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# Tunnel renovation by means of precast concrete structural elements

## Application examples and project process of a re-lining intervention

Supervisors:
Ing. Daniele Martinelli
Ing. Vincenza Floria

Student:
Fabio Mangaretto

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“Tunnels remain the most resilient, sustainable and cost-effective civil engineering construction ever devised - as long as they are maintained well.”

Cit. Institute of Civil Engineering of UK.

# Abstract

In recent decades, the technologies and requirements for the construction and maintenance of tunnels have changed to improve several aspects, both in the construction phase and in the operational phase. In order to take these aspects into account, not only the way of designing and building new infrastructures has been changed, but it is also necessary to take action on existing infrastructures to satisfy new requirements in terms of safety, efficiency, sustainability, durability. The need to carry out renovations has led manufacturers and designers to search for the best available technologies and methodologies, with the aim of achieving the fastest, cheapest, and least impacting modes of intervention. In this thesis, some innovative application cases made in recent years for the renovation of existing tunnels have been examined. Moreover, the design phases of a case study under development have been analyzed in detail, in order to deepen general technical aspects related to the tunnel re-lining.

# 1 Introduction for tunnel refurbishment

Many of the tunnels realized in the middle of the 20th century were built frantically riding the wave of economic boom, this has led to partial or total disregard for the quality of the work. These tunnels were built with no or minimal supports and using poorly resistant or unsuitable materials (unreinforced concrete, bricks, stone masonry, etc.), moreover they have no waterproofing or fireproof measurement, the drainage system was very simple and design life unknown. Often, little or no maintenance has been undertaken and the monitoring systems are poor or completely absent. However, these old tunnels are often realized in strategic positions and have an important economic and social role, for these reasons are still widely used and often undersized compared to current traffic. The increasing of the trade over land and of the traffic caused the question of under-dimensioning of both road and rail tunnels built in the mid-900, but also in more recent years.
Even the most recent tunnels, mainly built in the second half of the 20th century, often no longer meet current safety requirements or, due to external agents, no longer benefit from the initial structural performance. A clear example of these problems is represented by the numerous road and motorway tunnels of the Italian road network. Often, they have evident signs of deterioration caused by different causes: chemical and physical agents, earthquakes, or daily wear due to the increasing of the traffic. It is no coincidence that many of the case studies below deal with tunnels used for vehicular traffic.
Sometimes, there are newly built tunnels that due to design errors or wrong geology and hydrogeology assessment, require remediation measures. These types of interventions are generally aimed at improving a single aspect and require a very specialized technology such as grouting, bolts installation, special drainage systems, etc.

Nowadays, tunnels are constructed using several different structural elements each with specific functions and features. These elements are often combined to ensure maximum efficiency, safety, and to improve the service life. The structural elements are mainly divided into two categories: temporary and permanent. The first are used in the construction phase to ensure short-term stability and to allow the advancement of the excavation. The permanent elements, on the other hand, constitute the final lining, and therefore must ensure long-term stability, avoid water coming into the tunnel and meet all the specific needs of each case. Some of the most used structural elements are:

- Rock bolts
- Sprayed concrete
- Cast in place steel reinforced concrete
- Precast steel reinforced concrete
- Spheroidal Graphite Iron (SGI) segments

Tunnel design always requires a detailed maintenance plan to ensure the expected service life (usually for a minimum of 120 years), and a monitoring system must be provided to analyze the main aspects, both structural and non-structural.
The causes that lead to a structural intervention can be many and of different kinds. However, in the following list, a number of more common causes it is reported.

- Aging of the original structure, or need to extend the service life
- Deterioration of materials:
    - Concrete:
        - Physical: frost-thaw cycles, bumps, fire
        - Chemical: the carbonation and chloride attacks
    - Steel: corrosion
- Presence of asbestos
- Changes in state of stress due to:
    - Artificial events: creation of new infrastructure affecting the tunnel (ex: construction of an artificial basin), interventions on the tunnel itself (ex: waterproofing with hydrogel) or on near infrastructures (ex: buildings increasing the load),
    - Natural events: floods, landslides, earthquakes, fire. These events can compromise the stability of the tunnel by deteriorating existing structures and/or changing the original state of stress.
- Necessity of enlarging of the cross-section (ex: increased traffic, electrification of railway lines).
- Health and safety issues
- Change of use
- Modifications required

Depending on the situation, the renovation can be localized in specific parts of the tunnel or along its entire length. This distinction, although it may seem trivial, radically influences the choice of the method of intervention and realization.
In case the refurbishment affects only a small part of the tunnel (or the tunnel to be refurbished has a limited length), it is suitable to carry out the intervention by using small and versatile equipment in order to reduce the operational costs. Moreover, the time needed for the work is generally shorter and the closure of the tunnel is limited to some hours or few days. On the other hand, if the refurbishment involves several meters, it can be convenient to have a more specific and mechanized equipment able to speed up the working phases. This type of equipment, together with the optimization of the construction process, would reduce the impact of the working site on the operation of the tunnel and, sometimes, also to avoid the total closure of the tunnel during the intervention period.

# 2 Methods for tunnel refurbishment

The refurbishment of a tunnel may involve several aspects, both concerning the structure and other components of the work (i.e., fire-fighting system, ventilation system). Depending on the restoration to be performed, it is possible to select the most suitable method to adopt. The choice of the method is also affected by the site conditions and by the economic and social impacts that the respective construction site may have. The main structural measures carried out to modernize old tunnels are listed and briefly described below. Some of these interventions involve precise and localized structural aspects, while others are more general and require more complex design studies.

## 2.1 Strengthening

Strengthening is one of the main interventions of tunnel refurbishment. It consists of a set of interventions aimed at improving the mechanical characteristics of the structure and the surrounding soil or rock. These interventions can be carried out individually for targeted works, or they can be part of a more general renovation, in the second case they contribute, together with the other structural elements, to the improvement of the structure.

1. Filling voids
Due to the presence of water and its phenomena of transport and corrosion, it is possible that with time there may be voids in the area surrounding the tunnel. This problem is very frequent, for example, in karst environments where the dissolution of the material in contact with water causes the formation of voids that generate structural problems to the tunnel and, in the absence of a proper drainage system, it can lead to water leaks inside the tunnel.
The grouting can be performed with different techniques and different materials according to the type of soil and the purpose of the operation. Generally, the main objectives consist in waterproofing and improving the mechanical properties of the treated soil. To achieve these conditions, it can be suitable to perform a permeation grouting with a combination of micro-cement together and colloidal nanosilica or resin (depending on the granulometry of the soil). The pressure of the grouting is an important design parameter that must be correctly evaluated in order to fill all the voids without causing the “claquage” of the soil. To better control the injection pressure and location, a manchette pipe with a double packer can be used.
During the grouting process it is important to take under control the injection location in order to avoid the clogging of the drainage system. If it is not possible to ensure it, the drainage system should be restored at the end of the injection.

2. Internal supports
In case the main issue of the tunnel consists in its stability, internal supports can be added. These structural elements, generally, consist of steel ribs installed on the tunnel

vault and consequently covered with a layer of shotcrete. This intervention is often used in conjunction with other systems of reinforcement and waterproofing.
Steel ribs are elements consisting of arc-shaped metal profiles and electro-welded mesh that will form the structure's reinforcement. These steel elements must be fully covered with the shotcrete layer to prevent corrosion.

3. Ground anchoring and rock bolting
Another intervention aimed to ensure the tunnel stability, consist in the installation of anchors or bolts at the crown and at the walls of the tunnel. The bolting has the goal to stabilize unstable wedges in rock masses.

4. Underpinning
This intervention consists in improving the foundation of the lateral feet of the tunnel through the insertion of metal structural elements or the realization of jet-grouting columns.

## 2.2 Invert replacement and/or lowering

The invert can be replaced in case of deterioration or in case it is necessary to lower it, the structure is normally cast in place, but it can be also precast. When there is a problem of vertical space inside a tunnel, instead of raising the ceiling, it can be convenient and easier to lower the invert by excavate the existing floor. In other cases, the invert can be forced up by ground pressure due to bottom heave (ex: Netherton canal tunnel). In these cases, the invert must be replaced with a more massive structure able to face the ground pressure.

## 2.3 Re-lining with traditional method (concrete cast in place)

Re-lining is a method of tunnel renewal that consists in completely or partially replacing the existing coating. This method is usually applied in situations where the existing lining does not meet safety or waterproofing requirements. The traditional method of re-lining consists in the total or partial removal of the existing coating and the construction of a new reinforced concrete structure cast in place. According to the requirements of each case, it is possible to have many different interventions aimed to the renovation of the whole structure.

Usually, the first step is to remove the existing concrete layer. To estimate the thickness of concrete, preliminary analyses must be made, this type of analysis consists in the coring of samples and/or the execution of geophysical investigations. The removal of the old lining is generally carried out by means of milling machines able to scratch up to reach the wanted thickness. However, to obtain the removal of the old lining, also other machines can be adopted depending on the characteristics of each case: hydraulic hammers, roadheaders, hydro-demolition machines. This phase can also not be implemented, in which case the new lining is installed directly on the old one. The choice not to remove the old lining is motivated by the reduction in cost and time of the intervention, although a decrease in the useful section of the tunnel must be expected. Subsequently, it should be necessary to smooth the surface before the installation of the new waterproofing system consisting of a multilayer sheeting. Then it can be proceeded with the construction of the steel reinforcement on the vault of the tunnel and finally, by means of a moving formwork, the concrete casting is carried out on the entire vault.

This method allows to radically refurbish the structure of the tunnel in a fairly simple and repetitive way. However, this method has some disadvantages due to the timing of the work. In fact, the demolding can happen only after the concrete has reached a good mechanical strength and this can take even a few days. In addition, the installation of steel reinforcement bars must be done with extreme caution to avoid damaging the waterproofing sheets.

An innovative application of this method was adopted for the renovation of the Federal Roads Office (FEDRO) tunnel located in Ittigen, Switzerland. The intervention was carried out on the 196-metre tunnel T3 on the A16 motorway with the challenging goal of creating a new lining reducing the impact of work on daily traffic as much as possible. The existing coating was realized between 1960 and 1970 with non-reinforced concrete cast in situ directly against the rock, i.e. without a waterproofing system. The procedure involved the following steps:

- Installing an anchoring system
- Milling the non-reinforced concrete arch
- Installing a drainage layer across the entire surface
- Applying a new waterproofing system using two different options (each across a length of 100m):
    - a spray-on liquid waterproofing system
    - conventional synthetic waterproof sheeting
- Casting a new inner ring.

All work was carried out at night from 8.30pm to 5am five days a week. The work was completed in March 2020 after a total duration of twelve months.
Milling operations carried out over the tunnel arch inevitably weaken the existing coating and thus reduce the stability of the tunnel itself. However, the thickness and quality of the existing inner shell were known from numerous core samples taken before milling work began. At the end of the milling operations, the remaining inner shell profile was measured, and periodic measurements of the convergence were made at the same time. Before the tunnel was opened to traffic every morning, temporary rock nails were systematically installed to ensure the stability of the weakened arch to reduce the pressure of rock formation on the milled arch. Specialized milling machines were used to disassemble an unreinforced concrete ring in a controlled manner and ensure accurate and meticulous results. They also ensure a uniform finish, which facilitates the application of subsequent layers. The rate of progress achieved in this project was about 2m per night. However, it was not possible to use this technique at the entrances to the tunnel as the coating tends to consist of reinforced concrete in these areas. In this case the hydro-jet method was used to remove the coating.

*Figure 2.3-1 – FEDRO Tunnel: milling of the old lining*

After the milling phase it was possible to install the waterproofing system consisting of conventional waterproof synthetic sheets for a certain length and a liquid spray-on waterproofing system for the remaining part of the tunnel. Once the waterproofing was installed, a new 20cm thick arch of self- compacting concrete was installed to permanently support the tunnel. The new arch was cast in place in a single pass at night when the tunnel was closed to traffic, using a mobile formwork. Special anchors were inserted at the base of the tunnel formwork to counteract the pressure of the self-compacting concrete as it was being poured and at the same time to stabilize the existing arch after removing the abutment in order to install the drainage pipes. These anchors make it possible to minimize the dimensions of conventional tunnel formwork, which would otherwise not permit two lanes of traffic to pass through during the day. The time required to complete the laying of the new inner shell was 7 weeks, with a production rate of one 5-metre block per night. During the T3 tunnel refurbishment, an innovative solution that would also maintain traffic flow in both directions during the day was tested on three sections of concrete by using an ultrafast, self-compacting concrete. During casting, temporary mobile horizontal supports were used to offset the fresh concrete pressure and then be removed before the tunnel is opened to traffic. By this time the concrete in the inner ring had acquired specific mechanical properties (this happens within approx. 2 to 3 hours for a concrete strength $f_c > 20$ N/mm2). The tunnel formwork remains in place during this time and is not removed until the next day. Thanks to the formwork special characteristics the traffic can flow unrestricted during the day.

*Figure 2.3-2 - FEDRO Tunnel: casting of the new final lining with a moving formwork*

This refurbishment project is a good example of optimizing the building process. Thanks to the carefully controlled and safe partial removal of the primary lining, in combination with the installation of a special formwork, it was possible to keep all lanes open during the day.

## 2.4 Re-lining method with precast segments

One of the purposes of this thesis is to analyze the methods and the technologies to achieve the refurbishment of tunnels through prefabricated elements. This method can be applied both in case of re-lining, and in case of enlarging of the tunnel section. This type of intervention involves the complete or partial removal of the old lining and the installation of a new lining composed of prefabricated reinforced concrete elements. The main advantage of this type of intervention is the possibility to reduce the installation time and consequently to reduce the social and economic impact of the construction phase. This approach has developed in recent years as a result of a growing and widespread demand for re-lining of conventional tunnels. This refurbishment method turns out to be particularly effective regarding interventions on road tunnels, in fact it concurs to speed up the process of installation going to diminish the times of yard and consequently the impact on the traffic. The prefabricated elements consist of reinforced concrete slabs which are very similar to those used by TBM. The process of construction, transport, stacking, and installation is comparable to the process that takes place in the case of the construction of mechanized tunnels. The prefabricated segments can have different shapes and dimensions according to the tunnel geometry and to the choices adopted in the project, however there are features that are always present on the prefabricated elements. A key aspect that concerns all types of prefabricated segments for tunnel refurbishment are the inserts. The inserts are metal structural elements suitably placed on the segments that allow them to be moved during the different steps of the working execution and to facilitate the structural connections during the installation.

This method includes all the interventions that use prefabricated elements, but as can be expected, there are several applications with different characteristics. Some applicative examples are reported below in order to better describe the method and other examples are described in the following chapters.

The first example is a temporary safety intervention that was carried out on the Monte Galletto tunnel of the A7 Milan-Genoa Motorway. The infrastructure is located in a difficult geological context, characterized by the formation of Montanesi clays and, for this reason, the tunnel has already undergone several updates over the years. Following the inspection, significant defects in the shell coating were found which required a series of structural safety measures. Such interventions were carried out in 2020 and consist in the reconstruction of the tunnel vault, from the top of the right sidewall to the top of the left one. The reconstruction took place in accordance with the following stages. The first phase consisted in the execution of radial consolidation in order to preliminary improving the geotechnical characteristics of the surrounding soil before the milling phase. The consolidation was achieved through the drilling and insertion of 3 m long VTRs grouted with one-component premixed mortars, in the most critical areas further consolidation and additional drains were provided. The next step was to demolish the vault coating. First of all, cuts were made in order to delimit the areas to be milled. Subsequently, these areas were demolished to a maximum length of 7.2 m (equivalent to three 2.4 m prefabricated blocks). The milled thickness was between 40-50 cm and the demolition was carried out by means of milling machines. The sidewalls of the existing lining were left intact in order to have a support point for the segments to be installed. Following the eventual installation of a layer of fiber reinforced spritz beton to regularize the milled surface, the waterproofing system, consisting of a drainage panel coupled with PVC + nonwoven geotextile, was laid.

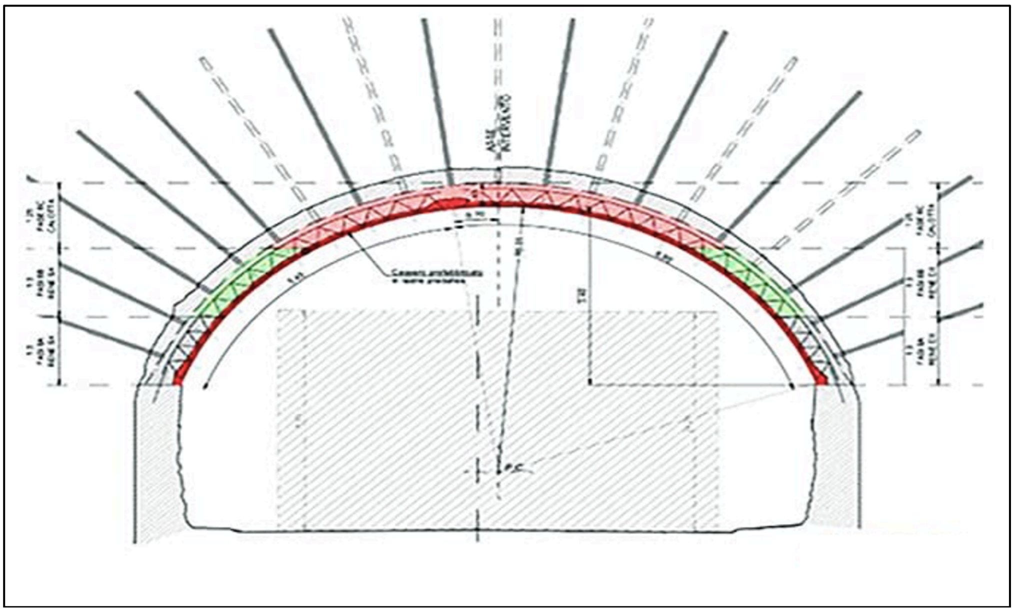


*Figure 2 – Monte Galletto Tunnel: design of the slabs installation over the existing sidewalls with sinking of reinforcement bars*

The prefabricated slabs were made in two different stages: the concrete part was made in a prefabrication plant, while the steel reinforcement was mounted directly in situ, on the slabs extrados. The assembly of these two components was made at the ground level, just before the positioning of the slab on the tunnel vault by means of special forklifts. Chemical resins were then used to fill the vertical connecting bars on the existing sidewalls. In addition, the preparation of the support surface of the prefabricated segments was carried out, by cleaning the head of the sidewalls, wetting, and laying of a layer of grout (5 cm). The slabs have been positioned in pairs so that, thanks to the connection to the top of the vault, they are able to self-sustain in a temporary phase. Finally, it was proceeded with the injection of the concrete in the gap between segments and waterproofing, adopting mixtures characterized by fluidity, quick setting, and development of mechanical strength.

*Figure 3 – Monte Galletto Tunnel: slabs installation and waterproofing sheets*

Another interesting example with a different type of application of the precast elements is the Glatschera tunnel located in Bergün, Sitzerland. This example constitutes the pilot project of a general renovation plan established by the Rhaetian Railway for the refurbishment of more than half of the tunnels present on the railway network (which counts 115 tunnels). To face this huge plan, the Rhaetian Railway developed a new, standardized method of refurbishment that guarantees regulated building procedures. Thanks to this newly developed standard construction method, the quality and cost-effectiveness of tunnel renovations will be improved, the safety standard increased, and the service life of the tunnels raised from the current average of 30 to 50 years to 70 to 100 years. The aim of the renovation project was to realize a complete re-lining of the tunnel by enlarging the useful section, lowering the track, to implement a new drainage system and to replace the entrance portals of the tunnel.

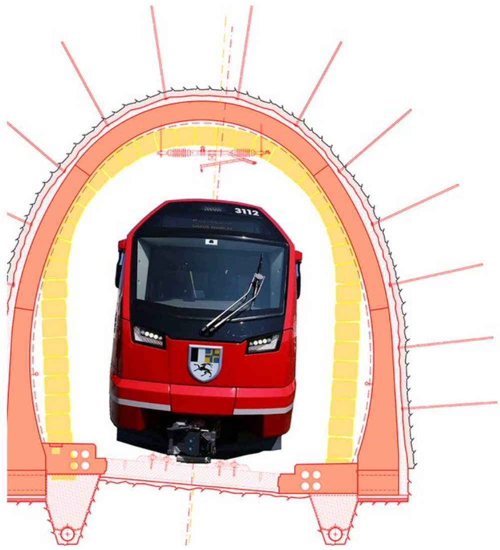


*Figure 2.4-1 – Glatschera Tunnel: technical drawing of the tunnel section showing demolished parts (yellow) and new elements (red)*

The first phase of the works concerned the removal of the old coating that was made of stone and did not include any waterproofing system. After the old coating was removed, it was proceeded with the enlargement of the tunnel section with drill and blast method. Subsequently, following the mucking of the blasted rocks and the scaling, a layer of shotcrete was provided to uniform the surface of the vault. At this point a rockbolt pattern has been installed on the tunnel vault in order to fix the unstable wedges and ensure safety conditions for the workers during the following phases. Before proceeding with the realization of the final layer of the new lining, the new rails were built (lowered of 52 cm respect the original position) in order to use them for the transportation of the precast segments inside the tunnel. In addition, to facilitate the installation phase, the erector was placed on a freight car running on the track. In this case no waterproofing sheet has been adopted, as the waterproofing has been guaranteed by the use of gaskets placed on the segments profile (very similar to the waterproofing system adopted by TBMs). Each section of the tunnel is composed of 5 segments, this number (higher than the other examples reported) has allowed to contain the size and improve the handling of the prefabricated elements. Once the new coating was assembled, it was preceded with the injection of the concrete in the residual gap between the shotcrete and the segments in order to assess the backfilling. Finally, before the reopening of the tunnel, a new system of electrification of the railway network was installed. The same construction method was subsequently used by the Rhaetian Railway to renew the Sasslatsch tunnel (Susch - Lavin) and the Mistail tunnel (Solis - Tiefencastel).


*Figure 2.4-2 - Glatschera Tunnel: segments installation*

## 2.5 Enlarging section and re-profiling

The methods that allow the enlarging of the tunnel section are similar to those used for the re-lining. In fact, the re-lining methods described above can also be adapted and used for re-shaping and/or enlarging the tunnel section. However, in the last decade, a new method has been developed that allows the space created by enlargement to be used to position the construction equipment and perform the work activities. In this way it is possible to advance with the enlargement works maintaining the tunnel in operational service.

The solution to widen the existing tunnel without interrupting traffic was carried out for the first time in the early 2000's for the Nazzano Tunnel enlargement, Italy. The safety of users was achieved by installing a special steel shield designed to physically separate construction areas in the tunnel from those used for traffic flow.
The protection in question must satisfy the following requirements: resistance to shocks from excavated material or from collapses, dimensions must be compatible with the transit of vehicles inside it and with the dimensions of the tunnel to be enlarged, resistance to potential impacts from vehicles in transit.

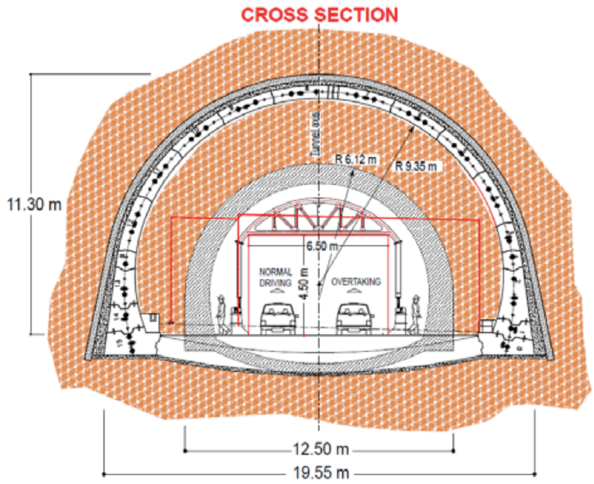


*Figure 2.5-1 – Nazzano Tunnel: tunnel section type for widening in the presence of traffic*

The technology chosen for the preliminary lining was that of mechanical precutting, which consists of making an incision in the face of appropriate thickness and length around the extrados of the tunnel to be excavated, which is immediately filled with fiber reinforced concrete. A shell with a truncated cone shape is thereby created and tunnel advance takes place beneath it. Operations for precutting and the erection of the active arch final lining are carried out using a single machine, named a Multifunction Machine (M.M.), consisting of a robust dual arch steel structure. The technology chosen for the construction of the final lining consists of erecting an arch of prefabricated concrete segments below the intrados of the precut shell. This arch is then immediately rendered self-supporting by the action of a flat jack fitted in the key segment. The erection of the prefabricated concrete segments was performed by making use of the same machine employed to carry out the mechanical pre-cutting, using the same supporting structure to handle, position and support the segments until the arch is put under pressure. This operation can also therefore be carried out by working above the traffic protection shield in a manner that is fully compatible with the unusual layout typical of the construction site used to widen a tunnel in the presence of traffic. Finally, the construction of the tunnel invert completes the tunnel widening operations.


*Figure 2.5-2 – Nazzano Tunnel: traffic protected by the steel protection shield while the old tunnel is being widened.*

The enlargement of a tunnel in the presence of traffic necessarily involves a layout of the construction site where the working spaces are very small that limit the size of the equipment that can be used, which must operate at considerable heights (10 m approx.). The excavation phase of the sandy material was carried out by means of two small wheel loaders that can reach a variable advancement speed that varied between 0.75 meters per day and 0.90 meters per day.
This method is very innovative and allows to minimize the impact of the construction site on traffic, however this type of construction site is not applicable in all cases of tunnel enlargement. In fact, the possibility to use the excavated space depends on the size of the enlargement, moreover, the material to be excavated must have mechanical properties that allow an excavation with systems that guarantee traffic safety (for example not drill and blast) and by means of small machinery.

# 3 Case history 1: Melide-Grancia renovation project

## 3.1 Tunnel history and description

The first project carried out by Pini Group company concerning the re-lining by means of prefabricated elements was the Melide-Grancia Tunnel renovation, in 2022. This infrastructure is located in Switzerland, Canton Ticino and allows the A2 motorway to cross the mountain San Salvatore. This route, so-called Gotthard route, is one of the most important north-south transport arteries in Europe and one of Switzerland's busiest motorways: thousands of car and lorry drivers use the route daily, which runs from Basilea, then through the Gottardo to Lugano and continues on the Italian side in the form of the A9 in the direction of Milan. Along the route passes through the Melide-Grange tunnel which is a twin-tube tunnel 1730 m long, built between 1965 and 1968. In the 10 years prior to the project, the tunnel underwent numerous adjustments to the new safety regulations. In the original structure no waterproofing system was planned and executed for the Melide-Grancia Tunnel. So far, any underground water had accumulated behind the concrete lining and there were therefore water leaks. In addition, there was also an intermediate ceiling in the upper section used for air recirculation.
The renovation works carried out in 2014 demolished the intermediate ceiling in order to restore the ventilation system and increase the useful section. In addition, works have been carried out on impermeabilization and sealing the joints in the surrounding rock and cracks in the concrete. The sealing intervention was realized through the use of hydrogel (hydro-expansive chemical compound) that was grouted in the rockmass to stop water leaks inside the tunnel. This type of intervention, however, did not achieve the desired effect, but instead made the situation worse. In 2017, only 3 years after the end of the works, the tunnel was affected by a collapse of the inner lining in North-South tube.

