invert. In addition, final cast of the node between the invert-and the sidewalls (second phase cast).
4. Removal of the supports and positioning of the vault prefabricated elements, placing them at the top of the sidewalls.

5. Positioning the steel reinforcement for longitudinal continuity at the vault-sidewall node, and cast of the solidarising kerb.
6. Setting the waterproofing system.
7. Lateral and top backfilling
8. Completion of interior finishing works (plumbing, motorway, installations).

Positioning tolerance was equal to 10-15 mm, requiring modest joint corrections with specific sealing. As regards the construction times, these average daily production were recorded: 12 m cast-in-place invert, 8 sidewall prefabricated elements, 16 vaults prefabricated elements and 50 m top kerb.

Figure 12. Construction-phases (cutting completion, casting of inverts, sidewalls erection, vault installation and waterproofing, lateral backfilling)

6 BACKFILLING MATERIALS AND METHODS

The methods used for the backfilling, as well as the materials employed, are of the utmost importance due to the “arch” geometries chosen for the artificial tunnels here described, as they determine the static performance of the work.

While the backfilling material above the vault determines exclusively a lithostatic load operating on the structure, the lateral backfilling material (up to the level dictated by the vault) should be considered an integral part of the resistant “structural system”, as it exerts the horizontal confinement necessary for the static effectiveness of the structure. For this reason, the static calculations were carried out taking into consideration not only the strength properties of the backfilling materials (necessary to define the earth pressure on the structure), but also the deformability parameters, in order to evaluate the level of confinement offered by the backfilling to the structure as it is placed in subsequent layers (lifts). Table 3 reports the geotechnical parameters used during the design phase. The geotechnical parameters of the backfilling soils (especially the value of the elastic modulus) depend heavily on the method used for construction, especially on the given level of compaction. For this reason, during the construction, the geotechnical hypotheses utilized were verified in detail, particularly due to the intention of using the excavated soil itself for the backfilling. Ground samples were therefore taken from the short-term storage sites and subjected to laboratory tests to obtain a complete geotechnical.

Table 3. Backfilling soil geotechnical parameters

γ [kN/m3]	c’ [kPa]	ϕ’ [kPa]	E [MPa]	kh [kg/cm3]
20	0	35	70-100	1.0

Classification tests (unit volume-weight, water content, granulometry usingboth sieving and sedimentation methods, Atterberg limits) and soil-mechanics tests (linear cutting TD and triaxial TX-CID tests) were carried out on the reconstituted specimens, drawn from the quartered sampling of the pile. Considering that the grounds were mostly sands and gravels (G+S>70-80%), suitable friction angles were found, within the 33-36° range, linked to a cohesion value < 5kPa (10-15 kPa for the most silty samples). “Proctor” compaction tests were used in order to determine the unit volume-weight-, and to take into account the compaction degree and the percentage of water content.

The unit volume-weight of loose materials remained within the range 17-19 KN/m^3, while values up to 21-22 KN/m^3 were reached at 100% compaction conditions and optimal humidity. Considering how the ordinary humidity level of on-site backfilling isn't above 80-90%, the measured volume-weights matched the design predictions.

In order to determine the deformability parameters of the materials, on site tests were carried out on trial-stretches of regularly backfilled material (necessary to define the confinement degree operated at the sidewalls).

Figure 13. Repetitive Static Plate Load Test- PLT (multiple load cycles) – Testing phases: backfilling compaction, PLT execution.

This was done for the fact that laboratory tests on reconstituted specimens are not appropriate to determine realistic and reliable values of elastic moduli. "Repetitive Static Plate Load Test-PLT" (multiple load cycles) were done, where the measuring of the in-situ density was carried out on site with a sand volumometer (figure 13).

The first cycle generally reached the vertical load associated with the foreseen backfilling height, which is linked to the progressive settling of the filling-soil layers.

These tests were performed on loading-unloading cycles, and allowed the calculation of Young's elastic modulus starting from the reversible elastic component of the vertical displacement (equal to the difference, within each cycle, between the total displacement reached under the maximum load, and the residual displacement at the end of the unloading phase) and implementing Boussinesq's analytical solution reported in Figure 14: "f" is the elastic component of the vertical displacement, "a" is the plate radius, "μ" is Poisson's modulus, and "p" is the pressure exercised on the plate (Croney, 1977; Jeuffroy, 1965).

