INTRODUCTION

The road tunnel Vedeggio-Cassarate is part of the new Plan of Transportation of the city of Lugano (PTL), in the southern part of Switzerland and is considered the main object of the first phase of this plan, which will redefine the concept of the public and private transportation policy of the region.

The 2.6 km long double way tunnel crosses the hills of the villages of Comano, Cureglia and Porza and joins the valley of the river Vedeggio on its western portal, with the valley of the river Cassarate on its eastern portal. The main tunnel is coupled on its south side with a safety tunnel and is connected to this one every 300 m with escape shafts. The distance between both tunnels is 30 m.

The different geological and geo-morphological conditions along the tunnel identify three main sections of the structure:
- rock section
- soft ground section
- cut and cover section.

The rock section begins directly at the western portal Vedeggio for a length of 2,370 m. The excavation in this part is carried out in the quite sound crystalline base rock of the southern Alps by drill and blast. The excavation of the rock stretch of the safety tunnel was carried out as first underground activity in 2006, with a TBM dia. 4.50 m, up to 15 m before the passage between rock and soft ground.

Eastwards to this section, the tunnel continues in soft ground for about 200 m. In this sector the distance between the main tunnel and the safety tunnel reduces progressively up to 15.50 m by the east underground portal Cassarate. The excavation occurs in a urbanized area, passing underneath or in near proximity of many buildings, among them a public school.

The final 60 m of the tunnel will be built in cut and cover. This last section is placed in the approach excavation to the east portal Cassarate (Figure 1). In this section the main tunnel and the safety tunnel will be jointed together in a single reinforced concrete structure.
GEOLOGY AND HYDRO-GEOLOGY IN THE SOFT GROUND SECTION
The geology of the soft ground section is presents a very heterogeneous sequence of quaternary glacial lake and fluvial sediments, formed mainly by a matrix of silty sands or gravels with little content of clay in a large consolidation range, going to the weak condensed silty sands to the quite pure over-consolidated silts. In this matrix are incorporated deposits of sand and gravel in variable quantity and thickness. The overburden in this section increases from 5 m right after the portal up to 40 m by the rock limit.

The permeability of the ground may be generally defined as small, with layers of greater permeability, along the early meanders and beds of the river Cassarate. In such a ground structure the ground water may be encountered in artesian conditions at many different independent levels with pressures reaching up to 20 m above the ground surface. Generally the geotechnical properties may be described between sufficient and poor and the hydro-geological conditions as extremely complex. Figure 2 shows a simplified geological model of the ground along the alignment of the main tunnel.

Figure 1 - Portal Cassarate with underground portals: left safety tunnel, right main tunnel.

Figure 2 - simplified geological longitudinal section.
As shown in the figure 2, three main ground typologies may be recognized. Beginning from the tunnel portal these are:

- a superficial sequence of weak consolidated silty sands and gravel with low permeability
- a layer of over-consolidated silt with little gravel, very stiff and with very low permeability
- deep silty gravels till gravelly silts and sands, with greater permeability.

**CHOICE OF THE EXCAVATION METHOD**

Due to the very heterogeneous and variable geological and hydro-geological conditions, since the earliest design phases it clearly emerged the necessity to choose an extremely flexible excavation method, which could be rapidly adjusted on site to the actual encountered geological and hydro-geological situations assuring a sufficient stability of the excavated tunnel section and, at the same time, reducing the impact of the same excavation on the surroundings (settlements). This implies the previous execution of systematic drilling holes in advance of the excavation front, in order to investigate the real ground conditions and to adapt the most adequate executive solutions for the continuation of the works.

*Tunnel advance with jet-grouting*

The targets mentioned above may be reached through the previously consolidation of the ground, in which the excavation should take place. With this method it is possible to reduce the global permeability and to improve the geotechnical properties of the ground, preventing, on one hand, the uncontrolled drainage of the groundwater and, on the other hand, restraining the deformability of the ground around the excavation.