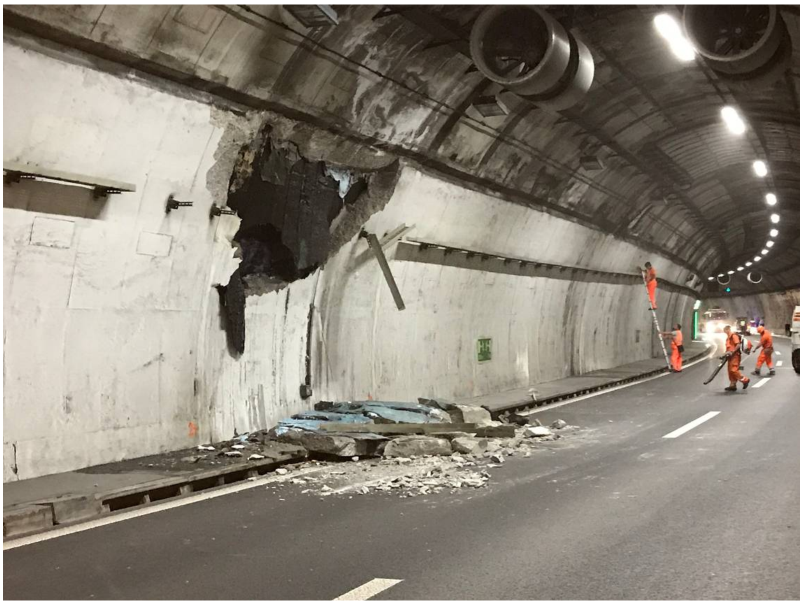


*Figure 3.1-1 - Melide-Grancia Tunnel collapse in 2017*

The reason for the collapse was attributed to poor execution of the works and design errors. The grouting operation, in fact, had caused the clogging of the groundwater drainage systems and, consequently, an accumulation of pressure on the tunnel lining had been produced. Moreover, the removal of the intermediate ceiling has led to a general weakening of the structure. Urgent drilling work was carried out between 2017 and 2018 to reduce hydrostatic pressure.

## 3.2 Renovation project by Pini Group

Between 2021 and 2023, global renovation works were carried out, with the design and construction management carried out by Pini Group. The objective of the renovation works is to ensure the structural safety of the tunnel for the period 2023-2040, i.e., up to the project "PoLume" which will provides for an enlargement of the tunnel section. The project has consisted of 2 main types of interventions: IT1 and IT2.

- IT1: re-lining of the tunnel in sections where structural safety is no longer guaranteed (about 60%).
- IT2: installation of a steel mesh on the tunnel vault to avoid the detachment of small concrete parts.

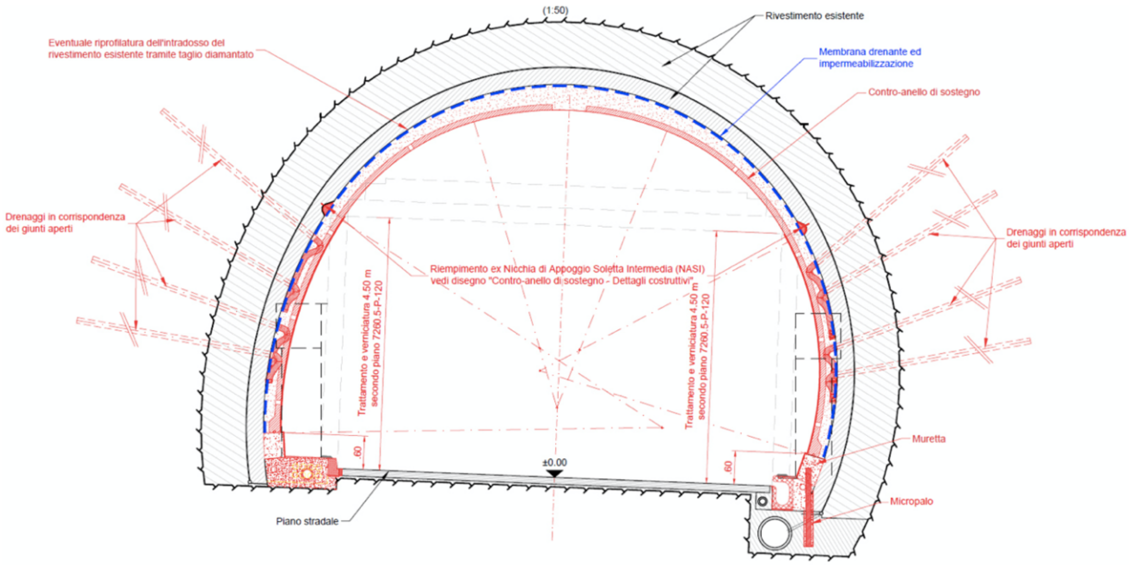



*Figure 3.2-1 - IT1 typical section with the new elements in red*

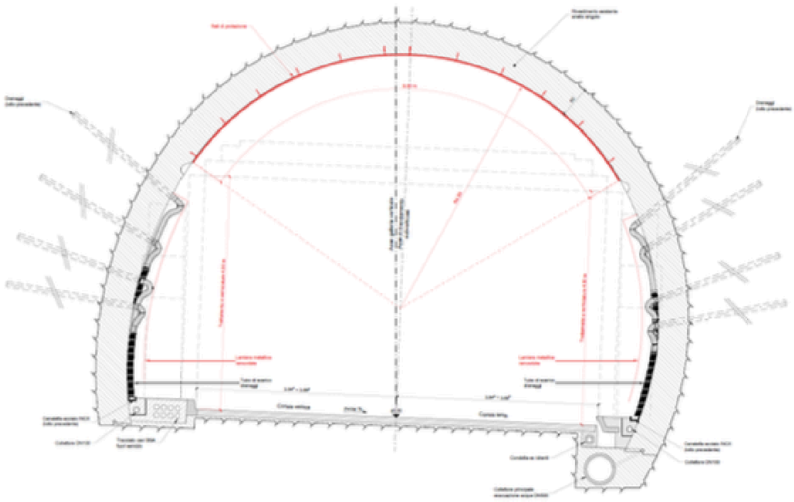


*Figure 3.2-2 - IT2 typical section with the new elements in red*

The prefabricated elements used for re-lining are "prédalles", in the images below are the drawings related to the executive design of these elements. As it can be seen, prédalles are characterized by an internal steel reinforcement connected to an external reinforcement positioned on the extrados. This solution was optimal for this case, as it allowed to minimize the thickness of the new coating. In fact, the absence of a preliminary milling phase (not performed due to uncertain structural conditions and due to the temporary nature of the project), made it necessary to intervene with elements with a low thickness.

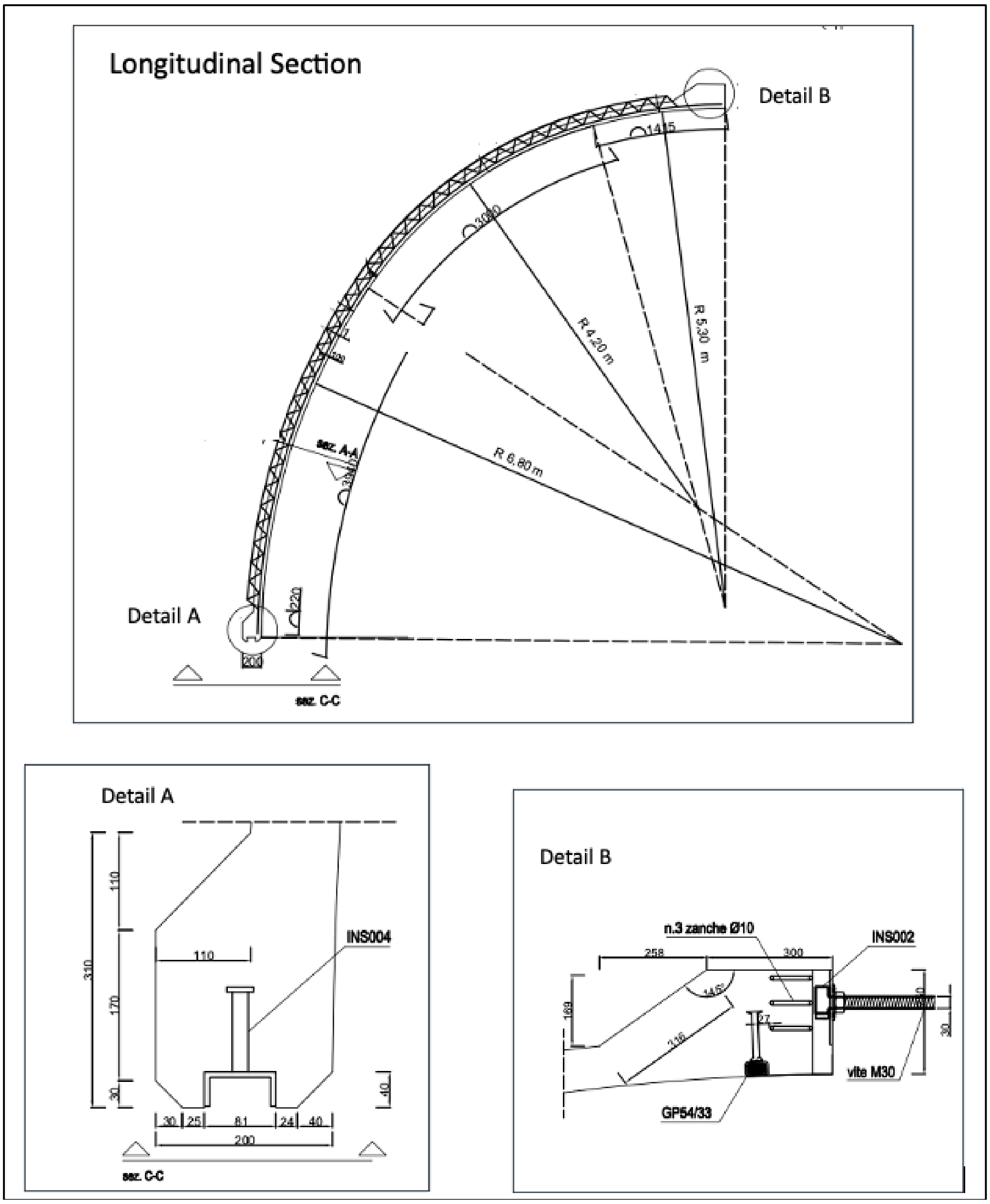



*Figure 3.2-3 – "Prédalles" executive project with details of installation inserts*

The prédalles are designed to constitute not only a temporary structural support, but also to allow the subsequent injection of concrete in the gap between the old and the new lining. The steel elements present on the extrados act as reinforcement to the injected concrete, so the final structural resistance will be given not only by the prédalles, but also by the reinforced concrete in the back. Although the building process is similar, this configuration is very different from the "backfilling" used by TBMs, as the concrete injection does not only create a pressure on the prefabricated slabs, but also contributes to the structural strength of the lining.

## 3.3 Building process

After a preliminary analysis carried out by coring to estimate the thickness of the existing coating, it was defined which parts of the tunnel to renew according to IT1 and

IT2. For those parts of the tunnel that have been re-lined, a detailed building process has been followed as described in the executive draft. The first part of the intervention consisted in the realization of a series of perforations to reduce the hydrostatic pressure acting on the concrete lining. These drillholes were then used to drain and channel the groundwater into the lateral manholes.  At the next stage the structural weakness left by the demolishing of the intermediate slab was repaired. To improve the load-bearing capacity of the foundation, an underpinning was carried out through the installation of micropiles at the side feet, moreover, side walls were built in order to create the support point for the prédalles. Subsequently, after the cleaning of the existing concrete surface, a new waterproofing system has been provided using 3 layers of sheets (draining geotextile, impermeabilizing geomembrane and an upper protection layer). On the sides of the vault new manholes have been created to improve water drainage.

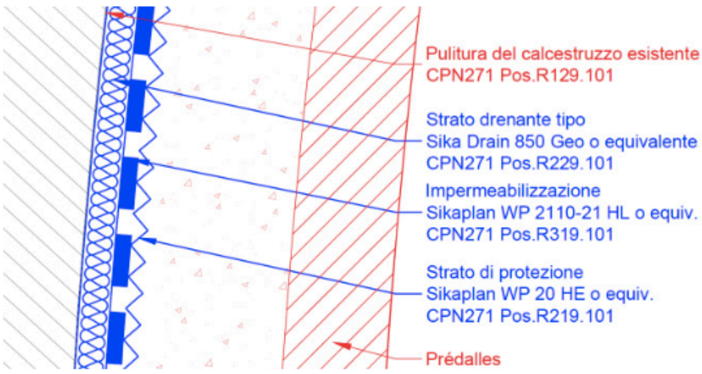



*Figure 3.3-1 - Detail of the composition of the drainage and waterproofing system of the vault*

Once the waterproofing was completed, the prédalles can be installed. During the executive project phase, the installation method was described, defining each handling step of the prefabricated elements. This allowed designers to evaluate the loads acting on the inserts at each step of the transient phase and consequently to evaluate strength and stiffness of prédalles. Given that the concrete thickness is very low (10cm), it is assumed that only the steel ribs contribute to the resistance of the slab for the transient load conditions.
After the erecting machine has put the slab in place, the operators have provided first the connection with the foot and then the connection in the top part of the vault. The connection at the foot of the prédalles was carried out through inserts connecting the precast elements with the sidewalls cast in place. The sidewall that supports the prédalles is a reinforced concrete beam 50cm high and 35cm wide. It is based on vertical micropiles and is stabilized by means of anchorages φ22 offset 50cm inclined 10° respect to the horizontal. The connection on the top of the vault consists of two inserts with a threaded rod placed symmetrically on the prédalles that were connected by means of a horizontal sleeve as shown as shown in the Figure 3.3-3. After the prédalles installation, it was provided the reinforcement on the keystone of the arch (Figure 3.3-4). This steel reinforcement was connected with the steel present on the prédalles extrados in order to create a continuity along the whole structure.

Figure 3.3-2 - Work in progress: prédalles installation

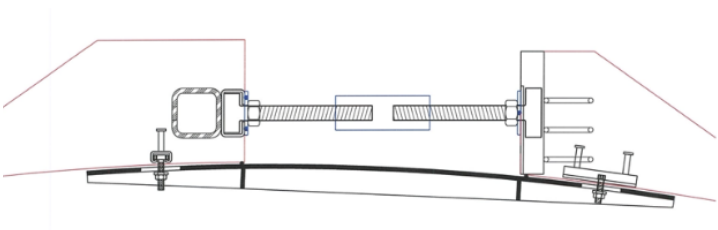


Figure 3.3-3 - Connection between prédalles at the top of the tunnel vault

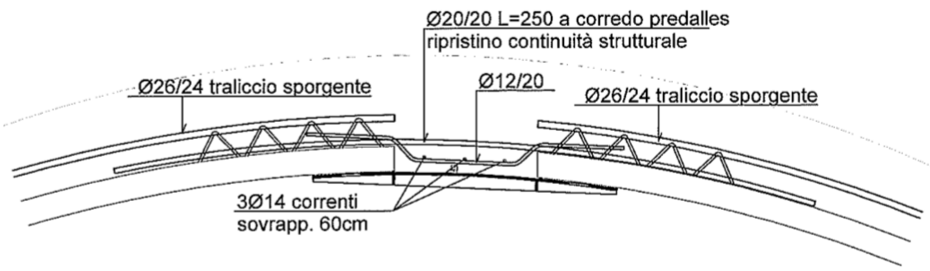



Figure 3.3-4 - Steel reinforcement on the top of the tunnel vault

*Figure 3.3-5 – Work in progress: 4th step concrete injection*

The re-lining then continued with the filling of the gap between the prédalles and the existing lining by means of concrete injection. Since prédalles are highly deformable, the casting will be carried out in 4 steps: 0-1.8 m, 1.8-3.4 m, 3.4-4.9 m, and 4.9-6.5 m (where the level 0m is placed at the base of the prédalles).

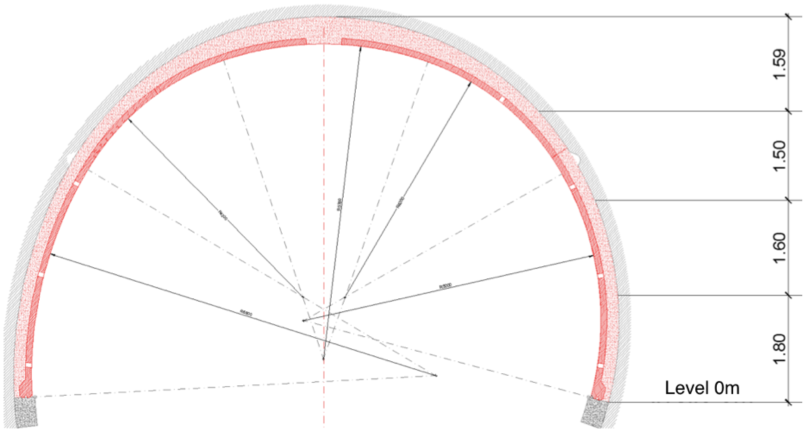



*Figure 3.3-6 - Injection of the concrete: 4 steps*

In order to limit the displacements of the prédalle induced by the pushing of the concrete during the injection, it is necessary to bind them to the outer concrete layer. For this reason, a constraint has been developed that block radial displacements. Each prédalle is bound by 6 constraints, these constraints have a longitudinal offset of 1.20m along the tunnel and are located at 0.60 m from the edge of the prédalles. Each of the 4 casting steps, representing a specific hydrostatic load, characterized a structural verification on prédalles in order to evaluate the maximum deformation and the maximum load to which the constraints are subjected. Regarding the structural verifications of the definitive phase, several cases were considered based on the geomechanical conditions received during the analysis. In particular, the 4 cases shown in the drawing below were considered.

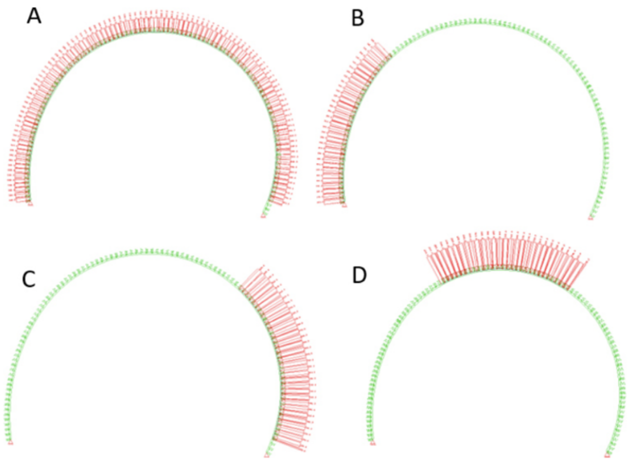



*Figure 3.3-7 - Load cases analyzed*

The IT1 ended with the renovation of the niches by realizing reinforced concrete structures cast in place. In correspondence to the niches, special prédalles with a different geometry were used.

# 4 Case history 2: project under development

## 4.1 Tunnel structure description

The tunnel object of the project was built in the first years of 60's, with traditional method excavation and it was opened to the traffic in 1965. It is a motorway tunnel with a length of more than 11 km, which consists of a current section hosting two road lanes (one for each direction) and has a width at street level of 7 meters. Every 300 meters of tunnel there are "garages": enlargement of the section (10.3 m at the level of the roadway), which allows to accommodate an additional road lane. Each garage develops for a length of 18 meters and can be used as emergency lane or for the storage of useful material for the infrastructure maintenance. Moreover, the tunnel is a key point for the motorway network and has great economic and social importance.

## 4.2 Refurbishment and chosen method justification

The refurbishment of the tunnel has been necessary as a result of an investigative survey that has identified diffused degraded conditions along the whole structure. Thanks to the geophysical survey carried out on the existing final lining concrete, it has been possible to identify two main zones with advanced deterioration levels. The essential cause of the deterioration conditions can therefore be identified in the ageing of the structure (the Tunnel was carried out in the 1960s), which inevitably leads to degradation of the characteristics and carbonation of the concrete, with the formation of alteration zones, cracks, voids, and local detachments. Another element of degradation is related to the presence of inflows of water which are in some cases rather abundant, leading to a leaching of the coating, the formation of intercalated limestone layers and, in some cases, a detachment of material. Moreover, following a

fire accident occurred in the late 90' which affected the structure, some parts of the original final lining were replaced. In particular, along some sections of tunnels, new steel bars were installed with a different composition respect to the steel of the original structure. This caused the generation of a potential difference in the structure which led to the corrosion and oxidation of the steel bars inside the concrete and, therefore, an overall decreasing of the mechanical resistance properties.
One of the main challenges for the realization of this work was to minimize the time of intervention due to the importance of the infrastructure and for the economic damage resulting from the closure of the tunnel to the traffic. This was one of the main reasons why traditional renovation methods were discarded, in favor of new solutions using prefabricated elements. Moreover, due to the presence of the garages, the use of a formwork with variable dimensions should have been carried out and this would cause a further increasing of the construction time.

## 4.3 Preliminary surveys

Through a geophysical analysis it has been possible to evaluate the thickness and the conditions of the existing concrete in a precise and punctual way. This survey was performed on the tunnel vault, along the parts object of the re-lining. The results were then used to assess the milling thickness and to forecast the resistance of the residual concrete.
According to the available geological surveys, the rock conditions are characterized in large part by solid granites with good mechanical properties. However, there is a tunnel portion that is characterized by fine-grained, compact metamorphic rock (mylonite), which affects a length of about 10 m. This material was detected at the time of the construction of the tunnel, and same consolidation measures were implemented. The geological survey also defined that the groundwater conditions inside the tunnel are characterized by moderate seepage and inflow, however, for areas dominated by mylonites, the inflows may become more significant (>80 l/s) and same mitigation measures must be taken into account. The granitic rock is characterized by a 2$^{nd}$ degree porosity, therefore, the water circulation in the rock mass is governed by fractures. The groundwater present in the tunnel surrounding comes from the infiltration of meteoric waters and from the melting of the overlying glaciers, therefore the amount of groundwater follows seasonal trends. It cannot be considered that the massif is saturated.

## 4.4 Intervention description

The object of the intervention consists in the lining refurbishment of two portions of the tunnel:
- A: 289 m, that includes one garage
- B: 248 m, that includes one garage

The first preliminary phase of intervention consists in milling the vault of the tunnel and ensuring stability before laying the final coating. An average milling thickness of about 36-46 cm on the vault, 56-37 on the side parts of the vault and 35-39 cm on the abutments is provided to ensure the internal shape of the tunnel. The difference values of milling along the section are due to the needed to slightly re-shaping the tunnel section. In fact, the existing section does not comply with the current standard on minimum dimensions of loading gauge, which is the diagram that defines the maximum height and width dimensions of the vehicles. In order to meet the new requirements concerning the minimum measures for the loading gauge, some specific parts of the section must be milled of a greater thickness. The stability of the work can be guaranteed by a sufficient thickness of the residual concrete or by a support when the residual concrete is insufficient to recover the induced forces. The class of existing (unreinforced) concrete is C20/25

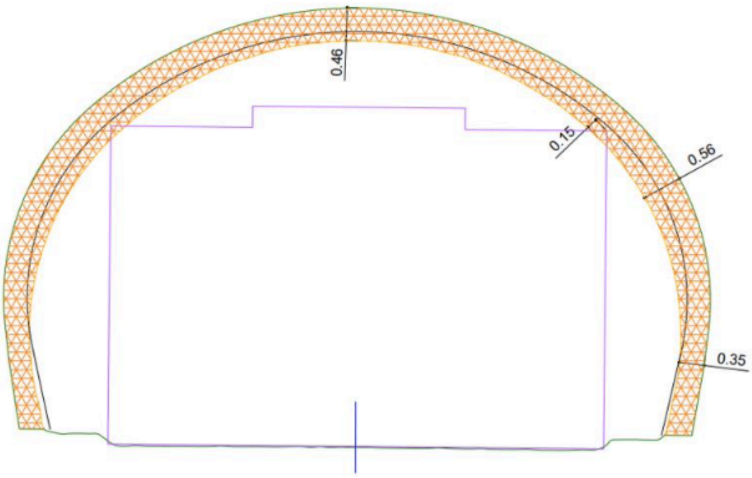



*Figure 4.4-1 - Thicknesses of the milling (in meters) with the representation of the gabarit (violet)*

In the second phase, once milling is done, shotcrete must be provided in order to level the surface and to provide a temporary support for the unstable zones of the rockmass. The shotcrete to be used is made of plastic fibers (polypropylene) and the thickness of the sprayed layer must be of 5 to 9 cm. Following the hardening of the shotcrete it will be possible to install the impermeabilization system. From extrados to intrados, the system consists of a geotextile layer (800 g/m2), a translucent geomembrane in PVC and an upper protection in black PVC to protect the steel reinforcement of the final lining concrete.


*Figure 4.4-2 - Detail of waterproofing and structural layers*

The cladding of the designed structure will be completely sealed by a DEG system to ensure the collection, drainage, and evacuation of water from the rock. The drains, located behind the sidewalls, have an outlet approximately every 10 m to convey the water in the main drain located in the lower part of the section, under the road level.

The third phase of the project concerns the realization of the new final lining. The mechanical operation of the new structure is independent from the existing structure, except for the support on the existing lower sidewalls. The section of the structure consists of 3 main elements that are installed in succession:

- Sidewalls cast in place and anchoring.
- Prefabricated segments.
- Key of the vault cast in place.

The concrete class of the final lining is C35/45 for all the structural elements, in the Table 4.4-1 the main mechanical properties are reported.

*Table 4.4-1 - Mechanical characteristics of final concrete*

| Élément structurel | Classe béton | $f_{ck}$ (MPa) | $f_{cm}$ (MPa) | $f_{ctm}$ (MPa) | $E_{cm,inst}$ (GPa) | $\varepsilon_{cu1}$ (‰) | $\varepsilon_{c2}$ (‰) | $\varepsilon_{cu2}$ (‰) | n | $\varepsilon_{c3}$ (‰) | $\varepsilon_{cu3}$ (‰) |
|---|---|---|---|---|---|---|---|---|---|---|---|
| Coques préfa.<br>Piédroits CSP | C35/45 | 35 | 43 | 3.2 | 34 | 3.5 | 2.0 | 3.5 | 2 | 1.75 | 3.5 |

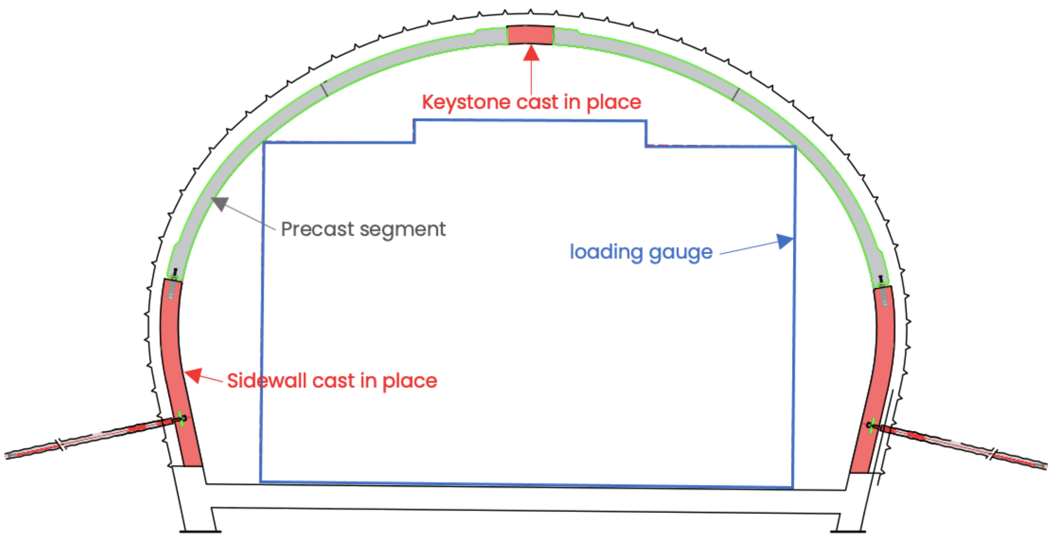



*Figure 4.4-3 - Section of the new final lining*

The sidewalls are made of reinforced concrete with a thickness of 24 cm and a height of 2.5 m. Before the casting of the concrete, a set of passive bolts has been provided to improve the connection between the structure and the rockmass. As it is possible to see in the image below (Figure 4.4-4), the bolts will be installed at a height of 40 cm from the sidewalk level with an inclination of 12° respect to the horizontal. For the running section the bolts will be installed each 2.5 m, while in the garages the offset will be 1.5 m. The selected bolts are GEWI-Plus-43 or equivalent type with a length of 5 m. Before the casting of the sidewalls, it is necessary also to treat the surface of the existing concrete in order to improve the mechanical properties at the interface. For this phase it is not planned to swallow steel bars, the connection will be improved only by roughening and humidifying the surface. Once the bolts will be installed and surface treated, it will be possible to proceed with the casting of the sidewalls by means of special formwork able to ensure the wanted shape.