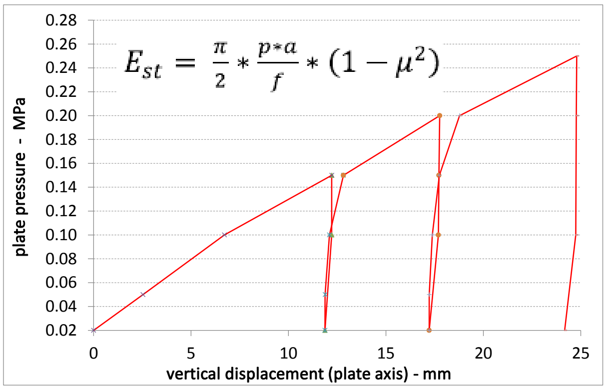


Figure 14. Repetitive Static Plate Load Test (multiple load cycles)

From Young's elastic modulus the horizontal reaction modulus k_h was determined through well-known Boussinesq's equation, $k_h = E / (B (1-\upsilon^2) I_w$, where I_w is the shape factor of the loaded surface, commonly applied for sub-vertical elements such as the sidewalls of artificial tunnels. During the load test, the use of increasing final pressures – linked to rising values of the collapse reversible component (figure 15) – has also lead to determine the subgrade reaction modulus "k_s", equal to the gradient of the line interpolating the experimental pressure-vertical elastic displacement points found from the test cycles.

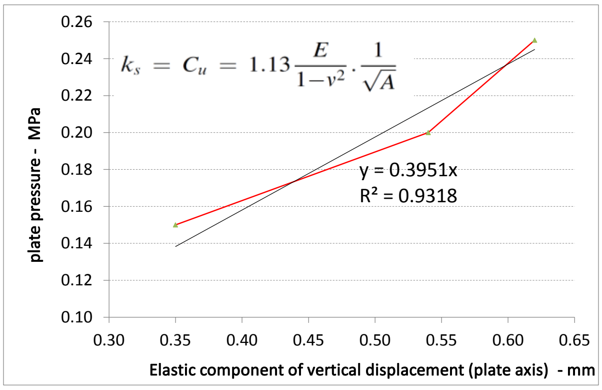


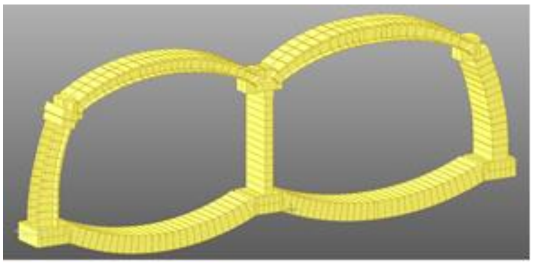
Figure 15. Subgrade Reaction Modulus calculation from Repetitive Static Plate Load Test (multiple load cycles)

Knowing "k_s", it is then possible to find the value of the elastic modulus by applying the Kamesara Rao relation (2000) reported in Figure 15, where ν is Poisson's modulus. The above-mentioned theoretical approached determined very similar Young's Modulus values: for granular materials such as the mainly gravel sandy soils– subjected to the projected backfilling methods on the site (spread and numerous runs by a tracked digging machine weighing 28,000 kg in mass) – the design-values range was reached, with reaction moduli matching the requested confinement action. Instead, for mainly silty materials, lime or cement stabilization techniques were applied, allowing the soil to chemically gain the desired mechanical properties. Thus, these trial-tests both helped to evaluate the properties of the materials used, and the idoneity of backfilling-material layering and compaction methods.

7 STATIC EVALUATIONS

The structural design of the works was performed by applying the new "Technical Regulations for Constructions", Decree 14/01/1008, thus following the Limit State approach. The structural analysis was carried out according to specific finite element numerical models, using the SAP2000 calculation software, using "beam" elements to model the final configuration of the twin tunnels structure. Winkler springs were introduced in order to simulate the interaction between the ground and the structure. These springs reacted solely to compressive stress, and their reaction modulus was 2 kg/cm^3 at the invert (natural ground), and 1 kg/cm^3 along the sidewalls, thus taking into consideration the lateral confinement offered by the compacted ground (see chapter 6).

As permanent actions, the following were considered: the self-weight of the structure, the backfilling load (γ=20 KN/m^3) and the lateral earth pressures, evaluated both with reference to the state of stress at rest (K_o= 0.426), and the one corresponding to the active thrust (Ka= 0.271), thus maximizing the stresses within the sidewalls in the first case, and in the vault in the second. The possible presence of vehicles on the surface was taken into account as accidental actions, thus introducing a uniform loading of 10 KN/m^2 where overburdens are above 2 m (load spreading effect throug the ground cover), or considering the heavy vehicle load, 3 axles 200 KN each, in presence of overburdens lower than 2 m. Then, the possible internal explosion on the sidewalls/pier was measured, as well as the impact of a diverting vehicle (50 KN), the seismic action (assuming the peak acceleration reported in chapter 3), and fire; considering all these, the structures were classified REI120.