After the analysis of different possible techniques, it was decided to apply the jet-grouting method for the consolidation of the soil. Considering the morphology of the ground surface, which on the first meters of the tunnels shows an almost flat slope, it was chosen to carry out the jet-grouting works from the ground surface over the maximum possible length and to continue on the remaining section operating from the underground, i.e. directly from the tunnel face to consolidate the soil in advance of the excavation. Figure 3 shows a situation of the tunnel with the distribution of the adopted soil treatments, i.e. from the ground surface and from the underground.

*Figure 3 - Situation of the soft ground section: representation of the applied consolidation methods.*
The consolidation from the ground surface was achieved with vertical secant jet-grout columns (2-Phase grouting) with a dia. of ca. 1.50 m. The treatment from the surface could be carried out over 30 m along the alignment of the main tunnel since along the alignment of the safety tunnel this had to be limited to 10 m. As visible in Figure 4, the results of this soil treatment were extremely satisfactory.

![Figure 4 – vertical jet-grouting behind the demolished pilewall.](image)

As mentioned above, after this first section, the consolidation of the ground was carried out from the tunnel face. For the safety tunnel as well as for the main tunnel this goal was achieved through the execution of an arch of sub-horizontal jet-grout piles around the tunnel perimeter to assure the stability of the excavation. The stability of the tunnel face was guaranteed from the implementation of others horizontal jet-grout piles in the tunnel face.

The jet-grout arch was composed from secant sub-horizontal piles of ca. 0.8 m dia. and 15.0 m length, reinforced with steel pipes bored and injected in their center in a successive phase but before the beginning of the excavation activities. The overlap between the piles of two successive jet-grout arches reached 5.0 m. The jet-grout arch of the safety tunnel was composed of 27 piles, while to complete the one of the main tunnel were needed 37 piles. The face stability was assured with 6 piles in the safety tunnel and with 10 piles in the top heading of the main tunnel.

The excavation was carried in steps of 1.0 m with the installation of steel ribs at the same spacing. In the safety tunnel the support was assured by steel ribs HEB 180 and 20 cm shotcrete reinforced with steel fibers, since for the main tunnel the support consisted in steel girder type 4G-180/36, with two layers of wire mesh and 35 cm shotcrete. A complete excavation stage had a length of 10.0 m (10 excavation steps of 1.0 m).

While the advance of the safety tunnel was carried out with a full face excavation, the advance of the main tunnel was accomplished in two stages with the top heading section foregoing the bench and invert section of about 30.0 m. Due to the poor geotechnical conditions and to the large dimensions of the tunnel section, the feet of the crown primary lining have been systematically underpinned with a sub-vertical jet-grouting pile every 1.0 m on both sides of the crown. These piles have a length of 6.0 m and are reinforced with steel pipes. These piles have the purpose, on one hand to
lead the support reaction forces into the ground and, on the other hand, to consolidate the soil in the lower part of the section, in order to reach a sufficient stability for the excavation of the invert.

A schematic description of the principle of the excavation method is shown in figure 5, on the example of the safety tunnel.

Before the start of the jet-grouting works of every excavation stage, the soil and groundwater conditions along the two following excavation stages were investigated through the systematical execution of 20.0 m long drilling holes, which could also be used, upon necessity, to drain a limited ground portion or single ground layers or lenses.

On the base of the results of this soil investigation it was possible to define every time the requirement of supplementary treatments in addition to the ones of the standard procedure for the continuation of the works. In case of very poor ground conditions, additional piles were carried out in the tunnel face, in order to assure its stability. In case of water inflows, in the lower portion of the tunnel section – eventually under pressure - it was possible to seal the soil in the area of the tunnel invert with additional jet-grout piles, as shown in figure 5. By heavy water inflows from the tunnel face the design provided the possibility to seal the ground with a jet-grout screen of 3.0 m thickness, executed at the end of the following excavation stage, as shown in figure 5. In this way the excavation could be carried out in a practically closed and sealed jet-grout chamber.