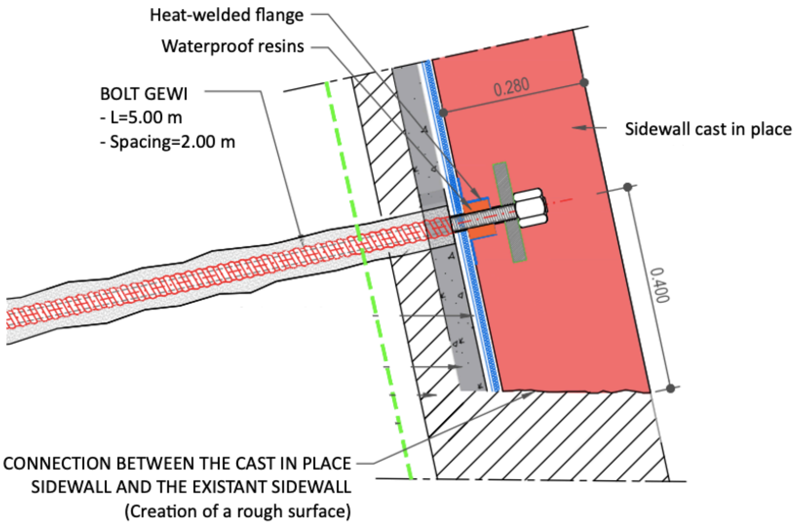



*Figure 4.4-4 - Detail design of the bolt (measures are in m)*

The prefabricated segments are made of reinforced concrete and will constitute the main supporting structure of the final lining. The installation of the segments will be carried out in a staggered manner between the left and right elements (as shown in figure). In this way, during the assembly phase, the structure will be able to stand on its own without additional temporary supports. For this reason, for the running section, two types of blocks have been designed:

- Segments of type A: will be used along the current section and consist of a width of 2.5 m
- Segments of type B: will be installed at the interface of the current section with the garage sections and are characterized by a width of 1.75 m

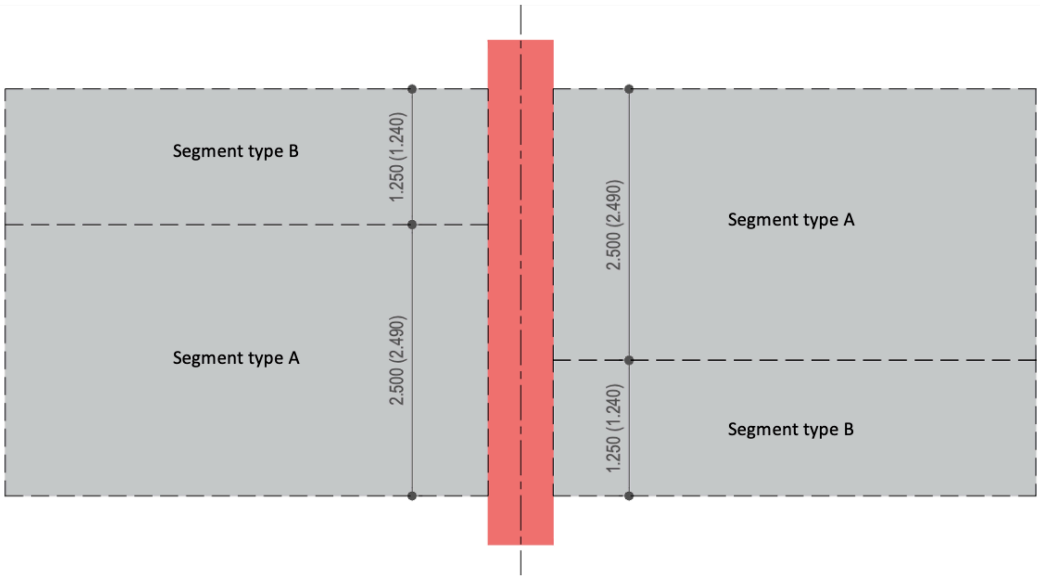



*Figure 4.4-5 – Plan view of the running section with A and B segment types*

Concerning the two garage sections subject to renovation, a third type of segment has been designed: type C. These will have a greater length respect the segments for the running section and a width of 1.5 m in order to improve handling. Also in this case, the installation of the segments will be carried out in a staggered manner. For parts of the garage vault that cannot be supported by segments due to geometrical incompatibilities, a support will be guaranteed through the realization of a layer of spritz beton.

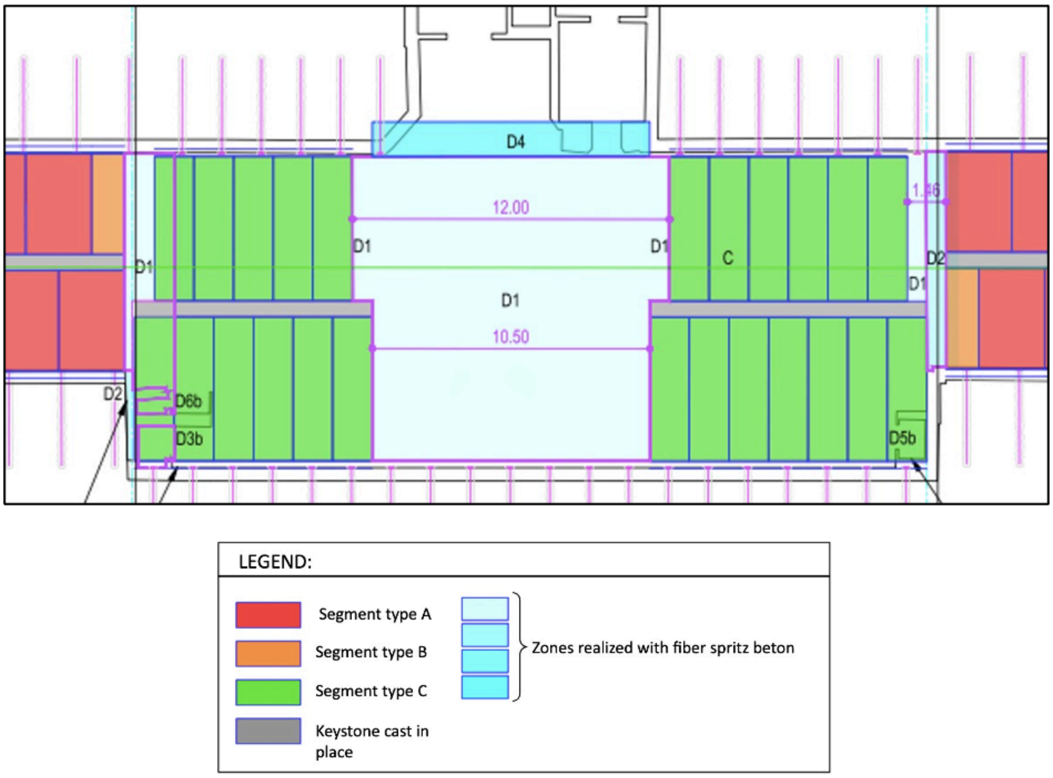



*Figure 4.4-6 – Plan view of the elements used in the garage vault*

The segments have a thickness of 18 cm that increases at the ends (i.e., in the parts of connection with the structural elements of the sidewalls and key). The reason for this shape is because segments parts close to the connections will be solicited by a higher bending moment.

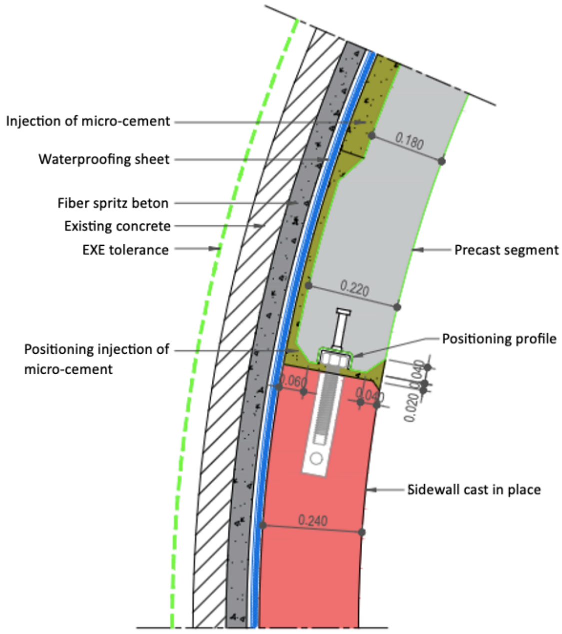



*Figure 4.4-7 - Detail of connection between segment and sidewall*

At the two ends of the segment there will be metal guides set in the concrete that will allow to connect the prefabricated elements with the other structural elements in a faster and easier way. Following the positioning of the segment, horizontal metal bars will be installed at the top of the vault to support the segments in a temporary phase. These bars will consist of two threaded elements installed on the segments in a specular way, these will be connected together with a metal sleeve as shown in the image below. For each segment there will be two supports so that, thanks also to the staggering, the previous segment will support the next one.

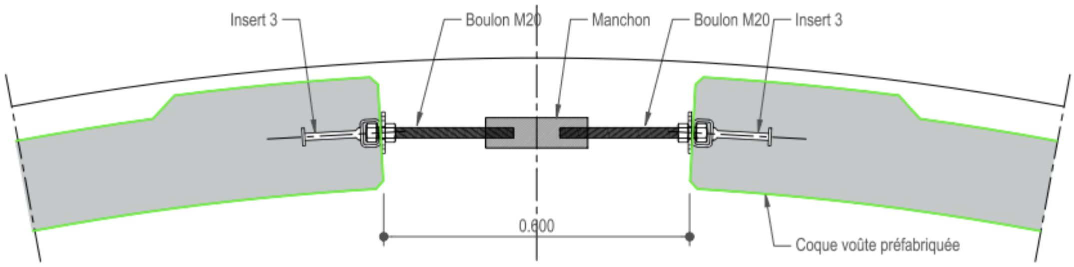



*Figure 4.4-8 – Keystone phase 1: connection between the two segments in the temporary phase*

After a series of segments will be assembled it will be possible to proceed with the phases of realization of the keystone. First of all, the steel reinforcements will be assembled at ground level and then placed on the vault.

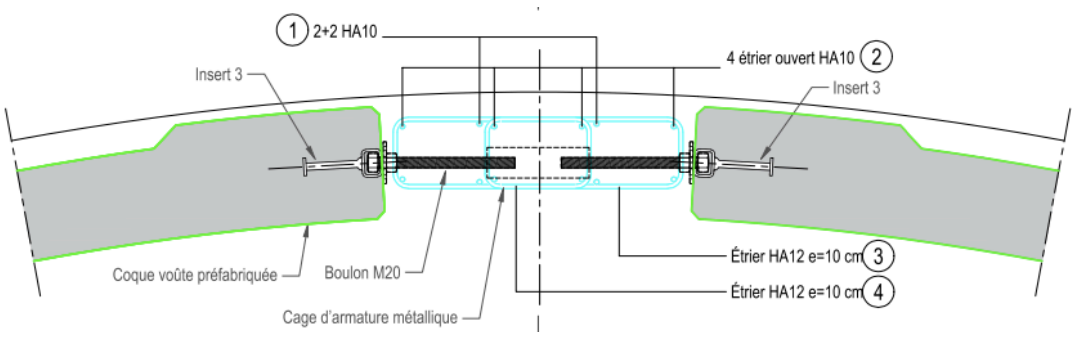



*Figure 4.4-9 - Keystone phase 2: steel reinforcement installation*

Then the formwork will be placed using special inserts designed in prefabricated segments. The concrete casting will be carried out by inserting a pipe that allows the escape of air and prevent the formation of air bubbles.

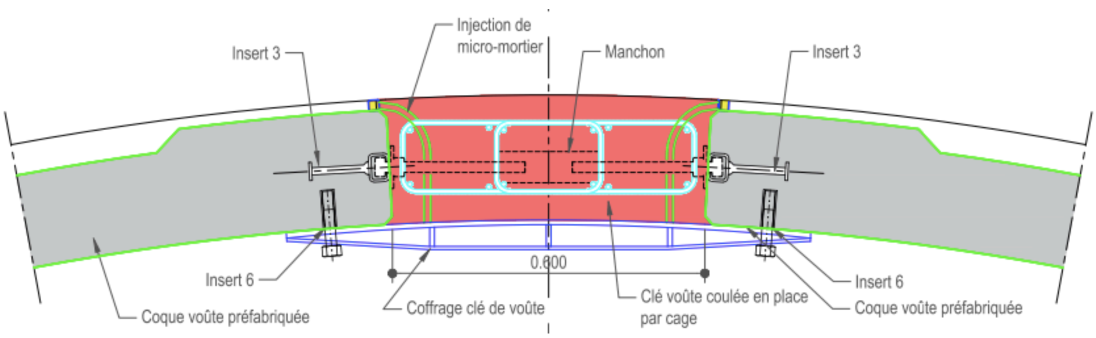



*Figure 4.4-10 - Keystone phase 3: formwork installation and keystone casting*

During the realization of the keystone, through the use of two tubes, two holes will be left inside the concrete. These will allow the injection of micro-cement in the gap between segments and the waterproofing sheet, also in this case there will be a pipe for to allow the air to escape. This injection will act as a backfilling, i.e., it will create a pressure on the arch structure in order to eliminate any tensional stress on the concrete.

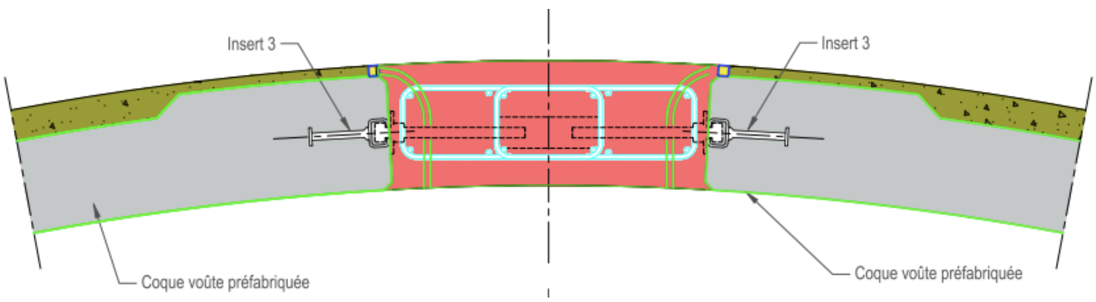



*Figure 4.4-11 - Keystone phase 4: micro-cement injection*

## 4.5 Segment features and production process

The prefabricated segments used for the final lining are exposed to several loads during the manufacturing process before finally being installed inside the tunnel. In the Figure 4.5-1 is reported a schematization of the production process of the segments in which are highlighted the most critical phases. For each phase, the relevant verifications to be carried out have been identified.

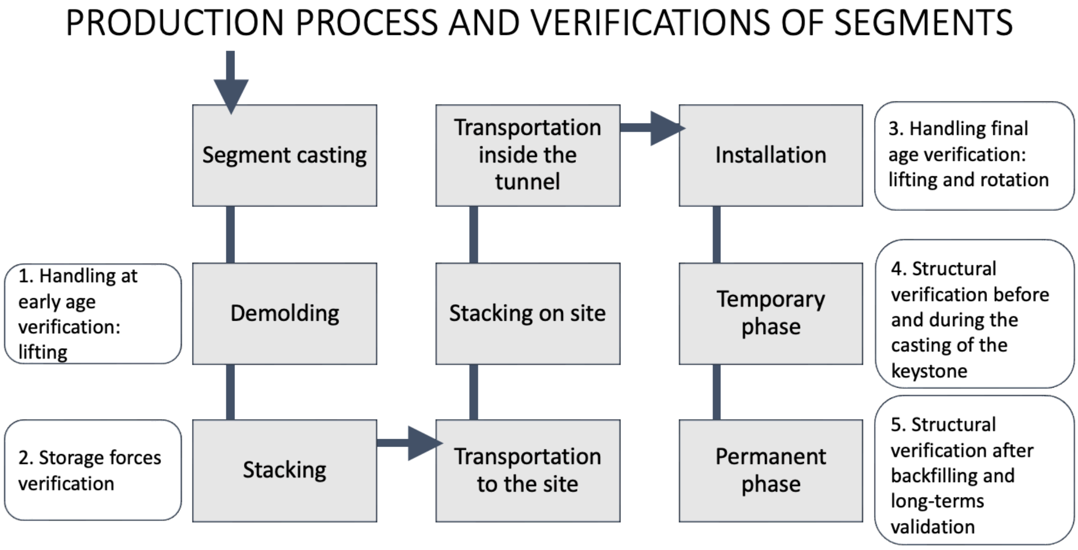



*Figure 4.5-1 – Production process and verifications of segments for the final lining*

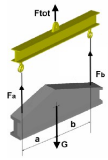
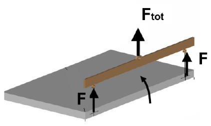
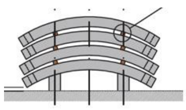
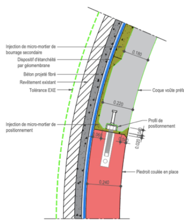
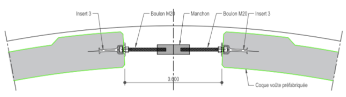
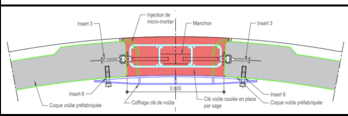
Specific inserts are used, embedded in the concrete of the segments, to allow lifting operations and connection during the installation. These inserts are used in specific phases of the productive process and segment handling, up to the installation of final lining. The structural verification of the segments, in some phases, can be carried out by analyzing loads acting on the inserts and their mechanical resistance. In the following table all the types of inserts are listed.

*Table 4.5-1 – Inserts on the final lining structure*

| Insert | Function | Position | Type |
|---|---|---|---|
| 1a | Segments lifting (extrados) | Segments type A, B and C | Foot anchor, type DSI ARTEON 5t L=120 mm (or similar) |
| 1b | Segments lifting (lateral) | Segments type A, B and C | Foot anchor eyelets HBS-MRd30-400 |
| 2 | Connection segment/sidewall | Segments type A, B and C | C profile 60x30, thickness 3 mm L=1000 mm (or similar) |
| 3 | Connection segment/key | Segments type A, B and C | Anchor rails, type GP 54/33, L=1000 mm Type A and B/ L=1200 mm type C |
| 4 | Connection sidewall/segment | Sidewalls | BT2069-CE drill bushings for M20 bolt |
| 5 | Injection of micro-cement | Segments type A, B and C | Drilling diameter 30 mm |
| 6 | Suspension of the formwork for the casting of the key | Segments type A, B and C | HBB bolt socket M20-140-TV + plastic positioner KU-10-M20-BR (or similar) |
| 7 | Segments lifting (intrados) and assembly | Segments type A, B and C | Lifting socket B30-140 (M30x150 mm, plate 100x100 mm) |
| 8 | Connection key/ segment | Key | Threaded rod M20 adjustable with sleeve and connecting head at the end (square nut) |

| | | | |
|---|---|---|---|
| 9 | Suspension of the formwork for the casting of the key | Segments type A, B and C | Bolt type M20 8.8 |
| 10 | Connection sidewall/segment | Sidewalls | Bolt type M20 8.8, L=70 mm with rounded head |
| 11 | Waterproofing during the filling of the extrados | Segments type A, B and C | Joint compriband |

*Table 4.5-2 – Phase of the segment production and final installation with the relative inserts adopted*

| PHASES | | Hypothesis | ID of the inserts adopted |
|---|---|---|---|
| Demolding and lifting |  | Demolding and lifting after about 15-18 hours of maturation.<br>Min. resistance fck = 20 MPa | 1a |
| Lifting (lateral) |  | Lifting in the factory and on site of each segment. | 1b |
| Turning |  | Turning from horizontal to vertical position. | 1a, 1b |
| Storage and stacking of segments, transport to the site |  | Intrados laid down. Maximum n.4 segments | To be defined |
| Transport inside the tunnel |  | Configuration on the truck | 1b |
| Lifting during the assembly |  | By means of 2-point grip tool, longitudinal | 7 |
| Connection between segment and sidewall |  | At the end of the segment there is a longitudinal metal guides set in the concrete | 2, 4, 10 |
| Temporary connection between segments on top of the vault |  | At the end of the segment there is a longitudinal metal guides set in the concrete | 3, 8 |
| Workform installation for of the keystone |  | | 6, 9 |

The final lining section considers the following inserts, arranged on the various elements (segments, sidewalls, keystone). Their layout (for segments type A and B) is illustrated in the following figures, and the verifications are shown in the following paragraphs.

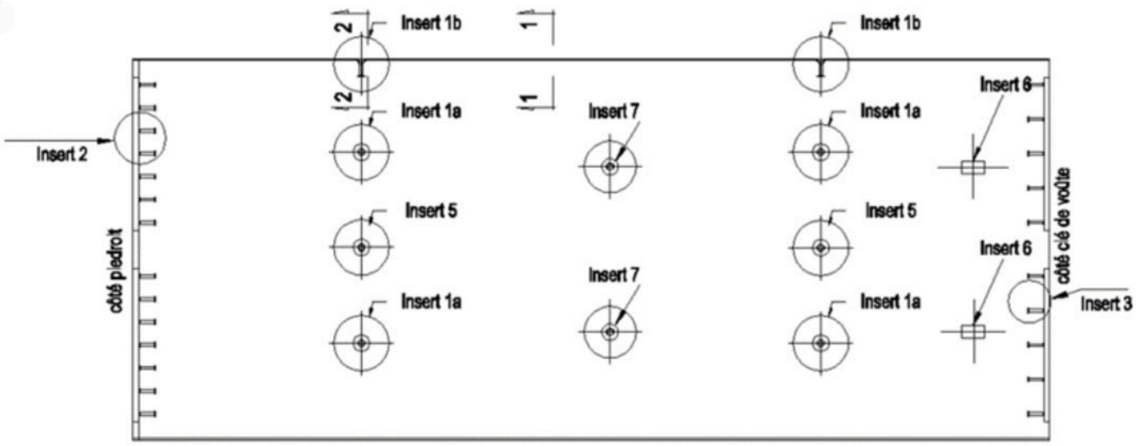



*Figure 4.5-2 – Main inserts (intrados and extrados) – Segment type A*

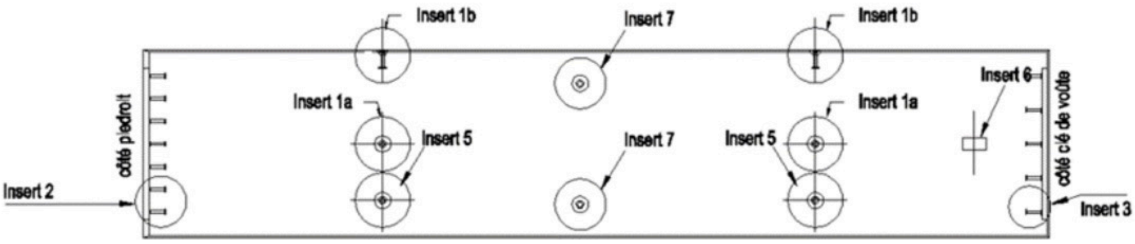



*Figure 4.5-3 - Main inserts (intrados and extrados) – Segment type B*

For the static validation of the insert used before the installation it has been adopted an analytical method, while for the validation of the final lining structure after the segment installation it has been used a combined (analytical and numerical) method.

Regarding the analytical method, the procedure consists of two steps: the individuation of shar and tensional loads acting on the analyzed insert and the evaluation of the resistance force. The verifications are carried out by using the technical report provided by the EOTA (European Organization for Technical Assessment): TR 047. This report identifies all types of failure mechanisms to which inserts may be subjected. In the following analyses, through the applications of the equations provided by the report, the resistance of all types of inserts present on the segments is verified.

*Table 4.5-3 - Insert verifications for tension load and possible failure mechanisms*

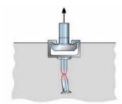
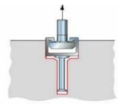
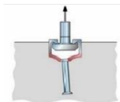
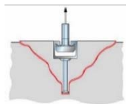
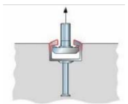
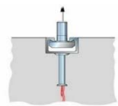
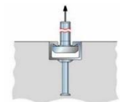
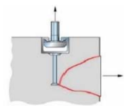
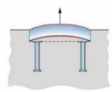
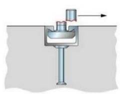
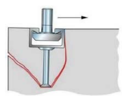
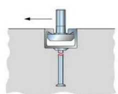
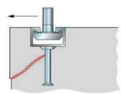
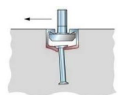
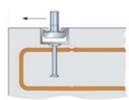
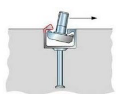
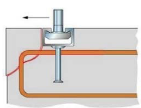
| VERIFICATIONS FOR ANCHOR CHANNELS UNDER TENSION LOAD | | | | | |
|---|---|---|---|---|---|
| | FAILURE MODE OF THE STEEL | | | FAILURE MODE OF THE CONCRETE OR SUPPLEMENTARY REINFORCEMENT | |
| **1** | Anchor |  | **6** | Pull-out failure |  |
| **2** | Connection between anchor and channel |  | **7** | Concrete cone failure |  |
| **3** | Local flexure of channel lip |  | **8** | Concrete splitting failure |  |
| **4** | Channel bolt |  | **9** | Concrete blow-out failure |  |
| **5** | Flexure of channel |  | **10** | Steel failure of supplementary reinforcement | |
| | | | **11** | Anchorage failure of supplementary reinforcement | |

*Table 4.5-4 - Insert verifications for shear load and possible failure mechanisms*

| VERIFICATIONS FOR ANCHOR CHANNELS UNDER SHEAR LOADS | | | | | |
|---|---|---|---|---|---|
| | FAILURE MODE OF THE STEEL | | | FAILURE MODE OF THE CONCRETE OR SUPPLEMENTARY REINFORCEMENT | |
| **1** | Channel bolt |  | **5** | Pry-out failure |  |
| **2** | Anchor |  | **6** | Concrete edge failure |  |
| **3** | Connection between anchor and channel |  | **7** | Steel failure of supplementary reinforcement |  |
| **4** | Local flexure of channel lip |  | **8** | Anchorage failure of supplementary reinforcement |  |

# 5 Design process and structure assessment

## 5.1 Design and calculation of the preliminary support

As already mentioned above, in case the residual coating is insufficient to ensure the stability of the work, it is necessary, before milling, to provide the installation of reinforcements and to design them according to the behavior of the rock mass to ensure stability. For the verification of the strength of the structure without reinforcements and for the design of the reinforcement systems, static analysis are carried out both in a continuous and discontinuous environment. The sections of tunnel characterized by a good quality rock (category 1: Class I/II and II, and category 2: Class II/III and III according to Bieniawski) are studied with a discontinuous approach and calculations are performed using UNWEDGE 4.0 software (developed by Rocscience).