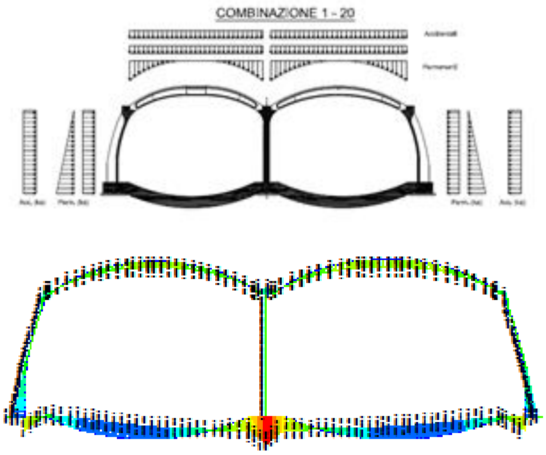


Figure 16. Calculation model, and example of the combination of load and bending stress

The actions here described were analysed with different load combinations, according to the method of partial safety factors. As an example, Fig. 16 represents the calculation model, the load distribution corresponding to one of the examined combinations and the related diagram of the bending moment.

The cross-sections verifications were carried out at the Ultimate Limit States and at the Serviceability Limit States, in particular limiting cracks opening to 0.2 mm. The nodes were verified using local models (of the strut and tie type; e.g. placing a vault element on the sidewall-element according to the stocky-shelf schematic), and so were the local statics on the prefabricated wings as far as the local statics of prefabricated wings. Finally, verifications were conducted also during the temporary construction phases, with reference to the sequence of backfilling stages. Further verifications were carried out for the loading conditions associated to formwork demoulding, transportation and assembly of the prefabricated elements.

8 LOAD TESTS AND MONITORING

The works were monitored, both during the construction phase and during the static load tests conducted after construction was completed. Specifically, monitoring focussed on the movement of the structure under both lateral and top backfilling operations, recording the flexural response of the sidewalls as the load increased.

The load tests were carried out once the backfilling was totally or partially completed, verifying (also) the deformation response of the structure following an accidental load on the surface – simulated with four-axis lorries weighing 500 KN overall, as shown in Figure 17 (the two front axels generate 180 KN, the two back axels generate 320 KN).

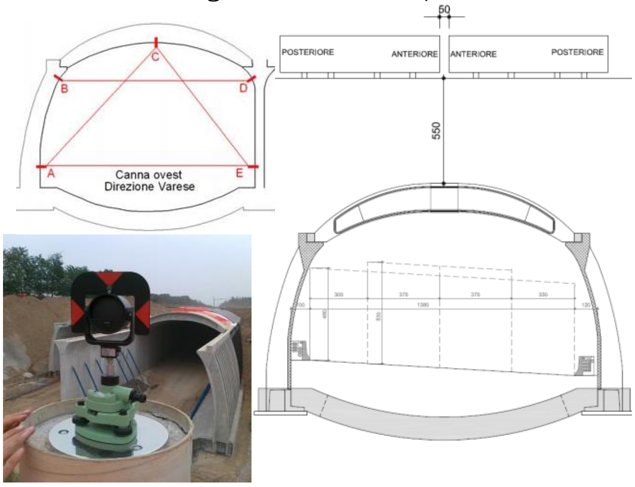


Figure 17. Load Tests and monitoring

Concurrently with the applying of this load, horizontal and vertical displacement monitoring was conducted on the points individuated on the intrados profile of the tunnel, as well as convergence measurements on 4 representative diagonals. During these load tests, bending stress of 85% on the design values was reached. The measured vertical displacement of vault (both during the backfilling phase and during the load tests) was equal to 0.40÷0.60 mm, within the design limits.

The horizontal movement shown by the sidewalls correspond to a structure-backfill interaction slightly below the theoretical conditions of state of stress at rest for the backfilling material set during the works.

9 CONCLUDING REMARKS

During the construction of the new “Pedemontana Lombarda” Motorway, an innovative system was successfully experimented: using prefabricated elements for the construction of arch-shaped cut and cover tunnels, and thus industrializing the construction process in favour of the construction times and the overall quality of the work. The system was applied for tunnel span-widths up to 19 m, in the presence of overburdens ranging from a few metres to 8-9 m. The two motorway carriages – one for each direction – were held in either “separate twin tunnels” or “adjacent twin tunnels”, meaning with a central shared pier. The curved geometries of the prefabricated elements, set on an invert cast during the works, helped to improve the static behaviour of the structure in bearing greater loads than usual box-culvert tunnels. This was made possible also by the collaboration offered by the backfilling material in laterally confinement of the sidewalls. For these reasons it was made necessary not only to carefully study the construction and details of the prefabricated elements (especially the connection nodes), but also the geotechnical parameters of the backfill soils and the operative methods by which they were to be laid-down during the works. The monitoring conducted while the works were underway, and the results of the load tests confirmed the static efficiency of the structures due to the limited displacements recorded.

REFERENCES

Croney, D. 1977. *The design and performance of road pavements*. London: Her Majesty's Stationery Office.

Nuove Norme Tecniche per le Costruzioni. *D.M. (Ministerial Decree) 14/01/2008,*

Jeuffroy, G. 1965. *Conception et construction des chausses*. Paris: Eyrolles.

Kameswara Rao, N.S.V. 2000. *Dynamic soil tests and applications*. New Delhi: A.H. Wheeler & Co.