During the excavation of the safety tunnel the first mentioned additional treatment, i.e. the sealing of the tunnel invert had to be applied several times, since the jet-grout screen at the end of the following excavation stage could fortunately be avoided. The entire excavation of the main tunnel could be carried out without any additional treatment, due to the fact that the ground was previously drained from the earlier excavation of the safety tunnel.

In any case, with or without additional treatments, after the conclusion of the jet-grouting activities and before starting the advance of the following excavation stage, additional 10.0 m long drainage holes were drilled in the tunnel face in order to dewater the soil, in which the excavation had to occur. In this way it was possible to locally sink the groundwater pressure, increasing the face stability during the excavation and allowing working in a non-saturated ground (absence of groundwater pressure). The picture in figure 6 shows the face of the safety tunnel with a graphical superposition of the jet-grouting scheme, in the area of the passage between the superficial brown silty sands and gravels and the grey over-consolidated silts (lower portion of the tunnel section).
Excavation with forepoling

For the excavation in the over-consolidated silt, the consolidation of the soil with jet-grouting was not possible because of the considerable stiffness and extremely low permeability of this type of ground, which does not allow the grout jet to penetrate sufficiently into the ground. However, these same characteristics made any such treatment unnecessary, limiting the interventions to assure the sole stability of the excavation and of the tunnel face. For this reason, in this section it was decided to apply a classical forepoling coupled with Glass-fibers Reinforced Plastic bolts (GRP) placed into the tunnel face.

The forepoling consisted in an arch of injected steel pipes with length of 13.0 m put in place on the outer side of the tunnel perimeter at a distance of 0.50 m one to each other. As already mentioned, in order to assure the stability of the tunnel face the design provided the use of GRP-bolts, injected on their whole length with cementitious grout. However, these were applied only during the excavation of the safety tunnel. In fact, the very good experience gathered during the excavation of the safety tunnel in this soil, made possible to avoid the application of the GRP-bolts during the excavation of the main tunnel, allowing limiting this measure to the excavation stages at the end of this stretch.

Otherwise, the excavation method, the length of the excavation stages, the composition of the primary lining as well as the concept of pre-investigation of the ground conditions remained the same as the ones adopted for the excavation with the jet-grout treatment.

A schematic description of the excavation with the forepoling is given in figure 7 showing for example the main tunnel section.
CONCEPT OF THE MONITORING AND RESULTS OF THE SURVEY

The excavation of tunnels underneath an edified area imposes the systematic survey and monitoring of the structures present in the influence zone of the works. By critical geological and hydrogeological conditions, as the ones encountered in the soft ground stretch of the tunnel Vedeggio – Cassarate, this activity should be constantly coordinated and followed through an efficient operational team, which should be able to decide rapidly and put into action in a short time previously planned measures and interventions, on the base of the results of the survey and the monitoring activities. The team should include every entity involved in the excavation works and in the survey activity, beginning from the customer up to the designer, the contractor and the whole site management.

Such an operational team was created for the construction of the Vedeggio – Cassarate tunnel. The survey and monitoring system of the soft ground section was managed in the following way:

− Group 1: parameters addressed to the safety of the excavation works,
− Group 2: parameters forwarded to the survey of the existing buildings in the surroundings and consequently of the safety of thirds.

For each type of parameter, warning and alarm limits have been fixed. Depending on the reached value and the type of the surveyed parameter, the attainment of one of these limits imposed the adoption of a previously planned measures or interventions, described in a specific document in possession of each member of the operational team. In this document were also defined and described the responsibilities and the tasks of every member of the operational team.

The constantly monitoring of the entire range of parameters gave to the design engineer precious information to compare with the results and with the assumptions made for the execution of the theoretical analyses carried out in design stage. In this way it was possible to better judge the measured deformations and settlements and to predict, with more precision, their future development. Further, the possibility to have such a general overview of the behavior of the ground represents a decisive help to estimate the plausibility and to comprehend eventually extraordinary events or unusual single measures.