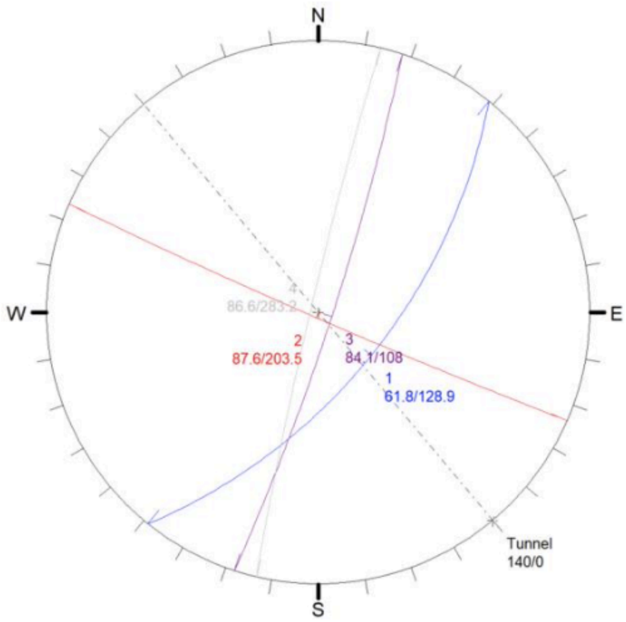



*Figure 5.1-1 - Stereographic Representation – Joints (1, 2, 3)*

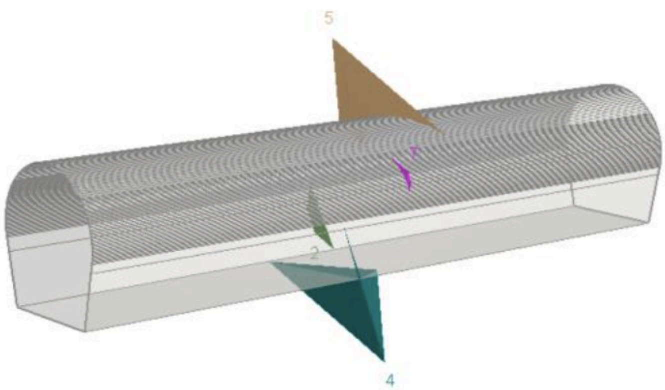



*Figure 5.1-2 - Perspective view of wedges developing under the combination (1, 2, 3)*

The procedure followed for checking the stability of the various wedges is shown below:

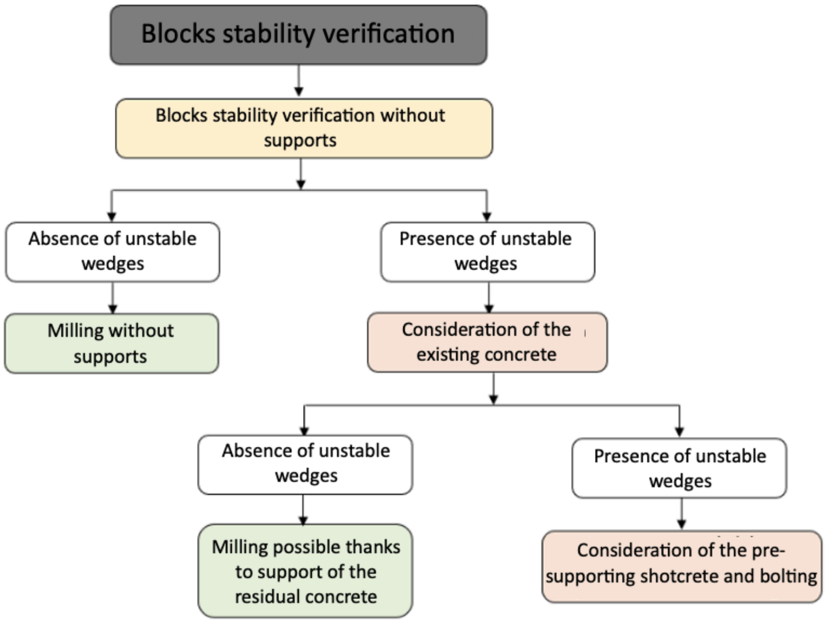



*Figure 5.1-3 - Approach to Verifying Wedges Stability*

At the end of the UNWEDGE modelling, it is verified that the safety factors on potentially unstable wedges are greater than or equal to the minimum safety factors based on block position:

- Tunnel vault wedges: FS ≥ 1.80
- Lateral wedges: FS ≥ 1.60

After analysis of the results with 28/12 VTR bolts with a length of 4 m, it was possible to evaluate the most suitable mesh. Even if it remains an unstabilized wedge with a safety factor of 1.64 on the vault, the most suitable solution is the one with a mesh of 2 m x 2 m with the possibility of creating a localized bolt reinforcement. It should be noted that, given the quality of the rock class I/II, wedges not stabilized by the mesh 2 m x 2 m have a low probability of occurrence as they are very sharp and very thin. It is also worth noting that during the construction of the tunnel many unstable wedges were removed.

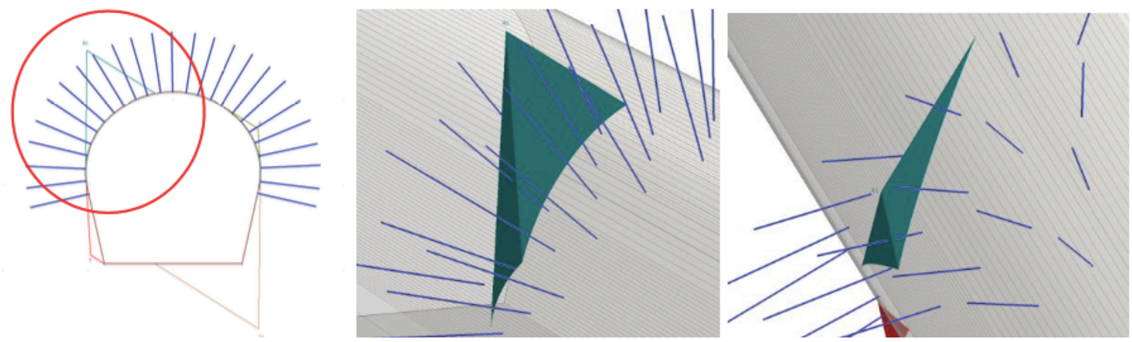


*Figure 5.1-4 – Views of the unstabilized wedge in case of 2mx2m mesh*

The sections of tunnel characterized by low-quality rock (category 3: Class IV according to Bieniawski), are studied by a continuous approach and the calculation are performed using PLAXIS 3D software (developed by Bentley System).

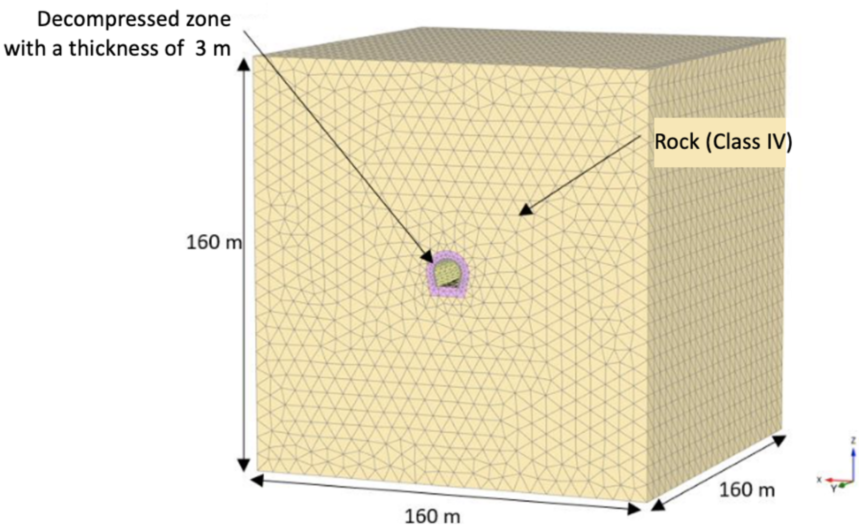



*Figure 5.1-5 - Finite element model made with PLAXIS 3D*

It should be noted that the sections tunnel applicable to rock of good quality and with a «Spalling and Rockburst» chipping behavior assimilated into a continuous medium, are not analyzed because the decompression inducing this phenomenon is already produced during the construction of the tunnel and the risk of having this type of behavior during the intervention of milling is unlikely.

In cases where the analysis showed the need to install supports, it has been proceeded with the verification of the stability going to add rockbolts to the realized models. During installation, the support elements will consist of VTRs since they must allow a low-cut resistance to facilitate the milling phase.

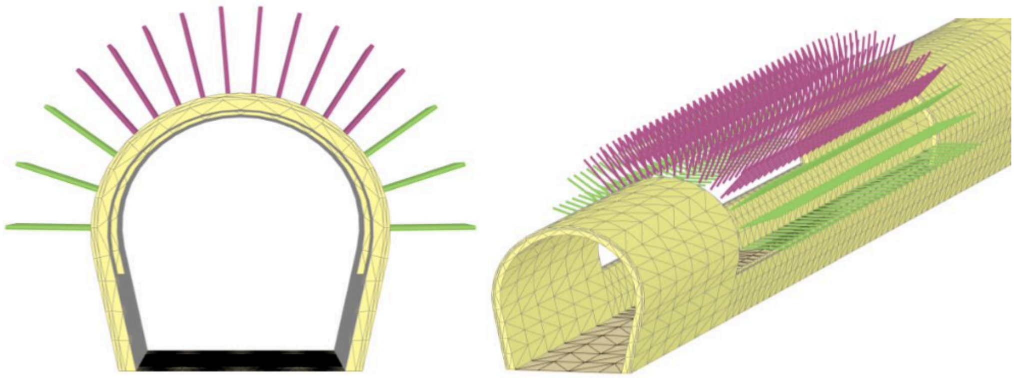


*Figure 5.1-6 - Modelling of the reinforced area with continuous approach: bolts 38/12 (vault, magenta) and 28/12 (abutments, green)*

The numerical analysis in a continuous environment made it possible to estimate the residual concrete thickness, which guarantees the stability of the work in the provisional phase in areas with poor rock quality. A minimum coating residual thickness of 25 cm was estimated with an fck=25mpa. In this case it is not necessary to provide support unless the concrete in the tunnel is found to be unsuitable. Where the residual coating thickness is less than 25 cm, a bolt shall be made before milling with bots 38/12 and a mesh of 1 m x 1,5 m on the vault and bolts 28/12 with a mesh of 1,5 m x 1.5 m on abutments and 4 m long sealed with resin or water-reactive resin. In the case of presence of water leaks, it is necessary to make a drainage to ensure the grip of the seal and drain the possible water pressures that can be encountered.

For the numerical analysis, the shotcrete was modelled for both discontinuous and continuous approaches. In the first case the UNWEDGE software allows to introduce in the model a layer with a proper cut resistance that represents the effect of the shotcrete on the unstable wedges. The shotcrete modelled with the continuous approach was carried out by bar elements attached on the milled profile. These are characterized by their axial rigidity EA and the bending EI taking into account the evolution of the Young modulus induced by the maturation (increase in strength) of the concrete.
For the concrete analytical verifications, the different breaking modes described in the AFTES GT20 recommendation and in the Barrett & McCreath article were analyzed. The positive effect of the fibers will not be considered in the calculation, whatever the fiber dosage chosen, and the verifications will be carried out considering a non-reinforced concrete.

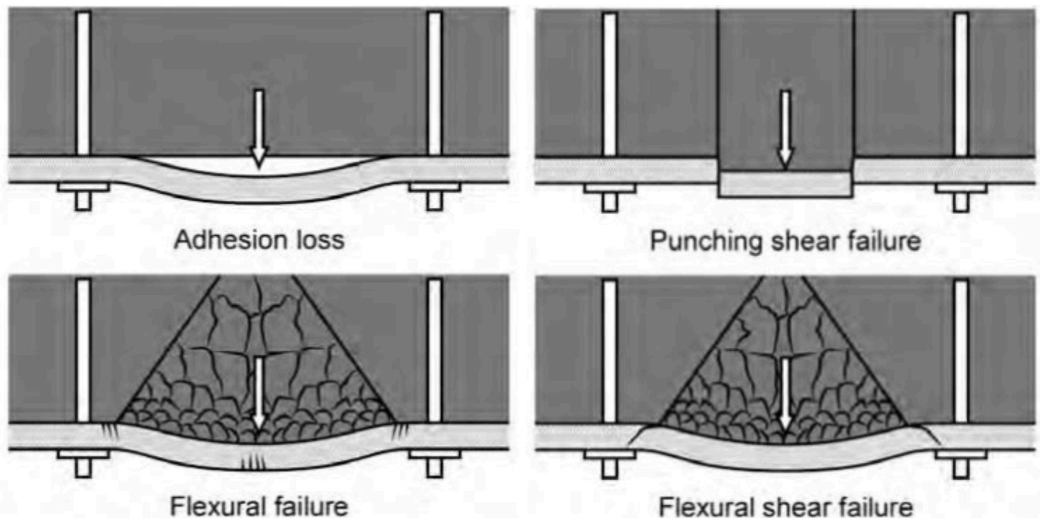


*Figure 5.1-7 - Concrete failure patterns after Barrett & McCreath (1995)*

## 5.2 Segments verifications: handling at early age (lifting)

The handling of the segments at early age consists in the demolding of the element after 15-18 hours of maturation. The lifting from the formwork is carried out by means of cable (with a maximum angle of 15° respect to the vertical, this value allows to approximate the horizontal component to zero; however, an angle amplitude factor must be considered) connected to the 4 inserts ID 1a located on the segment extrados. An additional release load of 14 kN is be considered in order to take into account the forces of adhesion between the segment and the formwork.
The analysis is performed on the Type A segment, which has the most critical conditions. The following parameters are considered:

*Table 5.2-1 - Considered parameters*

| | | |
|---|---|---|
| Segment weight | 68 | kN |
| Dynamic Lift Coefficient ELS (Service Limit State) | 1.15 | |
| Coefficient for justifications of anchors in reinforced concrete ELU (Limit State Ultimate) | 1.5 | |
| Additional release load | 14 | kN |
| Angle amplitude factor | 1.04 | |
| Number of lifting points | 4 | |

$$\text{Live Load } = (68 + 14)\ kN * 1.15 * 1.5 * 1.04 = 148\ kN$$

$$\text{Load at each lifting point } = \frac{148\ kN}{4} = 37\ kN$$

To satisfy the mentioned strength, an insert with a capacity of 5 tons (~49 kN) was selected.

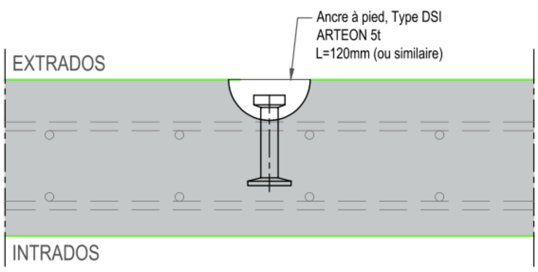



*Figure 5.2-1 - Detail of ID 1a inserts*

The element is certified by the manufacturer for the required load respecting the constraints of the intended uses and the specific lifting elements provided. However, an additional check is carried out on the mechanism of the insert, following (checks following EN1992-4: 2019). The expected resistance of the concrete after 15-18 hours of maturation is generally about 20 MPa, so this value is considered in the verification and it is noted that, with this value, there will be no cracking or breaking problems. In addition, and on the safety side, the verifications are carried out without considering the longitudinal and transverse steel reinforcement of the segment, so the safety factor is significantly higher.

- Insert section area:

$$A_s = \pi * r^2 = \pi * 10^2 = 314{,}16\ mm^2$$

- Ring area between hole and insert:

$$A_h = \frac{\pi}{4} * (d_h^2 - d_a^2) = \frac{\pi}{4} * (36^2 - 20^2) = 703{,}717\ \text{mm}^2$$

- Characteristic resistance for steel failure of the anchor:

$$N_{Rk,s,a} = A_s * f_{uk} = 314{,}16 * 480 * 0{,}001 = 150{,}80\ kN$$

- Characteristic resistance for pull-out failure of one anchor:

$$N_{Rk,p} = k_2 * A_h * \ f_{ck} = 10{,}5 * 703{,}717 * 20 * 0{,}001 = 140{,}74\ kN$$

- Basic characteristic resistance of one anchor in case of concrete cone failure (not influenced by adjacent anchors):

$$N_{Rk,c}^{0} = k_1 * \sqrt{f_{ck}} * h_{ef}^{1,5} = 11 * \sqrt{20} * 120^{1,5} * 0,001 = 64,67\ kN$$

- Characteristic resistance of one anchor in case of concrete cone failure (considering the effect of adjacent anchors):

$$N_{Rk,c} = N_{Rk,c}^{0} * \frac{A_{c,N}}{A_{c,N}^{0}} * \psi_{s,N} * \psi_{re,N} * \psi_{ec,N} = 64,67 * \frac{0.1296}{0.1296} * 1 * 1 * 1 = 64,67\ kN$$

- Basic characteristic resistance of one anchor channel in case of splitting failure:

$$N_{Rk}^{0} = \min\left(N_{Rk,p}, N_{Rk,c}\right)$$

- Characteristic resistance of one anchor channel in case of splitting failure:

$$N_{Rk,sp} = N_{Rk}^{0} * \frac{A_{c,N}}{A_{c,N}^{0}} * \psi_{s,N} * \psi_{ec,N} * \psi_{re,N} * \psi_{h,sp} = 64,67 * \frac{0.1296}{0.1296} * 1 * 1 * 1 * 1,1 = 71,14kN$$

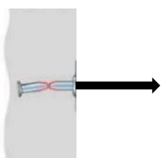
- Design value of the axial load acting on one anchor:

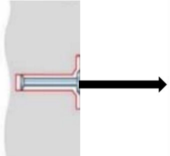
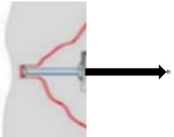
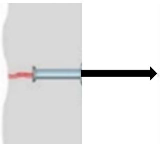
$$N_{Ed} = 37kN$$

*Table 5.2-2 – Definition of the used parameters for the tensile verifications*

| | |
|---|---|
| $k_1$ | Coefficient that considers the conditions of the concrete (for uncracked concrete $k_1 = 11$) |
| $h_{ef}$ | Length of the anchor inside the concrete |
| $f_{ck}$ | Nominal characteristic concrete compressive cylinder strength |
| $f_{uk}$ | Nominal characteristic steel ultimate tensile strength |
| $k_2$ | Coefficient that considers the conditions of the concrete (for uncracked concrete $k_2 = 10{,}5$) |
| $A_{c,N}/A_{c,N}^{0}$ | Factor accounting for the geometric effects of spacing and edge distance. |
| $\psi_{s,N}$ | Factor accounting for the influence of edges of the concrete member on the distribution of stresses in the concrete |
| $\psi_{re,N}$ | Factor accounting for the negative effect of closely spaced reinforcement in the concrete member on the strength of anchors with an embedment depth hef < 100 mm |
| $\psi_{ec,N}$ | Factor accounting for the group effect when different tension loads are imposed to the individual anchors of a group (e.g., eccentric loading) |
| $\psi_{h,sp}$ | Factor accounting for the influence of the actual member thickness on the characteristic splitting resistance |

*Table 5.2-3 - Tension load verifications insert ID 1a*

| Tension load verifications | | |
|---|---|---|
| Type of failure | Equations | Demand and Capacity |
| Anchor failure  | $N_{Rd,s} = \frac{N_{Rk,s}}{\gamma_{Ms}} = \frac{150,80}{1,4} = 107,71kN > 37kN$ | $\frac{37}{107,71} * 100 = 30,89\%$ |

| | | |
|---|---|---|
| Pull out failure  | $N_{Rd,p} = \frac{N_{Rk,p}}{\gamma_{Mp}} = \frac{140,74}{1,5} = 93,83kN > 37kN$ | $\frac{37}{93,83} * 100 = 39,43\%$ |
| Concrete cone failure  | $N_{Rd,c} = \frac{N_{Rk,c}}{\gamma_{Mc}} = \frac{64,67}{1,5} = 43,11kN > 37kN$ | $\frac{37}{43,11} * 100 = 85,83\%$ |
| Concrete splitting failure  | $N_{Rd,sp} = \frac{N_{Rk,sp}}{\gamma_{Msp}} = \frac{71,14}{1,5} = 47,43kN > 37kN$ | $\frac{37}{47,43} * 100 = 78\%$ |

The equations reported in the table consider the partial coefficients given in the "EOTA/TR 047, Section 4.3.2, Table 4.1" these values are in accordance with EN 1992-1-1.

## 5.3 Segments verifications: storage forces verification

The segments, after their production, are temporarily stored in the prefabrication plant before being transported to the construction site where a second storage and stacking takes place. Due to the thinned shape of the segments (proportionally they are much thinner than the segments used for final lining of mechanized tunnels), the stacking phase is particularly critical. Although there is not yet a dedicated part in the executive project, some preliminary provisions have been made.
Two cases are considered for the stacking of segments, considering two possible deviations when positioning the supports. Figure 5.3-1 shows the two considered cases where e is the distance from the vertical axis of the supports and is 0,1 m, and a is the distance of the supports from the segment edge and is 1,5 m.

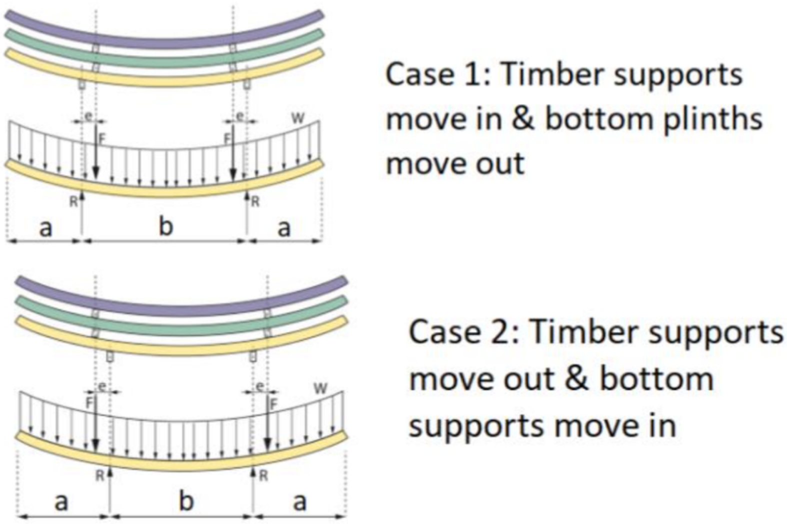



*Figure 5.3-1 - Segment stacking load cases*

For the analysis it was assumed a number of 4 segments, with a simplified assumption of a constant own weight equal to 12 kN per linear meter of planned length for each segment (evaluated by dividing the total weight segment of 68kN by 5.764m which is the segment length). The analysis allowed the calculation of the bending moment diagram, of the shear forces acting on the lower segment. Taking into account the structural controls carried out, the maximum number of segments is 4 for each stack.

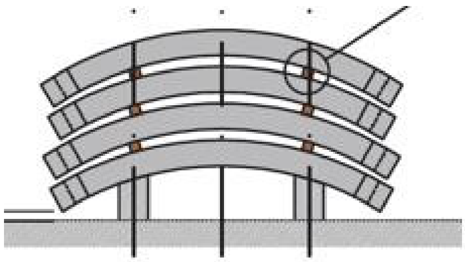


*Figure 5.3-2 – Segment stacking scheme*

## 5.4 Segments verifications: handling final age (lifting and rotation)

The handling at final age includes all the movement phases of the segment that occur when the concrete is completely hardened and has reached the maximum mechanical properties. These movements are carried out for loading on the truck, up to the installation phase inside the tunnel. For each type of movement there are specific inserts that allow to make the lifting or the rotation in different directions. In our case, the inserts used for this purpose are those named: 1a, 1b, 7. Depending on the type of movement, these inserts are stressed by tension or shear forces. This chapter provides the verifications of the main inserts used for this task.

The inserts named ID 1b are used for lifting and for turning the segments from the horizontal to the vertical position. It is planned to have N.2 elements, on the side of each segment. Due to the curved shape of the segment, special precautions must be taken when performing the rotation for the danger of breaking the corners at the base. The three forms of movement are considered:

a) Side flip with neoprene pads to support corners without damaging concrete.
b) Item raised using 4 lifting points (2 points 1a and 1 point 1b), with two independent lifting elements.
c) Lateral lifting of the segment.

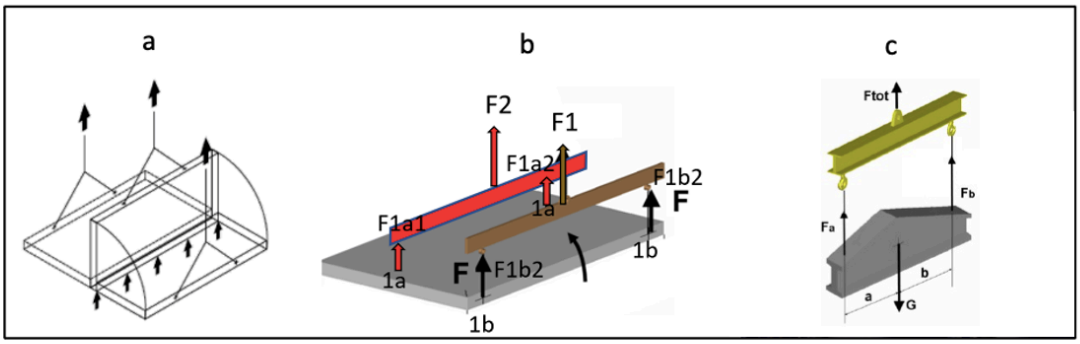



*Figure 5.4-1 - Types of rotation considered*

For the first rotation type (case a), the verification is carried out considering the support as a column on the edge with a dimension of the section equal to 30x30cm. The analysis is therefore carried out by considering the resistance of the concrete to the punching shear force. The calculations are performed on the Type A segment, which has the most critical condition.

$$\text{Live Load} = 68\,kN * 1.15 * 1.5 * 1.04 = 122\,kN$$

$$\text{Load at each lifting point} = \frac{122\,kN}{4} = 30.5\,kN$$

$$N_{Ed} = V_{Ed} = 30{,}5kN$$

*Table 5.4-1 - Rotation type a validation*

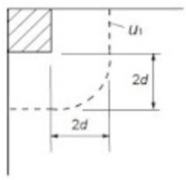
| Punching Shear check | | | | | | | | |
|---|---|---|---|---|---|---|---|---|
| № | *Size* | $N_{Ed}$ | $\beta$ | $\beta.V_{Ed}$ | $u_1$ | $v_{Ed}$ | $v_{Rd,c}$ | $V_{Rd,cs}$ |
| | - | - | *1** | - | - | 2 | 3* | - |
| - | mm | kN | - | kN | mm | MPa | MPa | MPa |
| Pad | 30x30 | **30.5** | **1.40** | 42.7 | 1028.0 | 0.39 | 0.59 | - |

$$1).\,\beta = \begin{cases} 1{,}15 - \text{middle column} \\ 1{,}40 - \text{end column} \\ 1{,}50 - \text{corner column} \end{cases}$$

$$2).\,v_{Ed} = \frac{\beta.V_{Ed}}{u_1.d_m}$$

$$3).\,v_{Rd,c} = 0{,}035.k^{3/2}.f_{ck}^{1/2}$$

if $v_{Ed} > v_{Rdc}$:




$v_{Rd,cs} = 0{,}75\,v_{Rd,c} + 1{,}5\,(d/s_r)\,A_{sw}\,f_{ywd,ef}\,(1/(u_1 d))\,\sin\alpha$

$*v_{Rdcs}$: *is the section punching resistance including the shear reinforcement.*

** The number of each formula corresponds to a column number in the table*

It is verified that the edge has no break in the initial time of turning. The acting forces will gradually reduce until the final time of turning, being this the more unfavorable condition.

For the second verification (case b), it is necessary to carry out the movements by controlling the symmetry of the loads, which must be guaranteed by the manufacturer. The rotation is carried out by using 4 lifting points (2 points 1a and 1 points 1b). The most critical condition occurs in the initial lifting, where the insert is shear loaded (90° respect to the axis of the inserts 1b). It is assumed that the angle with the concrete will not exceed 75° (ß = 15°), the angle amplitude factor used will therefore be at maximum 1.04. Also in this case, the analysis is performed on the Type A segments, which has the most critical conditions.

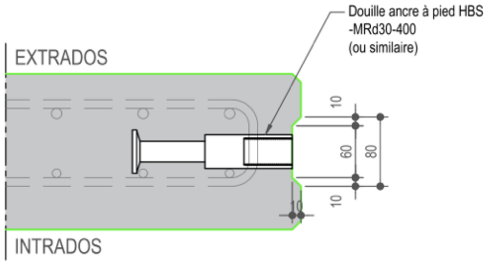


*Figure 5.4-2 – Detail insert ID 1b*

According to the previous results, the chosen element is certified for a load of 75 kN (7.5 tons) by the manufacturer. However, this (axial) force is only verified at the end of the rotation movement. For the first phases of the rotation, the most critical stress is verified by shearing, with the applied force orthogonal with respect to the insert. Therefore, an additional shear verification must be performed on the insert by considering the most critical conditions.

- Insert section area:

$$A_s = \pi * r^2 = \pi * 12^2 = 452{,}39\ mm^2$$

- Characteristic resistance for steel failure of the anchor:

$$V_{Rk,s} = A_s * f_{uk} * 0{,}8 = 452{,}39 * 480 * 0{,}8 * 0{,}001 = 173{,}72\ kN$$

- Basic characteristic resistance for concrete failure of an anchor loaded perpendicular to the edge (effects of spacing, further edges and member thickness are not considered):

$$\alpha = 0{,}1 * \left(\frac{l_f}{c1}\right)^{0{,}5} = 0{,}1 * \left(\frac{300}{90}\right)^{0{,}5} = 0{,}183$$

$$\beta = 0{,}1 * \left(\frac{d_{nom}}{c1}\right)^{0{,}2} = 0{,}1 * \left(\frac{24}{90}\right)^{0{,}2} = 0{,}077$$

$$V^0_{Rk,c} = k_9 * d^{\alpha}_{nom} * l^{\beta}_f * \sqrt{f_{ck}} * c_1^{1{,}5} = 2{,}4 * 24^{0{,}183} * 300^{0{,}077} * \sqrt{35} * 90^{1{,}5} * 0{,}001 = 33{,}65\ kN$$

- Characteristic resistance for concrete failure of an anchor loaded perpendicular to the edge:

$$V_{Rk,c} = V^0_{Rk,c} * \frac{A_{c,V}}{A^0_{c,V}} * \psi_{s,V} * \psi_{h,V} * \psi_{ec,V} * \psi_{a,V} * \psi_{re,V} = 33{,}6 * \frac{0{,}036}{0{,}036} * 1 * 1 * 1 * 1 * 1{,}4 = 47{,}11\ kN$$

- Characteristic resistance of an anchorage in case of pry-out failure:

$$V_{Rk,cp} = k_8 * \ N_{Rk,c} = 2 * 338{,}15 = 676{,}3\ kN$$

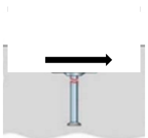
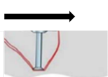
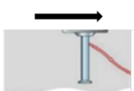
- Design value of the shear load acting on one anchor:

$$V_{Ed} = 30{,}5\ kN$$

*Table 5.4-2 – Definition of the used parameters for the shear verifications*

| | |
|---|---|
| $k_8$ | Factor to be taken from the relevant Approval |
| $k_9$ | Coefficient that considers the conditions of the concrete |
| $f_{uk}$ | Nominal characteristic steel ultimate tensile strength |
| $d_{nom}$ | Nominal diameter of the anchor |
| $l_f$ | Nominal length of the anchor |
| $c_1$ | Distance of the nearest edge |
| $A_{c,V}/A^0_{c,V}$ | Factor to take into account the geometric effects of spacing, member thickness and further edges |
| $\psi_{s,V}$ | Factor accounting for the influence of edges of the concrete member on the distribution of stresses in the concrete |
| $\psi_{h,V}$ | Correction factor to take into account that the resistance does not decrease linearly with the member thickness |
| $\psi_{ec,V}$ | Factor to take into account a group effect when different shear loads are acting on the individual anchors of a group (e.g., eccentric shear loading) |
| $\psi_{a,V}$ | Factor to take into account the angle between the shear load applied and the direction perpendicular to the free edge of the concrete member under consideration |
| $\psi_{re,V}$ | Factor to take into account the type of edge reinforcement |

*Table 5.4-3 – Shear verifications insert ID 1b*

| | Shear verifications | |
|---|---|---|
| Type of failure | Equations | Demand and Capacity |
| Anchor failure  | $V_{Rd,s} = \frac{V_{Rk,s}}{\gamma_{Ms}} = \frac{173,72}{1,4} = 124,09kN > 30,5kN$ | $\frac{30,5}{124,09} * 100 = 41,34\%$ |
| Pry-out failure  | $V_{Rd,cp} = \frac{V_{Rk,cp}}{\gamma_{Mc}} = \frac{676,3}{1,5} = 450,87kN > 30,5kN$ | $\frac{30,5}{450,87} * 100 = 6,76\%$ |
| Concrete edge failure  | $V_{Rd,c} = \frac{V_{Rk,c}}{\gamma_{Mc}} = \frac{47,11}{1,5} = 31,41kN > 30,5kN$ | $\frac{30,5}{31,41} * 100 = 97,1\%$ |

The equations reported in the table consider the partial coefficients given in the "EOTA/TR 047, Section 4.3.2, Table 4.1" these values are in accordance with EN 1992-1-1.

The equations reported in the Table 5.4-3 consider the partial coefficients according to the NTC 2018. It is noted that the verification on the concrete edge failure has a lower margin than the others. However, it is important to note that more critical conditions have been considered, in addition, the presence of steel reinforcement of the segments is not considered.

The third condition considered (case c) is simple lifting for loading the segments on the truck, in this case only the 2 inserts ID 1b are used and the acting load is coaxial respect to the inserts. The analysis is performed on the Type A segments that represent the most critical conditions.

$$\text{Load at each lifting point } = \frac{122\ kN}{2} = 61\ kN$$

$$N_{Ed} = 61\ kN$$

- Insert section area:

$$A_s = \pi * r^2 = \pi * 12^2 = 452{,}39\ mm^2$$

- Ring area between hole and insert:

$$A_h = \frac{\pi}{4} * (d_h^2 - d_a^2) = \frac{\pi}{4} * (39^2 - 24^2) = 742{,}201\ mm^2$$

- Characteristic resistance for steel failure of the anchor:

$$N_{Rk,s,a} = A_s * f_{uk} = 452{,}39 * 480 * 0{,}001 = 217{,}15\ kN$$

- Characteristic resistance for pull-out failure of one anchor:

$$N_{Rk,p} = k_2 * A_h * \ f_{ck} = 10 * 742{,}201 * 35 * 0{,}001 = 259{,}77\ kN$$

- Basic characteristic resistance of one anchor in case of concrete cone failure (not influenced by adjacent anchors):

$$N_{Rk,c}^0 = k_1 * \sqrt{f_{ck}} * h_{ef}^{1,5} = 11 * \sqrt{35} * 300^{1,5} * 0{,}001 = 338{,}15 kN$$

- Characteristic resistance of one anchor in case of concrete cone failure (considering the effect of adjacent anchors):

$$N_{Rk,c} = N_{Rk,c}^0 * \frac{A_{c,N}}{A_{c,N}^0} * \psi_{s,N} * \psi_{re,N} * \psi_{ec,N} = 338{,}15 * \frac{0{,}1858}{0{,}1858} * 1 * 1 * 1 = 338{,}15\ kN$$

- Basic characteristic resistance of one anchor channel in case of splitting failure:

$$N_{Rk}^0 = \min\,(N_{Rk,p}, N_{Rk,c})$$

- Characteristic resistance of one anchor channel in case of splitting failure:

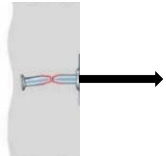
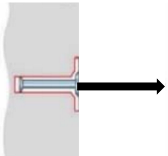
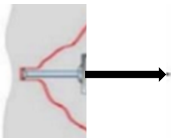
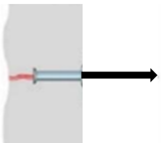
$$N_{Rk,sp} = N_{Rk}^0 * \frac{A_{c,N}}{A_{c,N}^0} * \psi_{s,N} * \psi_{ec,N} * \psi_{re,N} * \psi_{h,sp} = 259{,}77 * \frac{0{,}3715}{0{,}3715} * 1 * 1 * 1 * 1{,}1 = 285{,}75\ kN$$

- Design value of the axial load acting on one anchor:

$$N_{Ed} = 61\ kN$$

For the definition of the used parameters see Table 5.2-2.

*Table 5.4-4 – Tension load verifications insert ID 1b*

| Tension load verification | | |
|---|---|---|
| Type of failure | Equations | Demand and Capacity |
| Anchor failure  | $N_{Rd,s} = \frac{N_{Rk,s}}{\gamma_{Ms}} = \frac{217,15}{1,4} = 155,11kN > 61kN$ | $\frac{61}{155,11} * 100 = 39,33\%$ |
| Pull out failure  | $N_{Rd,p} = \frac{N_{Rk,p}}{\gamma_{Mp}} = \frac{259,77}{1,5} = 173,18kN > 61kN$ | $\frac{61}{173,18} * 100 = 35,23\%$ |
| Concrete cone failure  | $N_{Rd,c} = \frac{N_{Rk,c}}{\gamma_{Mc}} = \frac{338,15}{1,5} = 225,43kN > 61kN$ | $\frac{61}{225,43} * 100 = 27,06\%$ |
| Concrete splitting failure  | $N_{Rd,sp} = \frac{N_{Rk,sp}}{\gamma_{Msp}} = \frac{285,75}{1,5} = 190,5 > 61kN$ | $\frac{61}{190,5} * 100 = 32,02\%$ |

The equations reported in the table consider the partial coefficients given in the "EOTA/TR 047, Section 4.3.2, Table 4.1" these values are in accordance with EN 1992-1-1.

The insert is therefore checked for all acting loads.

ID 7 inserts are used in temporary phase, for the lifting of the segments to perform the installation inside the tunnel. The erector machine hooks up the segment by means of N.2 insert ID 7 located on the intrados. Verifications are carried out on the type A segment which presents the most critical conditions. The manufacturer of the inserts provided the values of Px, Py and Pz (representing the forces acting on the inserts in the 3 directions) for each step of lifting and installation. By combining the 3 values, it was possible to estimate the maximum values of the normal forces and shear forces acting on the inserts throughout the whole installation phase. At these values were multiplied the dynamic lifting coefficient (1.15) and the SLS weighting (serviceability limit state) for the verification of the anchorages (1.5). The final values of $N_{Ed}$ and $V_{Ed}$ used for the verifications are reported below.

- Insert section area:

$$A_s = \pi * r^2 = \pi * 15^2 = 706,86\ mm^2$$

- Ring area between hole and insert:

$$A_h = \frac{\pi}{4} * (d_h^2 - d_a^2) = \frac{\pi}{4} * (45^2 - 30^2) = 883,57\mathrm{mm}^2$$

- Characteristic resistance for steel failure of the anchor:

$$N_{Rk,s,a} = A_s * f_{uk} = 706,86 * 480 * 0,001 = 339,29\ kN$$

- Characteristic resistance for pull-out failure of one anchor:

$$N_{Rk,p} = k_2 * A_h * f_{ck} = 10 * 883,57 * 35 * 0,001 = 309,25\ kN$$

- Basic characteristic resistance of one anchor in case of concrete cone failure (not influenced by adjacent anchors):

$$N^0_{Rk,c} = k_1 * \sqrt{f_{ck}} * h_{ef}^{1,5} = 12,7 * \sqrt{35} * 150^{1,5} * 0,001 = 138,03\ kN$$

- Characteristic resistance of one anchor in case of concrete cone failure (considering the effect of adjacent anchors):

$$N_{Rk,c} = N^0_{Rk,c} * \frac{A_{c,N}}{A^0_{c,N}} * \psi_{s,N} * \psi_{re,N} * \psi_{ec,N} = 138,03 * \frac{0,2025}{0,2025} * 1 * 1 * 1 = 138,03\ kN$$

- Basic characteristic resistance of one anchor channel in case of splitting failure:

$$N^0_{Rk} = \min\left(N_{Rk,p}, N_{Rk,c}\right)$$

- Characteristic resistance of one anchor channel in case of splitting failure:

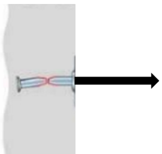
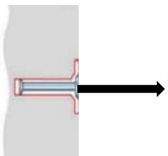
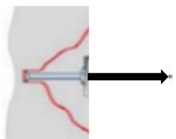
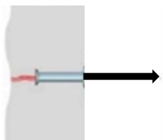
$$N_{Rk,sp} = N^0_{Rk} * \frac{A_{c,N}}{A^0_{c,N}} * \psi_{s,N} * \psi_{ec,N} * \psi_{re,N} * \psi_{h,sp} = 138,03 * \frac{0,2025}{0,2025} * 1 * 1 * 1 * 1,1 = 151,83\ kN$$

- Design value of the axial load acting on one anchor:

$$N_{Ed} = 72,15\ kN$$

For the definition of the used parameters see Table 5.2-2.

*Table 5.4-5 – Tension load verifications insert ID 7*

| Tension load verification | | |
|---|---|---|
| Type of failure | Equations | Demand and Capacity |
| Anchor failure  | $N_{Rd,s} = \frac{N_{Rk,s}}{\gamma_{Ms}} = \frac{339,29}{1,4} = 242,35kN > 72,15kN$ | $\frac{72,15}{242,35} * 100 = 29,77\%$ |
| Pull out failure  | $N_{Rd,p} = \frac{N_{Rk,p}}{\gamma_{Mp}} = \frac{309,25}{1,5} = 206,17kN > 72,15kN$ | $\frac{72,15}{206,17} * 100 = 35\%$ |
| Concrete cone failure  | $N_{Rd,c} = \frac{N_{Rk,c}}{\gamma_{Mc}} = \frac{138,03}{1,5} = 92,02kN > 72,15kN$ | $\frac{72,15}{92,02} * 100 = 80,15\%$ |
| Concrete splitting failure  | $N_{Rd,sp} = \frac{N_{Rk,sp}}{\gamma_{Msp}} = \frac{151,83}{1,5} = 101,22kN > 72,12kN$ | $\frac{72,15}{101,22} * 100 = 71,28\%$ |

The equations reported in the table consider the partial coefficients given in the "EOTA/TR 047, Section 4.3.2, Table 4.1" these values are in accordance with EN 1992-1-1.

- Insert section area:

$$A_s = \pi * r^2 = \pi * 15^2 = 706{,}86\ mm^2$$

- Characteristic resistance for steel failure of the anchor:

$$V_{Rk,s} = A_s * f_{uk} * 0{,}8 = 706{,}86 * 480 * 0{,}8 * 0{,}001 = 271{,}43\ kN$$

- Characteristic resistance of an anchorage in case of pry-out failure:

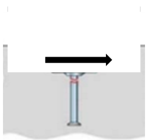
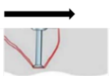
$$V_{Rk,cp} = k_8 *\ N_{Rk,c} = 2 * 138{,}02 = 276{,}06\ kN$$

- Design value of the shear load acting on one anchor:

$$V_{Ed} = 62{,}06\ kN$$

For the definition of the used parameters see Table 5.4-2.

*Table 5.4-6 – Shear verifications insert ID 7*

| | Shear verifications | |
|---|---|---|
| Type of failure | Equations | Demand and Capacity |
| Anchor failure  | $V_{Rd,s} = \frac{V_{Rk,s}}{\gamma_{Ms}} = \frac{271{,}43}{1{,}4} = 193{,}88kN > 58{,}74kN$ | $\frac{62{,}06}{193{,}88} * 100 = 32{,}01\%$ |
| Pry-out failure  | $V_{Rd,cp} = \frac{V_{Rk,cp}}{\gamma_{Mc}} = \frac{276{,}06}{1{,}5} = 184{,}04kN$ $> 58{,}74kN$ | $\frac{62{,}06}{184{,}04} * 100 = 33{,}72\%$ |

The equations reported in the table consider the partial coefficients given in the "EOTA/TR 047, Section 4.3.2, Table 4.1" these values are in accordance with EN 1992-1-1.

## 5.5 Temporary phase: analytical analysis structural verification before and during the casting of the keystone

The main aim of the temporary phase analysis it to verify the load-bearing capacity of the structure during installation. In this phase it is necessary to adopt numerical models capable of making complex calculations to evaluate the loads acting on the used inserts. As outlined above, the installation phase involves placing one segment at a time in a staggered way so that the previous segment supports the next and so on. The most unfavorable temporary condition for verification of the connection of the keystone was examined considering:

- two segments on each side (2.5 m each)
- two bolts for each segment
- segment staggered right/left of half segment (1.25 m)

The behavior of precast concrete segments was studied using a 3D model created in a SAP2000 finite element software. The model was created as close as possible to the final design of the project, in order to have a real understanding of the design aspects and internal actions acting on the shells. For this purpose, a shell element was created in SAP2000. The model created simulates two shells of 2.5 meters each and 18 cm thick, and the geometry was imported directly from the project drawings. In addition, the connection between the two opposite segments on top of the vault is modeled with M20 bolts, placed every 0.625 meters.

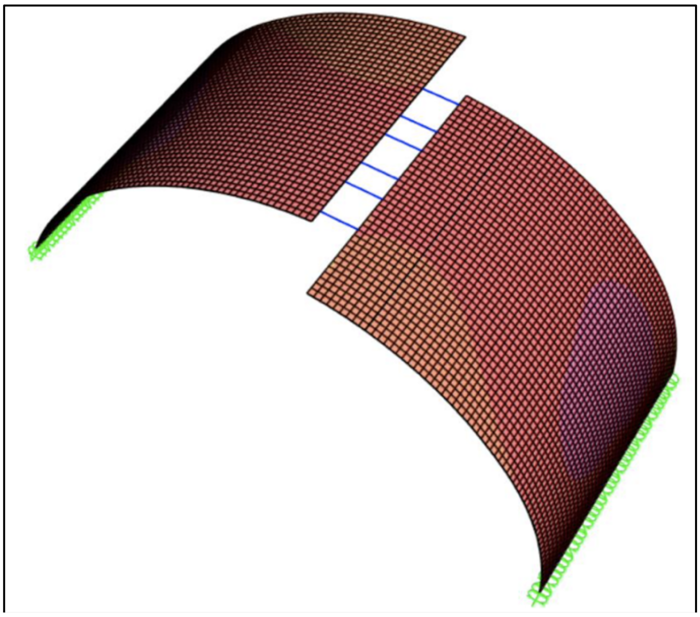


*Figure 5.5-1 - 3D view of the temporary phase made with SAP2000*

The support used at the connection between the prefabricated segments and cast in place elements is modeled as an articulated support:

- at the base (segment-sidewall) the condition is a simple support (unidirectional hinge)
- at the top (key) it is a tri-directional hinge only in correspondence with the bolt: the rest of the joint is free in order to maximize the bending in the construction stage.

These conditions simulate the most unfavorable phase, that is, when the segment is simply supported at the base and partially bolted in vault key.

The analysis allowed to evaluate the resulting axial force on the connection between the opposite segments (the rated maximum acting force is 33.85 kN), and the resulting shear force acting on the connection between segment and sidewall (the rated maximum acting force is 23,84 kN).

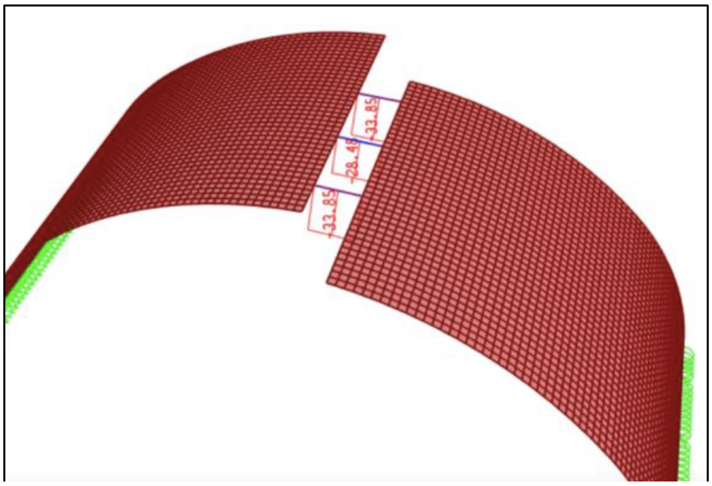


Figure 5.5-2 - Resultant axial force on the connection between opposite segments

The analytical analysis can be carried out by performing static verifications on the used inserts.
During the installation phase, the segments are connected in the lower part to the cast in place sidewalls, while in the upper part the segments are connected to each other thanks to the system of bolts and sleeves described in the previous chapter. To provide for such connections, it is planned to have on both edges of the segment special inserts consisting of anchor rail elements. All the type of inserts used in the temporary phase are shown in the Figure 5.5-3.

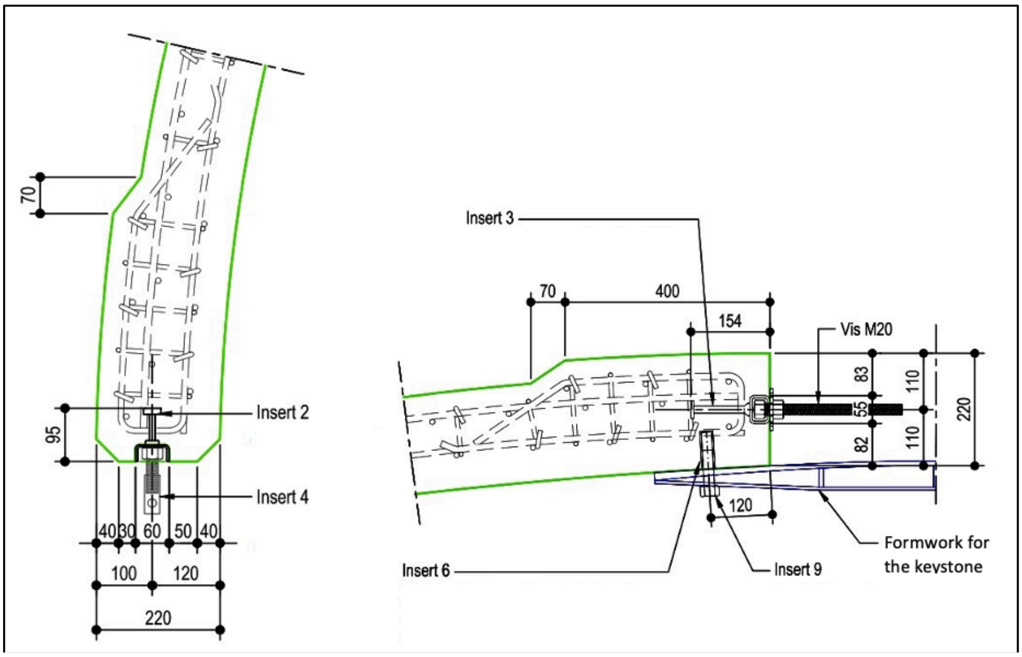


Figure 5.5-3 – Details of the anchor rail elements at the segment edges (lower edge on the left, upper edge on the right), (measures are in mm)

ID 2 inserts are used in the temporary and final phases, to precisely position the segments at level, in conjunction with inserts positioned on the sidewalls (ID 4 insert). It is planned to have N.2 elements "C" profile (reported in Figure 5.5-4) per segment anchored to the segment with N.5 bolts M20. According to the results of temporary conditions performed with the software SAP (reported in the chapter 5.8) a maximum shear force $F_y$ = 23,84 kN is applied on each anchoring bolt.

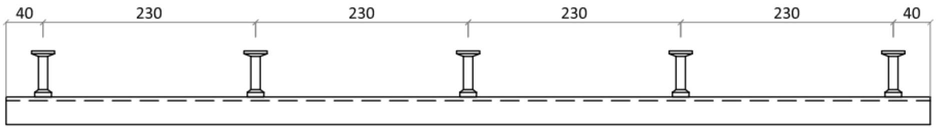



*Figure 5.5-4 – Anchor rail element installed on the lower edge of the segment (measures are in mm)*


*Figure 5.5-5 – Shear forces acting on the anchor rail*

Verifications:

- Insert section area:

$$A_{s,a} = \pi * r^2 = \pi * 5^2 = 78{,}54 \text{ mm}^2$$

- Area of the connection between anchor and channel:

$$A_{s,c} = a * b = 60 * 4 = 240 \text{ mm}^2$$

- Characteristic resistance for steel failure of the anchor:

$$V_{Rk,s,a} = A_{s,a} * f_{uk} = 78{,}54 * 520 * 0{,}001 = 40{,}84\ kN$$

- Characteristic resistance for steel failure of the connection between anchor and channel:

$$V_{Rk,s,c} = A_{s.c} * f_{uk} = 240 * 360 * 0{,}001 = 86{,}4\ kN$$

- Basic characteristic resistance for concrete failure of an anchor loaded perpendicular to the edge (effects of spacing, further edges and member thickness are not considered):

$$\alpha = 0{,}1 * \left(\frac{l_f}{c1}\right)^{0{,}5} = 0{,}1 * \left(\frac{80}{100}\right)^{0{,}5} = 0{,}089$$

$$\beta = 0{,}1 * \left(\frac{d_{nom}}{c1}\right)^{0{,}2} = 0{,}1 * \left(\frac{10}{100}\right)^{0{,}2} = 0{,}063$$

$$V_{Rk,c}^0 = k_9 * d_{nom}^{\alpha} * l_f^{\beta} * \sqrt{f_{ck}} * c_1^{1{,}5} = 2{,}4 * 10^{0{,}089} * 80^{0{,}063} * \sqrt{35} * 100^{1{,}5} * 0{,}001 = 22{,}96\ kN$$

- Characteristic resistance for concrete failure of an anchor loaded perpendicular to the edge:

$$V_{Rk,c} = V_{Rk,c}^{0} * \frac{A_{c,V}}{A_{c,V}^{0}} * \psi_{s,V} * \psi_{h,V} * \psi_{ec,V} * \psi_{a,V} * \psi_{re,V} = 22,96 * \frac{0,045}{0,045} * 1 * 1 * 1 * 1 * 1,4 = 32,14\, kN$$

- Characteristic resistance of an anchorage in case of pry-out failure:

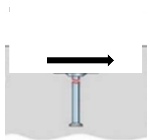
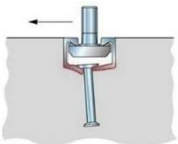
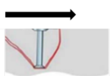
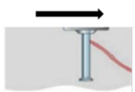
$$N_{Rk,c} = N_{Rk,c}^{0} = k_1 * \sqrt{f_{ck}} * h_{ef}^{1,5} = \left(11 * \sqrt{35} * 80^{1,5} * \frac{1}{1000}\right) * 0,5 = 23,28\, kN$$

$$V_{Rk,cp} = k_8 * N_{Rk,c} = 2 * 23,28 = 46,56\, kN$$

- Design value of the shear load acting on one anchor:

$$V_{Ed} = 23,84\, kN$$

*Table 5.5-1 – Shear verifications insert ID 2*

| | Shear verifications | |
|---|---|---|
| Type of failure | Equations | Demand and Capacity |
| Anchor failure  | $V_{Rd,s} = \frac{V_{Rk,s,a}}{\gamma_{Ms}} = \frac{40,84}{1,46} = 27,97kN > 23,84kN$ | $\frac{23,84}{27,97} * 100 = 85,23\%$ |
| Connection between anchor and channel  | $V_{Rd,s} = \frac{V_{Rk,s,c}}{\gamma_{Ms,ca}} = \frac{86,4}{1,8} = 48kN > 23,84kN$ | $\frac{23,84}{48} * 100 = 49,67\%$ |
| Pry-out failure  | $V_{Rd,cp} = \frac{V_{Rk,cp}}{\gamma_{Mc}} = \frac{46,56}{1,5} = 31,04kN > 23,84kN$ | $\frac{23,84}{31,04} * 100 = 76,80\%$ |
| Concrete edge failure[1]  | $V_{Rd,c} = \frac{V_{Rk,c}}{\gamma_{Mc}} = \frac{32,14}{1,5} = 21,43kN > 16,07kN$ | $\frac{16,07}{21,43} * 100 = 75,0\%$ |

The equations reported in the table consider the partial coefficients given in the "EOTA/TR 047, Section 4.3.2, Table 4.1" these values are in accordance with EN 1992-1-1.