The surveyed parameters for each of the above-mentioned group are gathered in the following.

− Group 1: Monitoring of the excavations
  • convergence of the tunnel section (1 section each 5 m)
  • level measurement in the tunnel (together with the convergence sections)
  • vertical multi-base extensometer (from ground surface) for the survey of the ground deformation in front of the tunnel excavation face (7 pieces)
- level measures of the ground surface along and across the safety and the main tunnel alignment, (coupled with the convergence sections and the extensometer) (28 points)
- changes of the groundwater situation with piezometers (9 pieces, many with more measure cells).

Group 2: monitoring of the existing buildings
- settlements of buildings, roads etc. for the evaluation of absolute and differential settlements through level measures (ca. 28 points)
- settlements of buildings and critical areas with a total station (automatic theodolite - 14 points)
- survey of development of existing cracks and fissures of buildings (13 measure points)
- behavior of slopes with slope-indicators (3 pieces).

The results of the monitoring and survey activities were saved in specific databases managed through the software SISO©.

Each person charged with the survey could insert or transmit the results of the measurements in electronic format directly to the software, if this was not done automatically from the measuring instrument.

After a first plausibility examination on the base of pre-inserted criteria for each parameter or measure point, the software SISO© evaluates immediately the data. This is carried out through the confrontation with the pre-inserted alarm limits and afterwards through the plot of simple graphic representations.

If one warning limit is reached the software can launch different types of alarm, going from the sending of e-mails, up to the sending of SMS to the operational team members, depending of the reached value and of the type of parameter. The behavior of each single parameter may be asked at anytime through internet. The access is restricted to the members of the operational team.

The extreme flexibility of this system allows the control of the general behavior of the area in real time and therefore to react very quickly in case of extraordinary events. At the same time it is possible to survey the development of the ground settlements on a great scale. This permits to early recognize the necessity of additional actions or treatments to carry out within the tunnel excavation.

Figure 8 shows a typical chart of settlements measured with the automatic geodic instrument.

Figure 8 - typical chart of piezometer (red) and settlements (green, yellow)
The survey and monitoring activities have shown a global situation, which corresponds to the theoretical forecasts. In fact, until the present stage of the excavation (see next section) no alert level has been reached and no intervention was necessary. The absolute maximum value of the settlement has reached 80 mm in a isolated level point, since the maximum settlement of the buildings near the tunnel excavation reaches 50 mm, although the differential settlements remained always underneath the admissible values.

As shown in figure 9 thanks to the monitoring it is possible to distinguish the influence of the different activities, i.e. of the drainage of the groundwater and of the excavation of the tunnel, on the points on the ground surface, and to face them, as in this example, to the behavior of piezometer standing. Thanks to the adopted software, the assembling of such kind of correlations between all the monitored parameters is possible within a short time on the base of the fresh gathered data.

STATE OF THE WORKS
The excavation of the safety tunnel was completed in autumn 2008, with the breakthrough on November, 26th, 2008.

At the end of February 2009 the excavation in the rock section of the main tunnel reached ch. 2'300 since the excavation in the soft ground jointed the ch. 170. The breakthrough of the main tunnel is expected on the middle of March 2009.

CONCLUSIONS
The excavation of tunnels in critical geological and hydro-geological condition in urbanized areas must be faced since the early design stages, with the target to recognize and to chose the most adequate excavation method, which can assure a sufficient safety during the excavation and which may be quickly adapted to heterogeneous soil and groundwater conditions and to eventual emergency situations.

At the same time, it is essential to dispose, on one hand, of sound theoretical analyses to allow the initial setting of alert and alarm levels, as first reference to judge the real behaviour of the ground and, on the other hand, of an efficient survey and monitoring system, as well as an effective operational team, involving the customer, the designer, the contractor and the whole site management, which must be able to take decisions and act rapidly, according to the actual behaviour of the ground, applying the executive solutions foreseen by the design. These solutions must be developed as part of the overall design of the underground works.

REFERENCES