ID 4 inserts are located into the sidewalls and are used in the temporary and final phase to connect the rails on the lower edge of the segment with the sidewalls. These inserts have the same characteristics and is subjected to the same loads of ID 2 inserts, therefore no additional verification is required.

[1] The verification of concrete failure is carried out considering 75% which is a cutelative value, the real load competent to the anchor is closer to 50%.

ID 3 inserts are used in the temporary and final phase, to precisely position the shells at level and guarantee their support during the assembly phase. According to the results of temporary conditions performed with the software SAP2000 shown in Figure 5.5-2, the acting force on the bolt and anchors is compression, so that shear and tension verifications are not needed during the temporary phase. For induced compression instability (buckling), refer to the following verification on the ID 8 inserts.

ID 8 inserts (Vis M20 in Figure 5.5-3) are used in temporary phase, to position the segment in connection with the keystone. It is planned to have bolt elements, Type M20 Quality 8.8. The insert was verified for induced compression instability (buckling) during installation in accordance with EN 1993-1-1:2005. The maximum compression value is the one evaluated by the software SAP2000 reported in Figure 5.5-2 (maximum compression acting on the bolt = 33.85 kN). The compressive strength of the ID 8 insert was evaluated according to the parameters given by the producer (maximum resistant force = 34.7 kN). The verification is satisfied because the resistance provided is greater than the acting force: 34.7 > 33.85 kN.

The analysis of the temporary phase must consider the static conditions present before and during the casting of the keystone. To perform the casting of the keystone, it is planned to install the formwork on top of the vault by means of a special inserts (ID 6) located on the upper edge of the segment (as shown in the Figure 5.5-3). It is planned to have 2 elements per segment characterized by M20 bolts.
To estimate the axial load acting on the insert ID 6, it is necessary to evaluate the weight of the concrete of the keystone. The Table 5.5-2 reports the calculation of the acting axial load by considering the weight of concrete and formwork for a length of tunnel of 2.5 m (that corresponds to the length of one segment).

*Table 5.5-2 - Insert ID 6 acting loads evaluation*

| | | **Concrete** | **Formwork** | |
|---|---|---|---|---|
| **Width** | [m] | 0.6 | 1.0 | |
| **Length (one segment)** | [m] | 2.5 | 2.5 | |
| **Height** | [m] | 0.24 | 0.01 | |
| **Volume** | [m$^3$] | 0.36 | 0.02 | |
| **γ** | [kN/m$^3$] | 24.0 | 78.5 | |
| **Factor** | [-] | 1.5 | 1.5 | |
| **Weight** | [kN] | 12.96 | 3.0 | Tot = 15.96 kN |

The evaluated weight will be divided into 4 inserts (2 on the left + 2 on the right). The load on each insert is therefore equal to 3.6 kN. Since the element is certified for a load of 20 kN by the manufacturer, the insert is therefore verified.

## 5.6 Permanent phase: structural verification after backfilling and long-terms validation

The principle is to study the behavior of the coating (final structure) under the action of external loads. Stresses are calculated by conventional methods, taking into account the strength of materials. The different parts of the final concrete structure (prefabricated segments and cast-in-place concrete structures) will be modeled using bar elements characterized by their inertia, cross-sections and parameters concerning the type of material.

It is important to note that the operation of the vault (including the pieces) is equivalent to a beam operation (and not slab) because of the absence of continuity longitudinally between segments. This results in compliance with the recommendations regarding the justifications for shear force and reinforcement of the associated beams.

The additional three-dimensional analysis plays an essential role in the study of conditions where the 2D behavior hypothesis is not valid. It allows a more complete understanding of the structural response by taking into account the complexities and interactions that occur in the three dimensions as was done in the garage section. Here are some specific aspects where 3D analysis offers significant advantages:

• Load distribution: In 3D analysis, the load distribution across the structure can be accurately captured. This is particularly important when dealing with irregular geometries, complex load patterns or load transfer mechanisms that cannot be adequately represented in a 2D model. The three-dimensional nature of the analysis ensures a more realistic representation of load paths and their effects on the structure.

• Localized effects: 3D analysis allows the study of localized effects that occur inside the structure. This includes the assessment of stress concentrations, localized deformations and effects of local details such as connections, joints or specific properties of materials. Taking into account these localized effects, the analysis can identify critical areas that may require special attention in terms of design or reinforcement.

The soil-structure interaction will be modelled using non-linear soil springs. These springs schematize the mechanical behavior of the surrounding ground. The reactions of the ground are mobilized only when the direction of the deformation of the surface puts the ground in stop, the modulus of reaction of the springs is zero in the direction of the extension and is active in compression.

The actions are applied by distinguishing the active loads, which are independent of the deformation state, and the so-called passive loads which correspond to the hyperstatic reactions resulting from the deformation of the lining structure.

Active loads include the pressures applied to the lining by the surrounding ground, the hydrostatic pressure which also depends on the type of drainage, the detachment of a block, the weight of the coating and the thermal actions.

Passive charges are the ground stop reactions. The latter are proportional to displacements and modeled by a series of springs whose reaction modulus is derived from the mechanical properties of the host rock.

For the long-term verification of the new final lining structure, numerical modelling programs were used to estimate and analyze the stresses and deformations of the final structure. The analysis was done using three software programs: PLAXIS 2D, SAFIR

2D, SAP2000. In the diagram of Figure 5.6-1, the steps and different tools used to verify and analyze the final coating are reported.

The use of different software, both geotechnical and structural, has several advantages to strengthen the robustness of the design during this phase. By comparing the results and validating the models, a more complete understanding of the project can be obtained. Specific advantages of adopting different software include:

-Realistic Node Behavior Assessment (PLAXIS). The PLAXIS software allows a realistic assessment of node behavior, allowing a better understanding of their response to different conditions and loads, appropriate to those of the PFD.

-Validation of spring stiffness hypotheses (PLAXIS-SAFIR comparison). By comparing the results between PLAXIS and SAFIR, hypotheses regarding spring stiffness can be validated. This ensures that the selected stiffness values are appropriate for the analyzed structure.

-Evaluation of three-dimensional design effects (cantilever effect - SAP2000 3D). The use of SAP2000 3D facilitates the evaluation of three-dimensional design effects, particularly in the temporary phase. This includes taking into account the cantilever effect, which might not be correctly captured in a 2D analysis.

-Validation of structural model assumptions (SAP-SAFIR comparison). Comparing results between SAP and SAFIR helps to validate all assumptions made in structural models. This ensures that the chosen assumptions are in accordance with the observed behavior in the analyzed structure.

-Precise analysis of segment structures with evaluation of longitudinal effects (Out of plan) not possible with 2D analysis only. A complete analysis of hull structures can be performed taking into account longitudinal effects, especially those occurring out of plan. This analysis goes beyond the limits of a 2D analysis, offering a more accurate representation of the behavior of the structure.

-Accurate analysis of localized effects on the inserts of the segment structure. Using available software, localized effects on the inserts within the segment structures can be accurately analyzed. These effects may not be fully captured in a 2D analysis, underscoring the importance of a more detailed approach.

-Evaluation of discontinuity effects in the longitudinal direction of the segment structures. Software allows modelling and evaluation of effects resulting from discontinuities in the longitudinal direction of hull structures. This is an aspect that cannot be properly addressed in a 2D analysis.

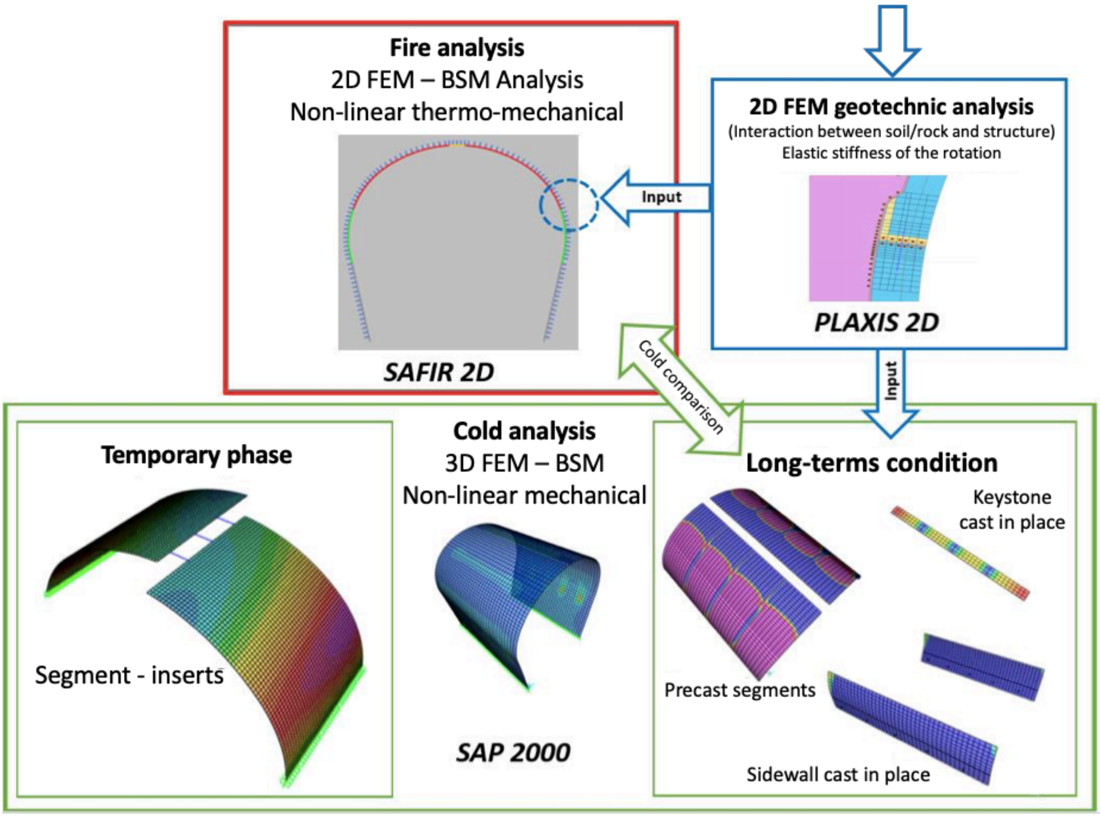
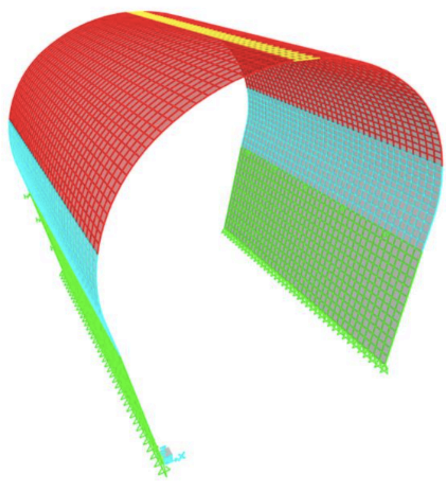



Figure 5.6-1 – Steps and different programs used to check and analyze the new final lining

One of the main phases of modelling consists in the insertion of the mechanical properties of the materials. In the following image the type of concrete and of reinforcement steel are reported. The class of cement makes it possible to deduce the main mechanical properties and, through the use of numerical models, makes it possible to predict the behavior of the structure of the final lining.




| COLOUR | TYPE | MATERIAL | THICKNESS (cm) |
|---|---|---|---|
| | EXISTING CONCRETE | ELASTIC MATERIAL | 40 |
| | IN-SITU WALL | C35/45 | 24 |
| | PRECAST SEGMENT | C35/45 | 18 |
| | CROWN | C35/45 | 24 |

Figure 5.6-2 – Materials mechanical properties for numerical modeling

The aim of the injection is to create a backfilling just like the one realized in the mechanized tunnel excavation process. This avoids the possibility of settlement of the surrounding rock, and, in addition, it increases the degree of waterproofing of the tunnel itself. After the injection of the micro-cement, the structure will have to

withstand a compression. The forces acting on the structure can be approximated as a constant load acting perpendicular to the structure. This condition was considered among those for the long-term verification of the new final lining.
The different structure loading configurations considered for the long-term analysis are reported in Figure 5.6-3. These configurations allow to simulate the most frequent and the worse load cases to which the structure is subjected according to the geometry and the size of the wedges found in the UNWEDGE analysis. For each case a load of 100 kPa was considered.

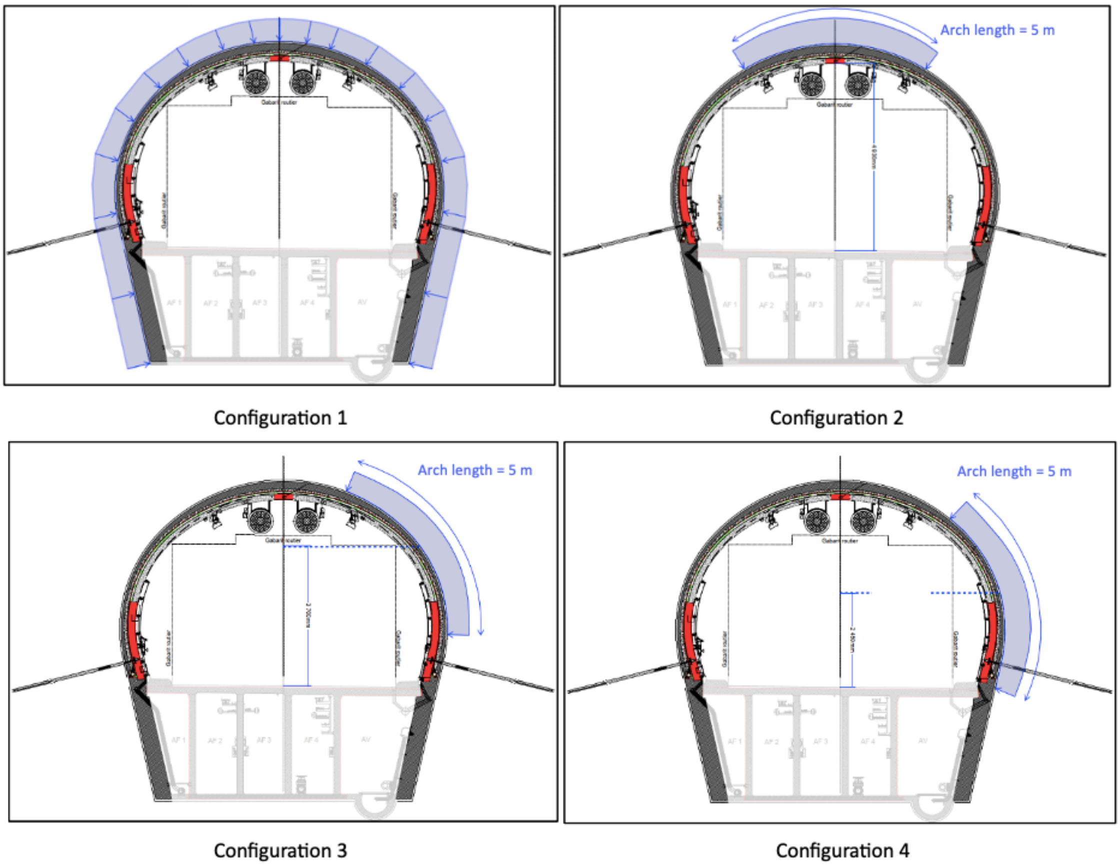



*Figure 5.6-3 - Different loading configuration considered for the long-terms verification*

1. PLAXIS 2D (geomechanical analysis)

PLAXIS 2D is a finite element method (FEM) software to be used for 2D analysis of deformation and stability in geotechnical and mechanical rock engineering. PLAXIS 2D includes all the essential elements to perform deformation analysis and safety for soil and rock that do not require consideration of flow, stationary groundwater or thermal flow, consolidation analysis or any time-dependent effect.
It was used to perform a structural analysis of the connection between the cast in place concrete elements and the precast segments (sidewall-segment and keystone segment). The connections are considered as structural hinges. PLAXIS 2D defines the elastic stiffness of the rotation hinge and checks the localized pressure taking into account the appropriate interaction between the soil/rock and the structure. The result of PLAXIS 2D is a function of the strain-stress relationship of the ground springs.

Modelling is divided into the following steps:
1. Initialization of stress state according to conditions
2. Release of total stresses
3. Simulation of final lining
4. Evaluation of rotational stiffness for both connections

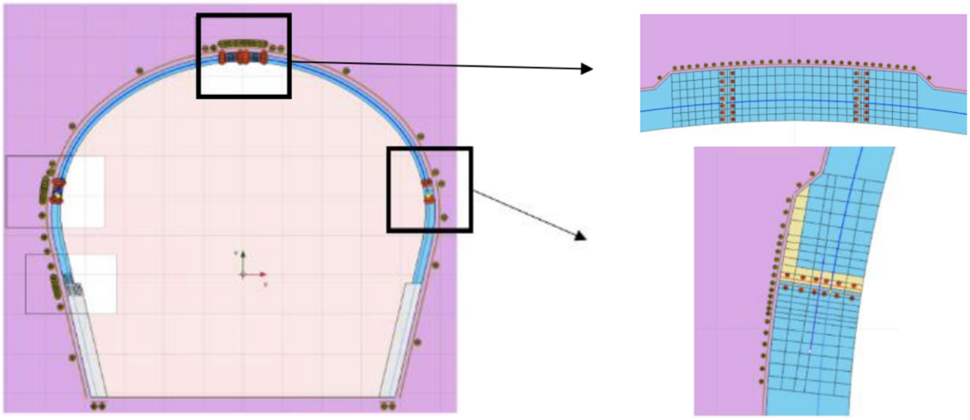


*Figure 5.6-4 – PLAXIS 2D: analyzed connections*

The rotational stiffness of the joint is simply calculated by dividing the bending moment by the angular rotation, the results are summarized in the table below:

*Table 5.6-1 – Rotation stiffness of the connection sidewall/segment, results*

| θ [rad] | M [kNm] | $k_{ROT}$ [KNm/rad] |
|---|---|---|
| 1,78E-04 | 15.2 | 85382 |

*Table 5.6-2 – Rotation stiffness of the connection keystone/segment, results*

| θ [rad] | M [kNm] | $k_{ROT}$ [KNm/rad] |
|---|---|---|
| 1,08E-04 | 8.1 | 75218 |

The analysis of the connections also allowed to estimate in first approximation the values of the forces acting at the interface between the prefabricated and cast in place elements. The following image shows the stress state at the interface between precast segments and keystone.

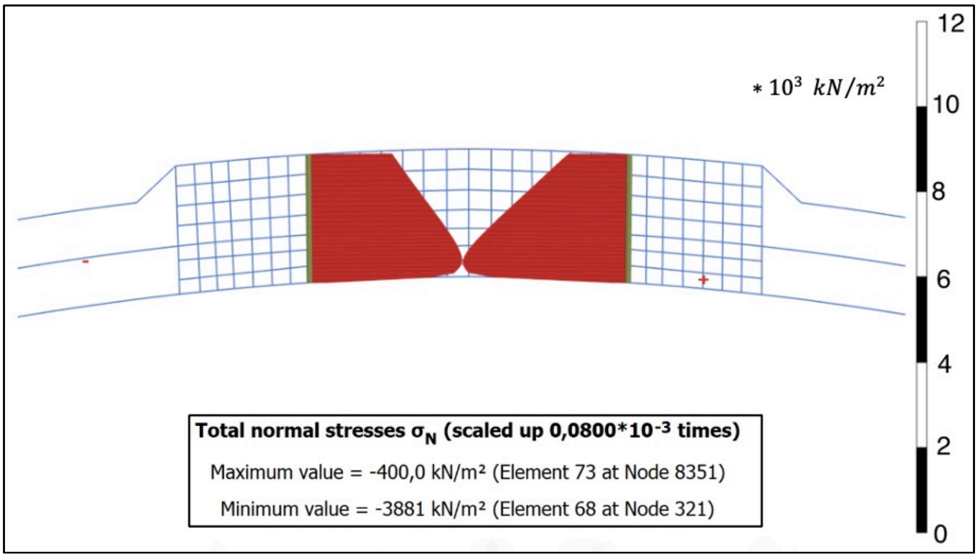



*Figure 5.6-5 – PLAXIS 2D: State of stress at the interface segment/keystone*

It is important to note that stress agents are only compressive stress acting along the x-direction, so the keystone is a structural element that works exclusively under compression.

2. SAFIR 2D (thermal and mechanical analysis)

The SAFIR 2D software was used for the thermal analysis of the final lining, the 2D software takes into account a non-linear behavior of the material, but also the degradation of mechanical properties induced by fire.
Fire analyses are carried out with the different load configurations indicated above and the effect of taking into account or not the detachment of a 5 cm layer of concrete due to the phenomenon of desquamation is studied. Convergence is reached after 120 minutes for all the studied cases and, therefore, the design requirement is validated.

3. SAP 2000 3D (the structural analysis)

The SAP 2000 3D software includes 2 models: a first model to justify temporary phases (discussed above), and a second model to justify the final phase by considering long-term properties and geometries. These two models are used for cold analysis of the structure: the SAP2000 software is not used for fire-induced degradation calculations. Regulations recommend and require the adoption of diversified modeling tools to compare results and demonstrate robustness of design solutions. For this reason, SAFIR has been used for mainly fire but also cold 2D analysis, and compared with 3D analyses made with SAP2000 that were essential for the evaluation of the off-plane effects. The comparisons of the results of the 3D analysis with those obtained by SAFIR in the absence of thermal constraints were made, obtaining a good consistency.

The model of the definitive phase has been used to verify the single elements of the structure of the final lining.
- Definitive phase: segment flexural verification

The main aim of flexural verification was to estimate the minimum quantity of reinforcing steel inside the prefabricated segments. The analysis carried out with SAP 2000 determined the minimum steel quantity of the segments along 2 directions (local axes: 1-Longitudinal 2-Transverse).

The reinforcement values shown in the images below represent the minimum necessary to ensure the satisfaction of the verification under the pressure and flexural combination, therefore the actual steel reinforcement quantity will be higher than the values stated below.

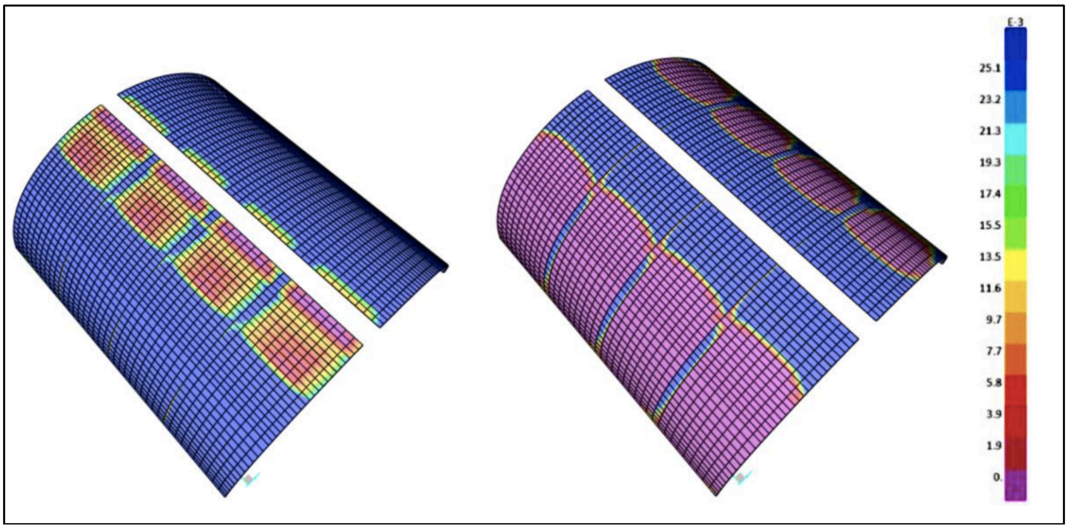



*Figure 5.6-6 – Minimum required steel reinforcement (values on the lateral scale: cm²/cm) on the segment extrados (left) and intrados (right), transversal direction*

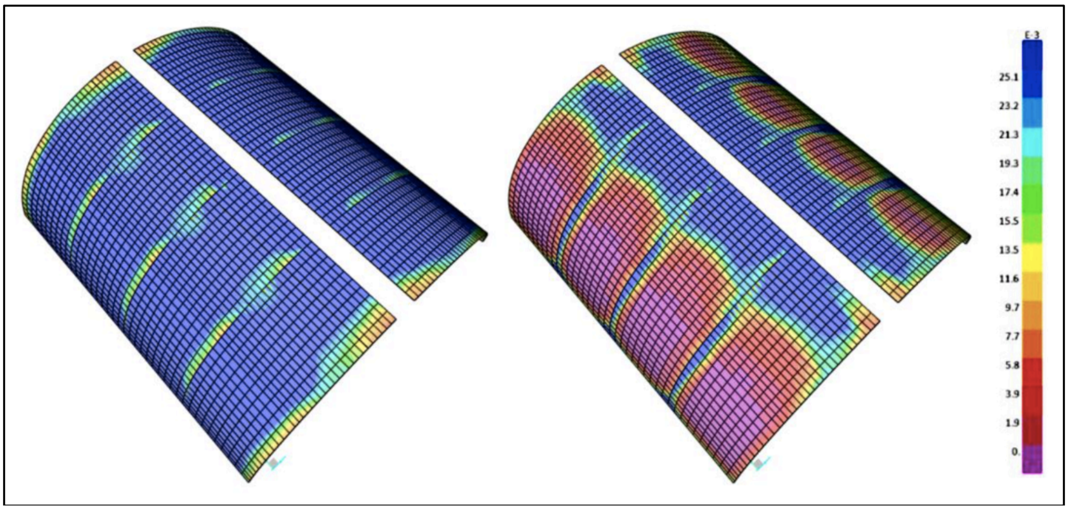



*Figure 5.6-7 – Minimum required steel reinforcement (values on the lateral scale: cm²/cm) on the segment extrados (left) and intrados (right), longitudinal direction*

The quantities effectively adopted for the design are reported in the following table.

*Table 5.6-3 – Adopted quantities of steel reinforcement for precast segments (bars diameter/longitudinal spacing/transversal spacing in mm)*

| Position | Renforcement |
|---|---|
| Transv. Intrados | Welded reinforcing mesh $\phi$10/100/100 |
| Long. intrados | |
| Transv. Extrados | $\phi$12/100 with an additional reinforcement of $\phi$12/100 |
| Long. extrados | $\phi$10/100 |
| Section élargie | $\phi$12/100 |

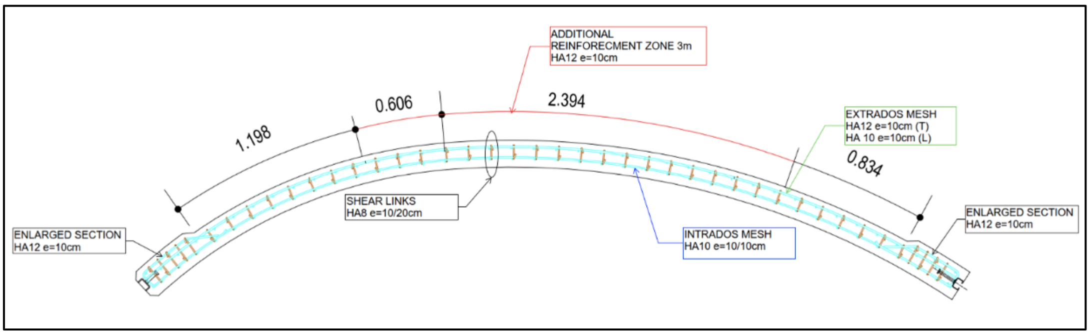



*Figure 5.6-8 - Steel reinforcement of precast segments*

- Definitive phase: sidewalls flexural verification

The same procedure was adopted for the cast in place sidewalls. The output of the numerical model is reported below.

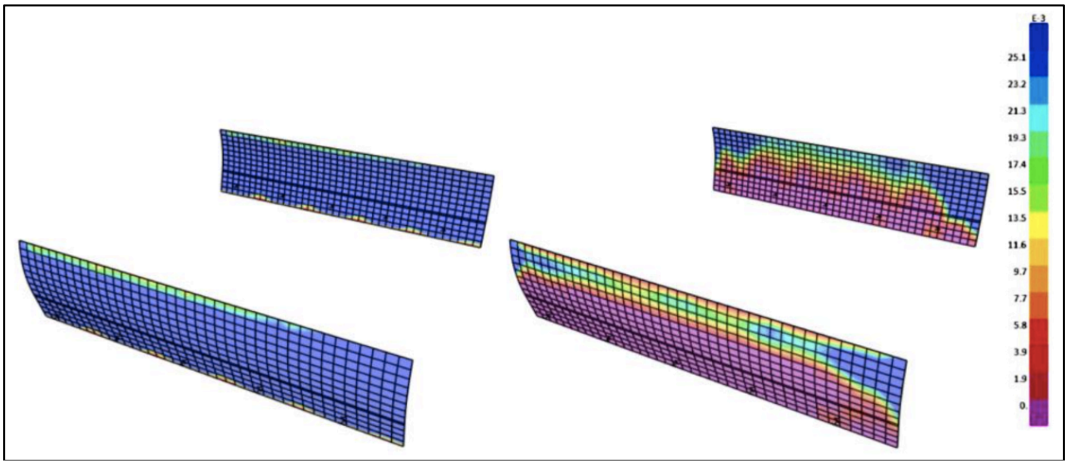



*Figure 5.6-9 – Minimum required steel reinforcement (values on the lateral scale: cm²/cm) on the sidewall extrados (left) and intrados (right), transversal direction*

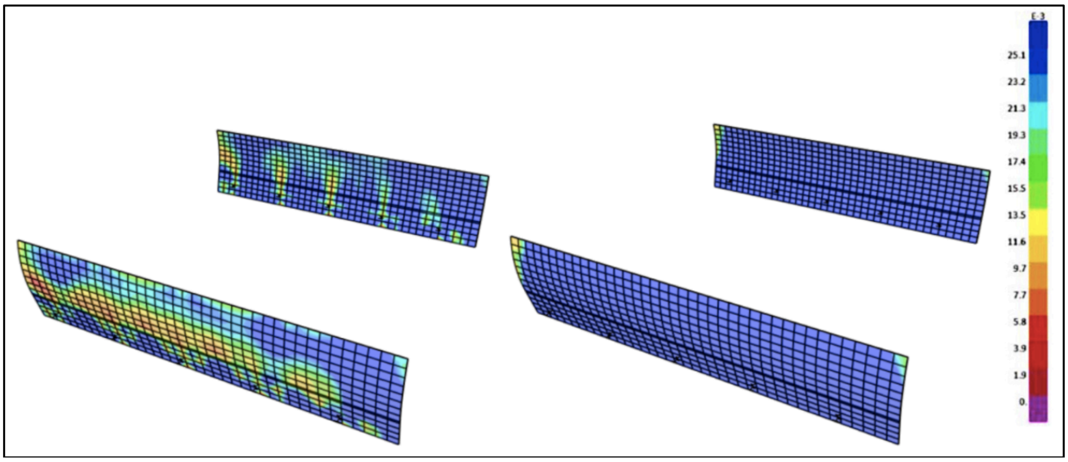


*Figure 5.6-10 – Minimum required steel reinforcement (values on the lateral scale: $cm^2/cm$) on the sidewall extrados (left) and intrados (right), longitudinal direction*

*Table 5.6-4 – Adopted quantities of steel reinforcement for cast in place sidewalls (bars diameter/longitudinal spacing/transversal spacing in mm)*

| Position | Renforcement |
|---|---|
| Transv. Intrados | ϕ16/100 |
| Transv. Extrados | ϕ16/100 |
| Long. intrados | ϕ10/150 |
| Long. extrados | ϕ10/150 + ϕ12/150 |

In this case an additional steel reinforcement must be provided in the surrounding zone of the rockbolt that will be installed with a spacing of 2 m. This type of reinforcement is shown in Figure 5.6-11 in the red rectangle.

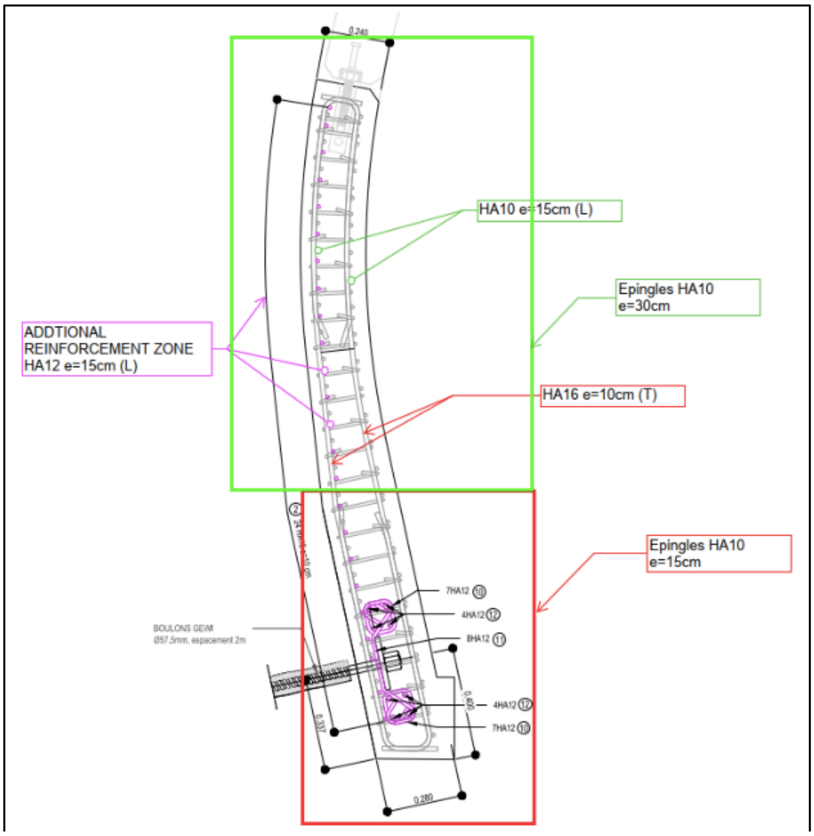



*Figure 5.6-11 – Steel reinforcement of the sidewalls*

- Definitive phase: keystone flexural verification

The same procedure was adopted for the cast in place sidewalls. The output of the numerical model is reported below. The keystone, being a structural element that works mainly at compression will need a smaller amount of reinforcing steel. This is also evident in the following images, where, especially in the parts that refer to the intrados (where the compression is greater), there is a lower value respect to the previous structural elements.

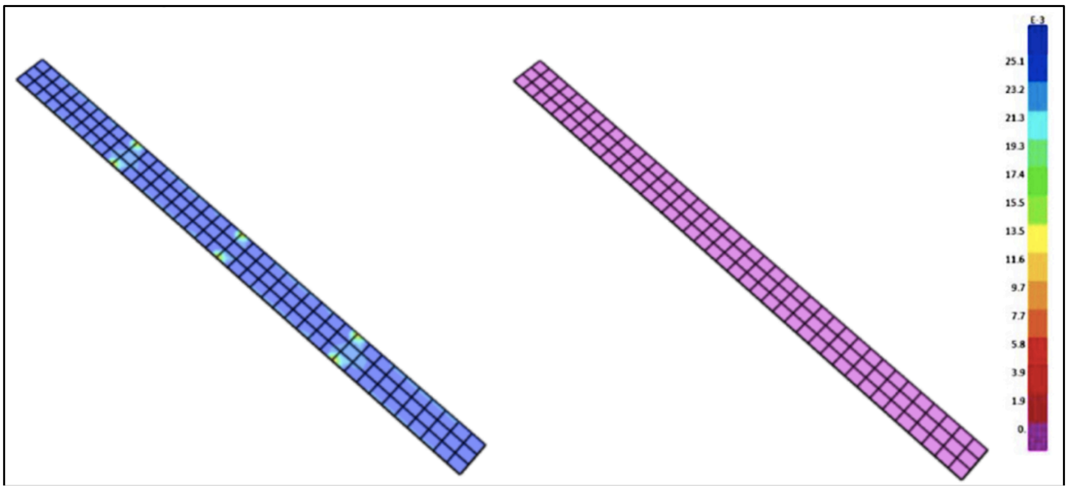



*Figure 5.6-12 – Minimum required steel reinforcement (values on the lateral scale: cm2/cm) on the keystone extrados (left) and intrados (right), transversal direction*

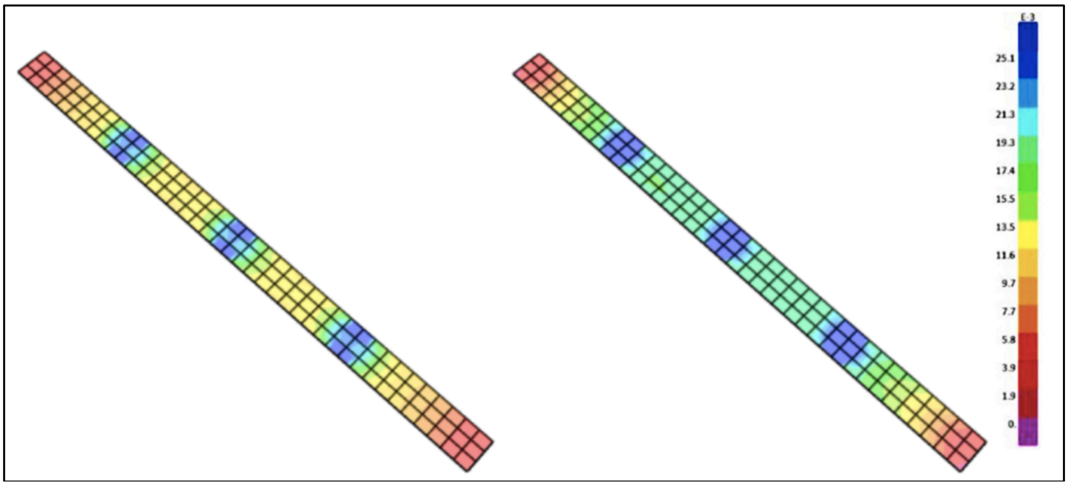


Figure 5.6-13 – Minimum required steel reinforcement (values on the lateral scale: cm2/cm) on the keystone extrados (left) and intrados (right), longitudinal direction

Table 5.6-5 – Adopted quantities of steel reinforcement for cast in place sidewalls (bars diameter/longitudinal spacing/transversal spacing in mm)

| Position | Renforcement |
|---|---|
| Long. intrados | 4 ϕ10 |
| Long. extrados | 4 ϕ10 |
| Trans. Intrados | ϕ12/100 |
| Trans. Extrados | ϕ12/100 |

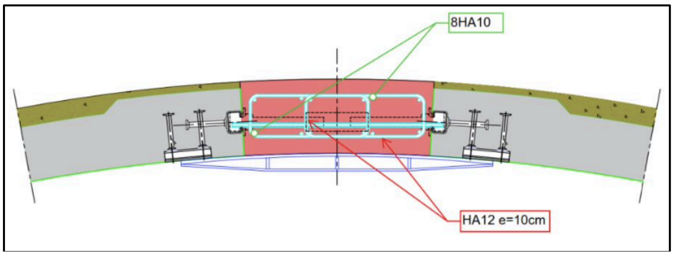



Figure 5.6-14 – Steel reinforcement of the keystone

## 5.7 FLAC3D introduction to the software

During the activities carried out in the company Pini Group, I had the opportunity to learn a software for numerical modeling: FLAC3D, and to use it in some application cases concerning the project dealt with in this thesis.

FLAC3D (Fast Lagrangian Analysis of Continua in 3 dimensions) developed by Itasca is a software based on FDM (Finite Difference Method) that allows to carry out numerical analysis with the aim of finding a solution to engineering problems. The problem is represented by differential equations, the FDM allows to solve them through an approximation to linear variations on finite intervals of space and time. The calculations are repeated following a series of iterations (steps) until the results converge and then the equilibrium state is achieved.

Each element of the model will have a specific stress-strain behavior that can be linear or non-linear. To simulate these differences between materials, in FLAC3D it is possible to assign different constitutive models to individual elements. The analysis will evaluate the response of each element to specific external actions.
The coordinates of the individual finite elements are determined with the implementation of the Lagrangian formulation, this feature allows the numerical model to realistically represent the plastic deformations of the elements by updating the coordinates at each iteration. Mainly for this reason this software is largely used for the simulation of complex geotechnical problems with 3D analysis of soil, rock, and structural supports and interactions between them.

Data files are text files that form the basis of FLAC3D numerical modeling. Each file contains a code with a set of commands that can be executed to perform certain actions inside the software. Normally, a project is made up of a series of data files in order to organize and distinguish the various phases of modeling.
Data files can be also "called" inside other data file.

The workflow to perform a numerical analysis with FLAC3D can be roughly subdivided into the following steps:

- Geometry
- Constitutive model and properties
- Boundary conditions
- Initial conditions
- Solution (instantaneous or gradual)

FLAC3D also gives the possibility to create parametric projects using the "fish" commands. These allow you to use within the code variables that can be modified by the user in order to adapt the processing to each specific case.

1. GEOMETRY
The geometry of the model, which also includes the definition of the discretization level, can be developed by creating grid zones. FLAC3D has three main built-in methods for creating grid zones: primitives, extrusions and building blocks. In addition, the lasts version of the program, allows to create grid zones from imported files (CAD, Rhino, etc.).

**Primitive-Based Grids** is the most direct way to create different geometries. This approach is generally most suitable for regular shapes; however, the zones can be distorted to fit arbitrary and complicated volumetric regions. Primitive-based grids is based on a series of commands each of which corresponds to a simple geometry (see table below). These commands can be associated with "keywords" that allow to define position, size, level of discretization and other features of the grid zone. Through the combination of these simple shapes, it is possible to get even more complex and elaborate geometries.

*Table 5.7-1 - Primitive mesh shape available with "zone create" command*

| Keyword | Definition |
|---|---|
| `brick` | brick-shaped mesh |
| `wedge` | wedge-shaped mesh |
| `uniform-wedge` | uniform wedge-shaped mesh |
| `tetrahedron` | tetrahedral-shaped mesh |
| `pyramid` | pyramid-shaped mesh |
| `cylinder` | cylindrical-shaped mesh |
| `degenerate-brick` | degenerate brick-shaped mesh |
| `radial-brick` | radially graded mesh around brick |
| `radial-tunnel` | radially graded mesh around parallelepiped-shaped tunnel |
| `radial-cylinder` | radially graded mesh around cylindrical-shaped tunnel |
| `cylindrical-shell` | cylindrical shell mesh |
| `cylindrical-intersection` | intersecting cylindrical-shaped tunnels |
| `tunnel-intersection` | intersecting parallelepiped-shaped tunnels |

**Extrusion-Based Grids** can be used through the extrusion pane. This pane provides a facility for creating grids based on two-dimensional geometries that are extruded in the third dimension. This method, through a simple graphical interface, allows to insert points (graphically or through the insertion of coordinates for greater precision) and segments in order to delimit triangles and quadrilateral that will form the zones of the model. The discretization can be customized to each individual segment to determine the thickness of the final mesh. Once the two-dimensional model is completed, it is possible to proceed with the extrusion in the third dimension by defining the length and discretization of the segment along which the extrusion will take place. Extrusion pane also allows to import bidimensional CAD drawings (.dxf), and, in the latest versions of FLAC3D, it is possible to automatically recognize contour lines and zones by using the “automatic zone” command. Extrusion-Based Grids is particularly useful for the realization of tunnels geometry with even complex sections, although it is limiting in defining elements that are not continuous along the entire length of the extrusion.

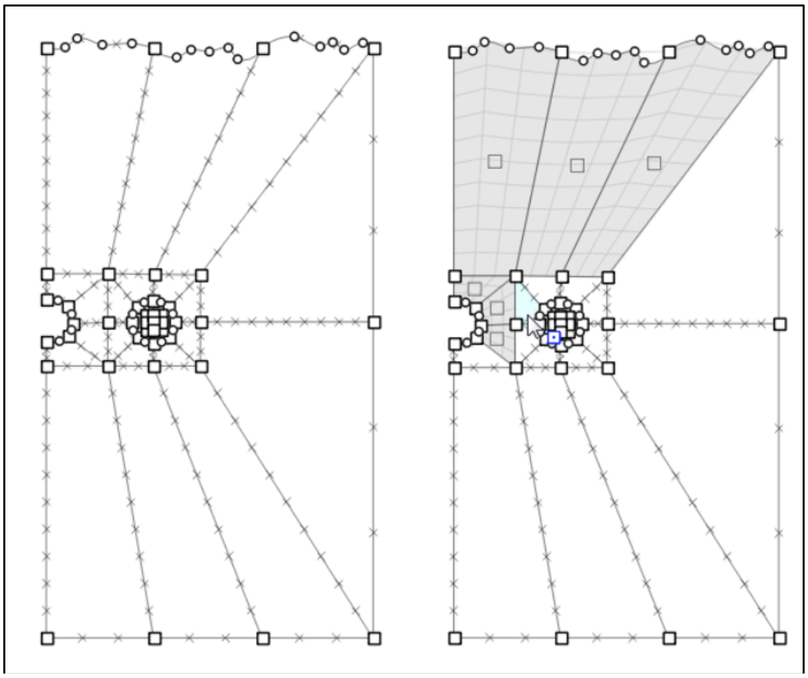


*Figure 5.7-1 - Extrusion pane*

**Building Blocks-Based Grids** can be used through the building blocks pane. This pane provides a facility for creating grids based on three-dimensional "building blocks" that are fitted together and then zoned. This method, through a simple graphical interface, allows to create the geometry by adding tridimensional blocks and to locate them inside the model. With simple graphical actions it is possible to manipulate the shape of single blocks to create edges or different inclinations. The building blocks pane also allows to import tridimensional CAD drawings (.dxf) and to automatically define the 3d geometry. This method, with respect to the Extrusion-Based Grids, is more complicated and takes more time, but allows to define any type of tridimensional geometry.

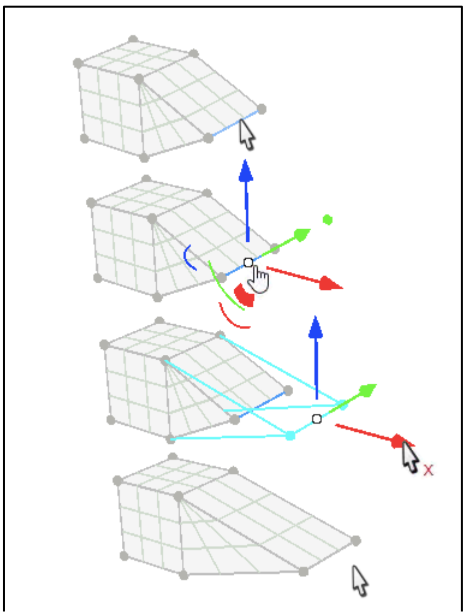


*Figure 5.7-2 - Selecting and moving an edge of a block*

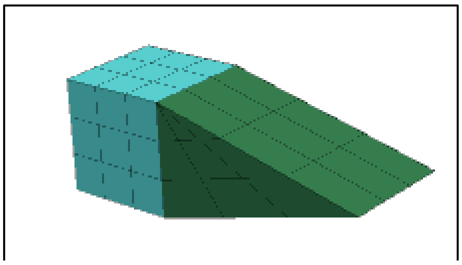


*Figure 5.7-3 - Zone generated from a building blocks set*

Once the tridimensional geometry is defined, it is possible to save it as a data file. If the Primitive-Based Grids method is used, the data file is simply constituted by the series of primitive functions and keywords. If it is used the Extrusion-Based Grids or Building Blocks-Based Grids methods, it is possible to convert the series of graphical command into code through the "state record" command and save that code as a data file. Every time we need to recreate the geometry, it is possible to simply run that file and the program will recover the designed shape.

2. CONSTITUTIVE MODEL AND PROPERTIES

The constitutive models are used to describe the mechanical properties of each material inside the numerical model through mathematical equations. By using the command “zone cmodel assign” it is possible to set the most suitable constitutive model to each element. FLAC3D allows the user to choose among a wide list of constitutive models concerning both elastic and plastic behavior, for each available constitutive model, the software implement a description of the mathematical model in order to guide the user in choosing the most suitable governing equations.

*Table 5.7-2 - Constitutive Models available in FLAC3D*

| Model | Representative Material | Example Application |
|---|---|---|
| null | void | holes, excavations, regions in which material will be added at later stage |
| elastic | homogeneous, isotropic continuum; linear stress-strain behavior | manufactured materials (e.g., steel) loaded below strength limit; factor of safety calculation |
| orthotropic elastic | materials with three mutually perpendicular planes of elastic symmetry | columnar basalt loaded below strength limit |
| transversely isotropic elastic | thinly laminated material exhibiting elastic anisotropy (e.g., slate) | laminated materials loaded below strength limit |
| Drucker-Prager | limited application; soft clays with low friction | common model for comparison to implicit finite-element programs |
| Mohr-Coulomb | loose and cemented granular materials; soils, rock, concrete | general soil or rock mechanics (e.g., slope stability and underground excavation) |
| strain-hardening/softening Mohr-Coulomb | granular materials that exhibit nonlinear material hardening or softening | studies in post-failure (e.g., progressive collapse, yielding pillar, caving) |
| ubiquitous-joint | thinly laminated material exhibiting strength anisotropy (e.g., slate) | excavation in closely bedded strata |
| anisotropic-elasticity ubiquitous-joint | thinly laminated material exhibiting stiffness and strength anisotropy (e.g., slate) | excavation in closely bedded strata |
| bilinear strain-hardening/softening ubiquitous-joint | laminated materials that exhibit nonlinear material hardening or softening | studies in post-failure of laminated materials |
| double-yield | lightly cemented granular material in which pressure causes permanent volume decrease | hydraulically placed backfill |
| modified Cam-clay | clay | geotechnical construction on clay |
| Hoek-Brown | isotropic rock material | geotechnical construction in rock; factor-of-safety calculation |
| Hoek-Brown-PAC | isotropic rock material | geotechnical construction in rock |
| CYSoil and CHSoil | soils | excavation, tunnel, slope stability, embankment, foundation analysis |
| Plastic-Hardening | soils | excavation, tunnel, slope stability, embankment, foundation analysis |
| swell | soils with wetting-induced deformations | application on soils where wetting-induced deformations are significant |
| Mohr-Coulomb-Tension | rocks and soils | dynamic response or deformation of the overburden above the undercut |

After choosing the constitutive models it is possible to assess the set of properties for each material present inside the model by adding specific "keywords" to the main command. Different constitutive models require different properties, inside the guide (accessible with the help pane) are reported all the properties requested for each constitute model. In the image an example of code to assign the constitutive model is reported, in this case it assigns two different set of properties to two different groups of elements.

```
; assign Mohr-Coulomb material model and properties
zone cmodel assign mohr-coulomb
zone property bulk 4e8  shear 1.5e8 friction 20 cohesion 50e3 ...
              tension 5e3 dilation 3 density 2200
zone property bulk 50e6 shear 18e6  friction 20 cohesion 25e3 ...
              tension 0 dilation 0 density 2200 range group 'soil'
```

*Figure 5.7-4 – Example code: to assign a Constitutive Model and properties of a specified group*

3. BOUNDARY CONDITIONS

Boundary conditions in a numerical model consist of values of field variables (i.e., stress and displacement) that are prescribed at the boundary of the numerical grid. The boundaries are of two categories: real and artificial. Real boundaries exist in the physical object being modeled (i.e., a tunnel surface or the soil surface). Artificial boundaries do not exist in reality but must be introduced to encompass the chosen number of zones. Mechanical conditions that can be applied at the grid boundaries are of two main types: prescribed stress or prescribed displacement.

By default, the boundaries of a FLAC3D grid are free of stress and any constraint. Forces or stresses may be applied to any boundary, or part of a boundary, by means of the "zone face apply stress" command, after the keyword "stress" it is possible to specify the directions along which the stress will be applied. With the addition of "keywords" it is possible to define the surface and the value of stress to be applied, moreover, this command also allows to give a gradient of variation along a given direction.

```
zone face apply stress-xx -10e6 gradient (0,0,1e5) range position-z -100 0
```

*Figure 5.7-5 – Example code: definition of boundary conditions by applying a stress along x-direction*

Displacements cannot be controlled directly in FLAC3D; in fact, they play no part in the calculation process. In order to apply a given displacement to a boundary, it is necessary to prescribe the boundary's velocity for a given number of steps. In most geotechnical problems, the velocity applied to the boundary is zero as it has the function of simulating the continuity of the medium. In FLAC3D there are two command that can be used to assess this condition: "zone face apply normal-velocity = 0" or "zone gridpoint fix velocity", in both cases it is possible to define the direction and the position of the condition to be applied. The main differences between the two commands lies in the possibility or not to give a speed different from zero, moreover the command "zone gridpoint fix velocity" fixes the condition on all the nodes of the grid present in the declared range, while the “zone face apply” command applies the condition only to a surface.

```
; assign boundary conditions
zone face apply velocity-normal 0 range group 'East' or 'West'
zone face apply velocity-normal 0 range group 'North' or 'South'
zone face apply velocity-normal 0 range group 'Bottom'
```

*Figure 5.7-6 – Example code: definition of boundary conditions by fixing the velocity*

In the example reported below the two commands are perfectly equivalent because both fix the velocity at zero and are both applied to a surface identified with the name ‘East’.

```
zone gridpoint fix velocity (1e-5,0,0) range group 'East'
zone face apply velocity (1e-5,0,0) range group 'East'
```

*Figure 5.7-7 – Example code: definition of boundary conditions by setting an initial velocity*

4. INITIAL CONDITIONS

Initial conditions in a numerical model are used to Initialize zone stresses based on the density of the zones above them and gravity. In FLAC3D it is possible to set the initial conditions by using two main commands: “model gravity” and “zone initialize-stresses”. The first is used to define the value of the gravity acceleration, while the second command is used to transfer to all zones in the range, the stress given by the total column of zone densities above them. The “zone initialize-stresses” command also enable to define the “K” parameter of the soil (horizontal to vertical stress ratio; by default is taken equal to 1) to define the overburden (the depth at which the top surface of the model is located).

```
; assign initial stress state
model gravity 10
zone initialize-stresses ratio 1.0 0.5
```

*Figure 5.7-8 – Example code: definition of the initial state of stress*

5. MODEL SOLVE AND SAVE

The command "model solve" is used to start the calculation operations. FLAC3D uses a timestep-based resolution method to solve differential equations, therefore the solution is reached after a series of computational iterations called steps. By default, the software iterates until it reaches a convergence value of $10^{-5}$, however there are many options that allow the user to manually manage the number of steps by adding the appropriate keywords after the command mentioned above (i.e., to set a specific number of cycles, to define a maximum time of calculation, to set a different convergence value, to specify a solve limit for the creep process).

After the “model solve” it is often convenient to enter the “model save” command, this action allows the user to save the result as a “.sav” file inside the project. This allows to open or recall in other parts of the code the result obtained by "model solve" without having to fix the model again. The command to recall the results in another data file is “model call” followed by the name of the saved file.

6. MODEL RESULT AND PLOT

Once the program has evaluated the model solution, it is possible to see the results by adding new plots. This tool allows to see many aspects of the model result by showing graphs or 3D mesh models with a color scale that can represent different features of the result (i.e., stresses, displacements, strain). By convention, in FLAC3D, stress of tension and elongation deformations are positive.

## 5.8 Structural analysis with FLAC3D

The use of FLAC 3D for the application of the current project has allowed to carry out further structural verifications in both temporary and long-term conditions. My task in the company was to carry out a structural analysis of the current section by using the Itasca software.  The main aim of this analysis was to increase the robustness of the results obtained with the software described above. In particular, FLAC3D was used for the detailed study of the keystone/segment connection for the long-terms verification.

The FLAC project was subdivided into 4 data files representing different modeling phases.

00_setup: definition of the geometry, level of discretization and groups.

01_initial: definition of the boundary conditions and initialization of the model by activation the stress inside the entire rockmass

02_excavation: removal of the void representing the tunnel and evaluation of the total release of the rockmass.

03_lining_activation: activation of the new lining.

04_load: activation of a localized load on the lining.

00_setup

To perform this analysis, a three-dimensional finite element model was developed taking into account the entire structure of the final lining.  The geometry was realized by means of the Extrusion-Based Grids going to define the structural elements as individual blocks. To have a good degree of detail of the analyzed area, the key element has been discretized with a mesh of 6x15. The two-dimensional geometry was then extruded for a length of 25 m by considering 10 consecutive segments 2.5 m each.

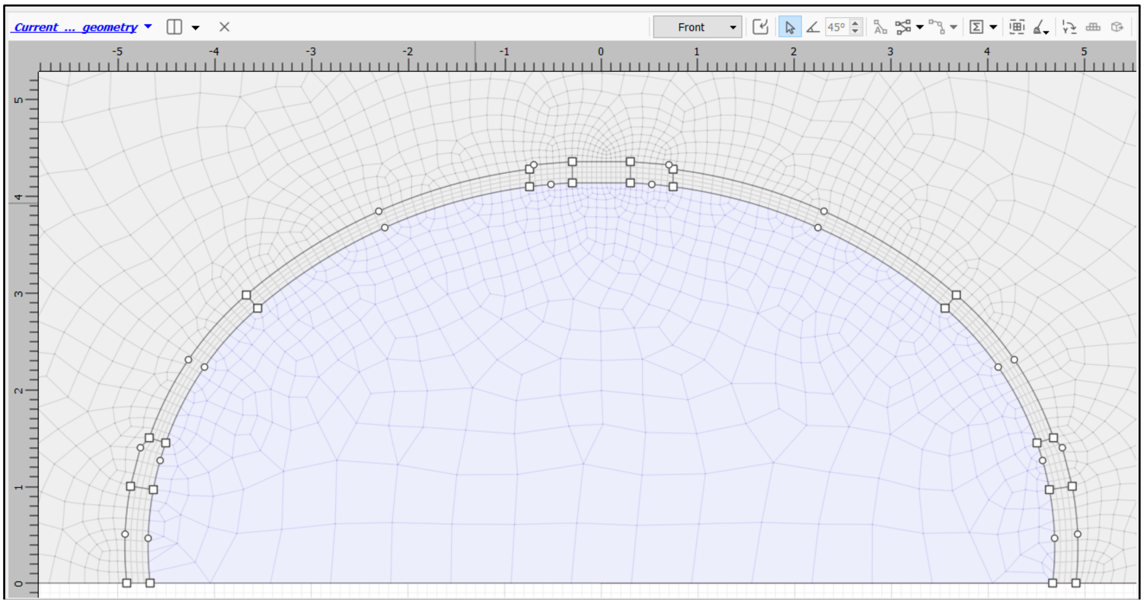


Figure 5.8-1 – FLAC model: geometry of the extrusion

Groups have been defined in order to distinguish the different elements of the models: rock, lining, void; and also to identify the concrete elements of the lining structure: key, segments, sidewalls.

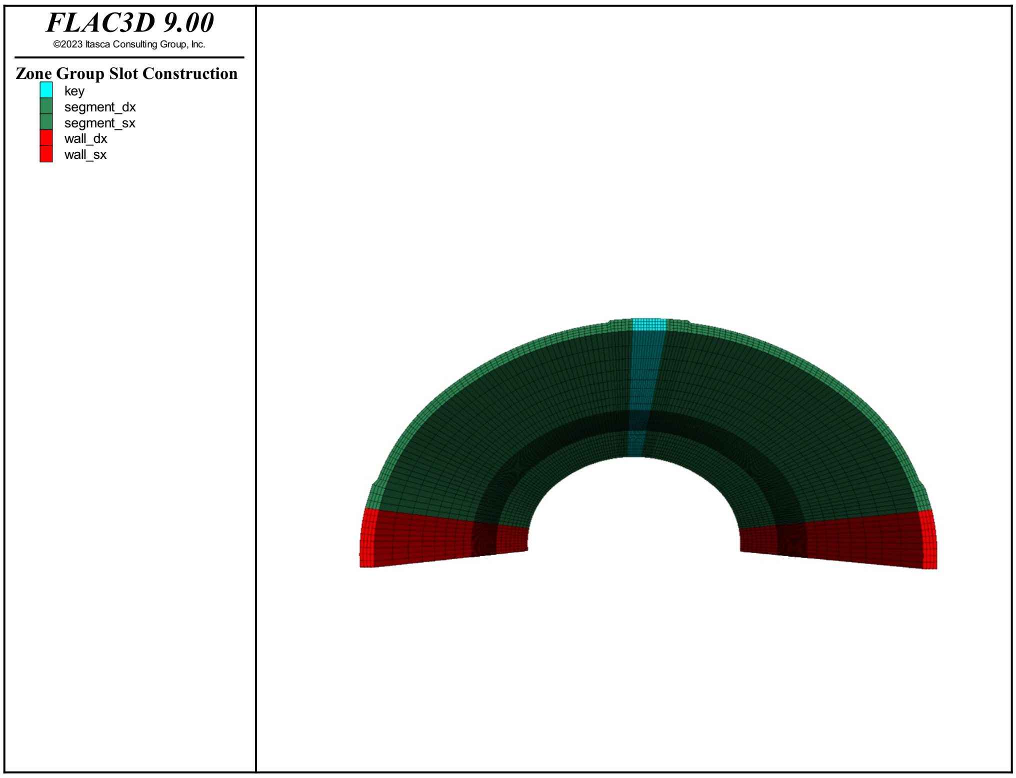



Figure 5.8-2 - FLAC model: groups identifying the structural elements

01_initial

Boundary conditions were defined on each side of the model to simulate the presence of confining rock at the boundaries of the model. These conditions were applied by fixing the velocity along the perpendicular direction respect to the boundary surface.

To initialize the state of stress of the rockmass, a constitutive model has been assigned. According to the geological data, the Hoek and Brown constitutive model was adopted and, to be conservative, the equation parameters were evaluated according to the rockmass of Class IV, which represent the most critical conditions.

$$\sigma_1 = \sigma_3 + \sigma_{ci}\left(m_b * \frac{\sigma_3}{\sigma_{ci}} + s\right)^a$$

*Table 5.8-1 – FLAC model: rockmass properties*

| | | |
|---|---|---|
| $m_b$ | 2,2 | - |
| $s$ | 0 | - |
| $a$ | 0,51 | - |
| $\sigma_{ci}$ | 23 | $MPa$ |
| $density$ | 27 | $kN/m^3$ |

Once properties and boundary conditions are defined it has been possible to proceed with the initialization of the state of stress.

02_excavation

Even if the excavation phase it is not present in the real project, since it is not a new tunnel, it is important to take into account the total release of the rockmass. Respect to a new tunnel, the final lining is installed after a long time from the excavation. For this reason, the rockmass surrounding the tunnel has already undergone the stress relief. To simulate these conditions inside the software, it is possible to “excavate” the tunnel zone and solve the model up to reach the equilibrium of the forces.

03_lining_activation

At this point the new final lining can be activated. The concrete elements are modeled as an elastic material by assigning the elastic constitutive model with the following properties.

*Table 5.8-2 - FLAC model: concrete properties*

| | | |
|---|---|---|
| Bulk Modulus | 20 | GPa |
| Shear Modulus | 7 | GPa |
| Density | 2400 | $Kg/m^3$ |

Since the final lining structure is not made of a single concrete element, it is important to understand what append at the interface between cast in place concrete and the segments. To do this it was necessary to insert interfaces and define the properties of the various connections (axial and shear stiffness, friction angle and cohesion). Another important aspect to be considered is the connection between lining and surrounding rock, also in this case it was modeled by means of an interface element considering a shar stiffness = 1/10 respect to the axial stiffness due to the presence of the impermeabilization layer that generate a “smooth” surface.

04_load

The final phase was to consider the application of a localized load acting on the lining. Although the rockmass has already released tensions, it can still produce wedges detachments. According to the UNWEDGE analysis, the heaviest wedge has been considered. The simulation of this load was done by inserting an applied stress on the final lining of 100 kPa acting on a portion of tunnel lining. In order to consider the most unfavorable condition, the load was located just at the interface between the segments and the key and the stresses at the interface were evaluated. However, this is a very unfavorable case that can hardly occur.

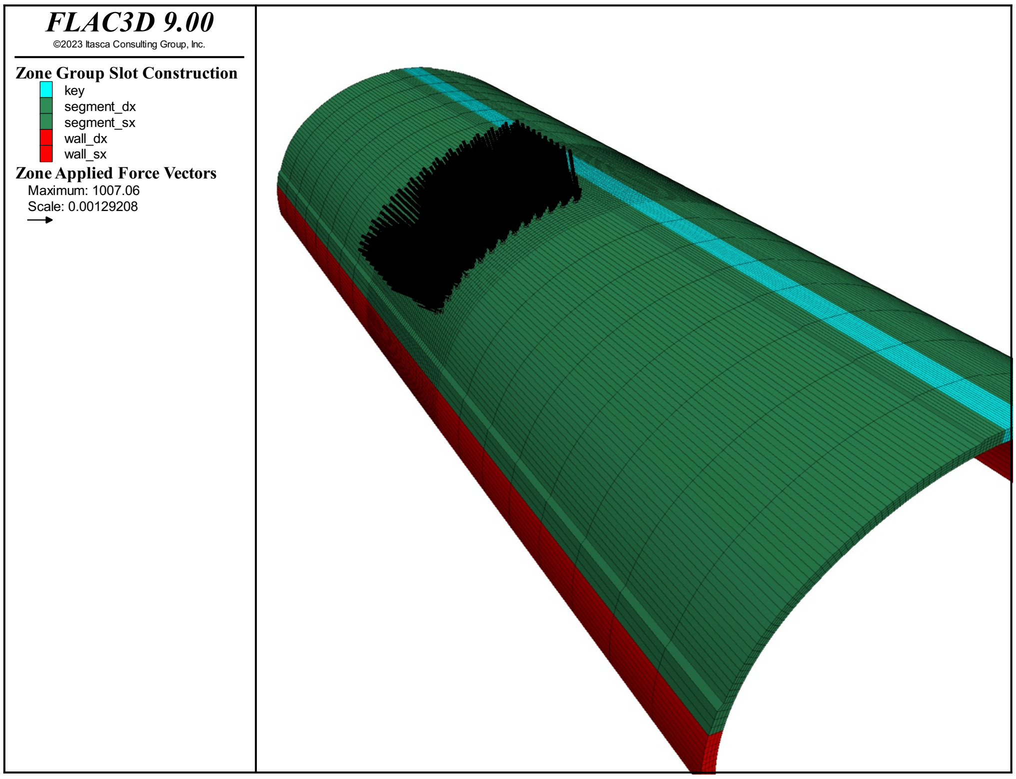



*Figure 5.8-3 – FLAC model: localized load acting on the lining*

Results pre-load.

In order to analyze the horizontal stresses acting at the interface between the key and the segment, two strategic points have been studied. These points are located respectively on the extrados and on the intrados, in correspondence of the connection, and well distant from the boundary conditions. The graph represents the state of stress in relation to the number of iterations. As it can be seen from the graph, the calculation process reaches a convergence after about 3000 iterations. The values found are 14 kPa in traction for the point on the lower surface (cyan) and 4.5 kPa in compression for the point on the extrados (red). Even if there is a tension, it is only a temporary load that will be compensate thanks to the injection of the micro-cement on the extrados. Moreover, the value of the tension is low compared to the tensile strength of the reinforced concrete. Before the application of the localized load, the vertical displacement on the key is very close to zero and the maximum value is less than 1 mm.

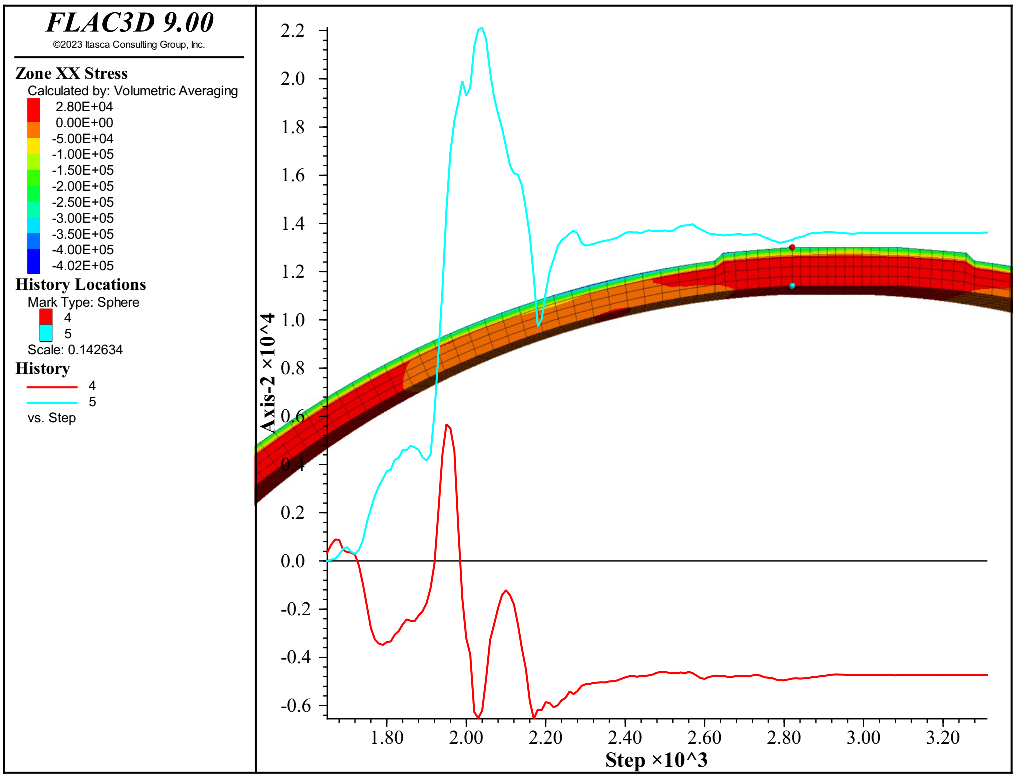



*Figure 5.8-4 – FLAC model: stress-xx pre-load*

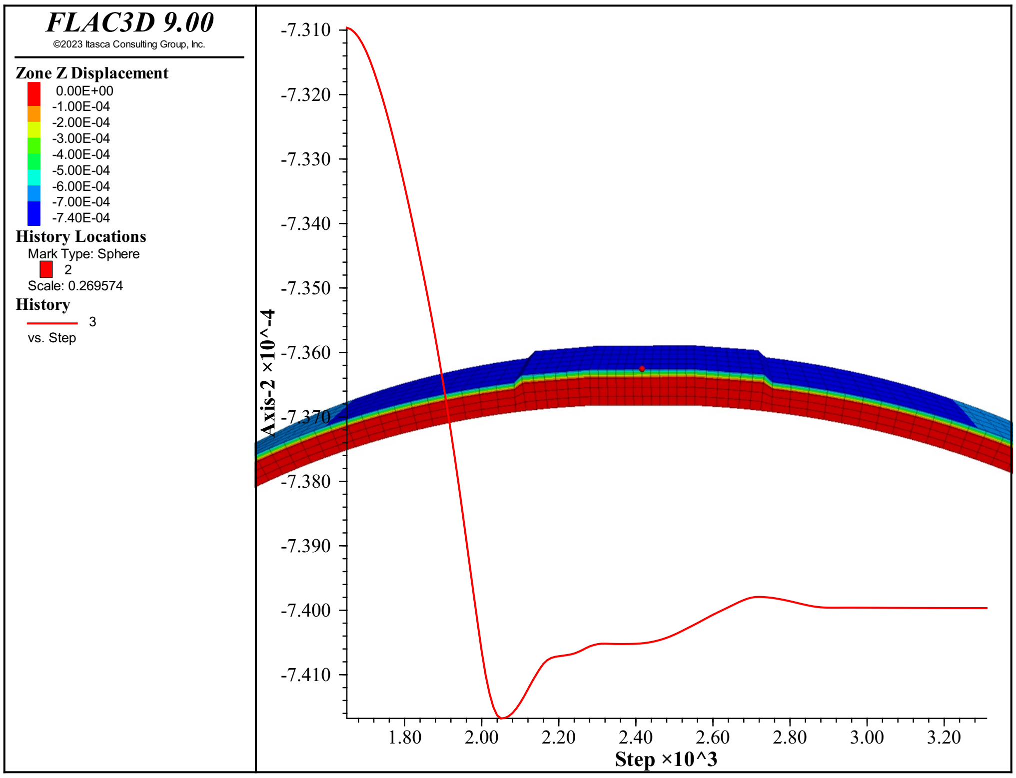



*Figure 5.8-5 – FLAC model: displacement-zz pre-load*

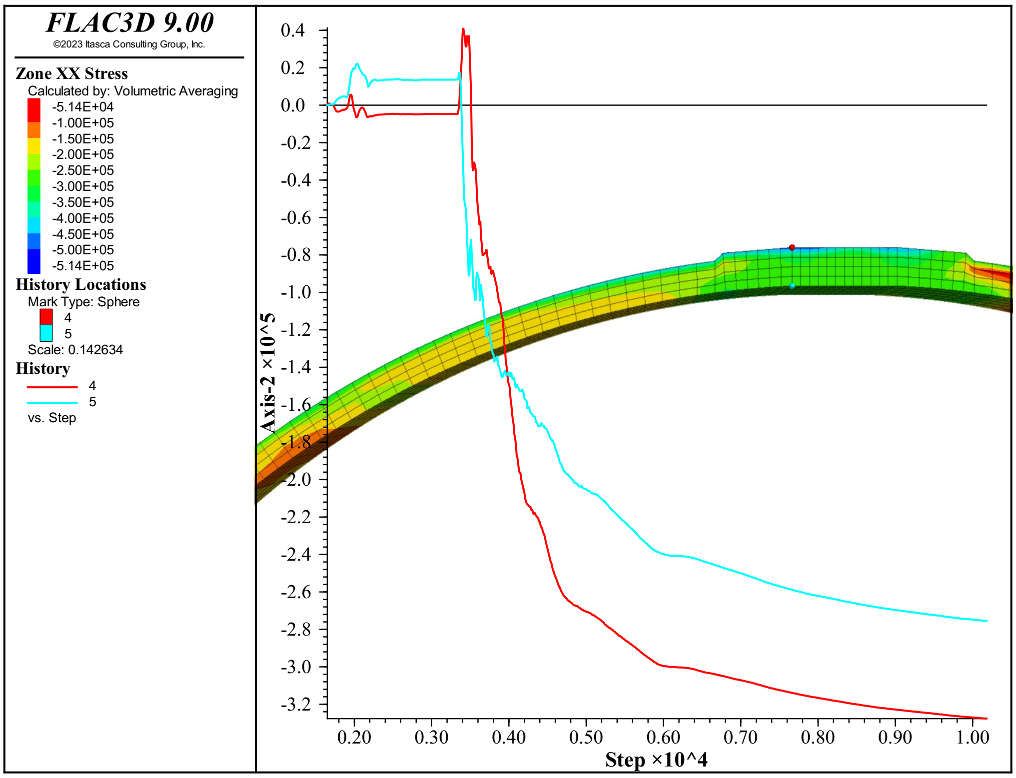



*Figure 5.8-6 – FLAC model: stress-xx post-load*

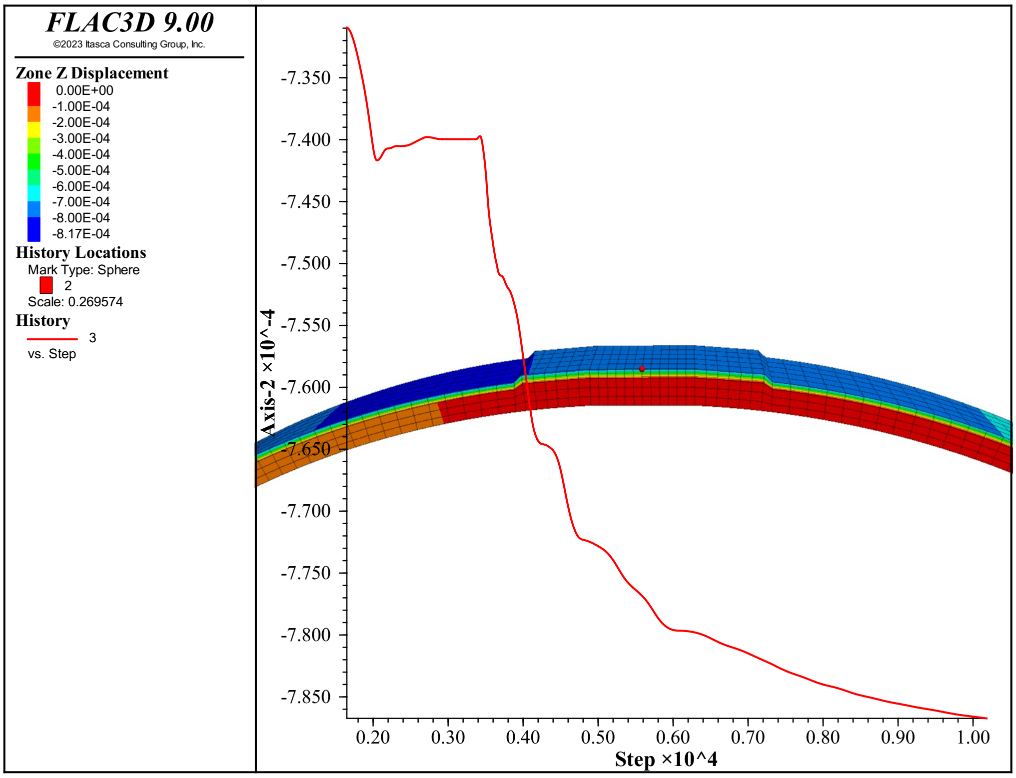



*Figure 5.8-7 – FLAC model: displacement-zz post-load*

Results post-load.
After the application of a localized load of 100kPa, the state of stress changes significantly and the structure is completely in compression. In this case, the model reaches the convergence after about 10.000 steps and the final values are 275 kPa for the point on the intrados (cyan) and 325 kPa for the point on the extrados (red).

By looking at the displacements, also in this case the value are less than 1 mm. The maximum value is 0.8 mm and no dislocation occurs in correspondence of the connections between the structural elements.

In conclusion, considering the results and the state of stress obtained, it is possible to say that the numerical model made with FLAC 3D is in agreement with the analyses previously carried out with the other software. In addition, the obtained stress values in correspondence of the interfaces are consistent with the resistance of the inserts used for the connection between the segments and the key.

# 6 Conclusions

The development of this thesis gave me the possibility to identify and analyze some case studies concerning tunnel re-lining. Thanks to the research and data provided by the company Pini Group, it has been possible to make a comparison between the different methods adopted for each intervention considered. Although the applicative characteristics are quite different from one case to another, some common points have also been highlighted, which have allowed me to recognize pros and cons of each method. The table below summarizes the data from the case studies analyzed. Unfortunately, some cells are empty due to the impossibility of finding the respective data for the lack of information in the reference bibliography or because the intervention in question has not been realized yet.

*Table 5.8-1 – Resume of the main differences among the analyzed cases*

| Tunnel | Material | Structure | Dimension of the re-lining | Waterproofing system | Impact on the traffic | Time for re-lining | Costs |
|---|---|---|---|---|---|---|---|
| FEDRO | Rock | Cast in place | 52 m² x 196 m | 3-layer sheeting | Closed at night | 5 m/night | - |
| Monte Galletto | Clay | Cast in place sidewalls + 2 precast predalles | 49 m² x 8 m | 3-layer sheeting | Closed | 15 d* | 5,3M €* |
| Glatschera | Rock | 5 precast segments | 20 m² x 334 m | Gaskets on the segments | Closed | - | - |
| Nazzano | Clay | 15 precast segments | 160 m² x 337 m | 3-layer sheeting | Open | 0.9 m/d* | 57,37m €/m* |
| Melide-Grancia | Rock | Existing sidewalls + 2 precast predalles | 52 m² x 2070 m | 3-layer sheeting | Closed at night | 10 m/night | 50M CHF.* |
| Project in progress | Rock | Cast in place sidewalls + 2 precast segments | 44 m² x 537 m | 3-layer sheeting | Closed | - | - |

*For all the intervention (not only re-lining)

As we have seen from the cases reported, the methods to carry out the re-lining are many and each case study has innovative characteristics which permit a better adaptation of the intervention to specific context. The applicative example of the project under development from the company Pini Group is an unconventional project that, in many aspects, represents an engineering and structural innovation.
During my permanence in the company, I had the opportunity to follow the designing process of this innovative re-lining project and actively contribute to the realization of static verifications and numerical models. Despite the encountered challenges in the realization of the numerical models, it was a formative experience which allowed me not only to improve my technical knowledge, but also to increase my teamwork skills and problem solving.

# 7 Bibliography

*EOTA, "Technical Report: Design of anchor channels", TR047, September 2015, amended September 2017, amended March 2018*

*FIB – International federation for Structural Concrete, "Design of anchorages in concrete", July 2011*

*CIRIA, "Tunnels: inspection assessment and maintenance", C671, 2009*

*BSi British Standards, "Design of fastenings for use in concrete - Part 4-2: Headed Fasteners", DD CEN/TS 1992-4-2:2009*

*M. Nilsson, U. Ohlsson, L. Elfgren "Effects of Surface Reinforcement on Bearing Capacity of Concrete with Anchor Bolts"*

*G. Balbi, "FEDRO's tunnel renovation method – Renovation of a non-reinforced primary lining using night shifts while keeping all traffic lanes open during peak traffic periods", 2020*

*Strade e Autostrade, E. Crippa, A. Damiani, "Monte Galletto: il caso della galleria A7. Gli interventi di sistemazione del rivestimento definitive delle gallerie della rete ASPI: la descrizione delle principali soluzioni e della loro messa in opera", December 2022*

*Rhomberg Sersa, "Tunnel renovation for the Rhaetian Railway – The Rhomberg Bahntechnik Method.", January 2020*

*G. Lunardi, A. Belfiore, A. Selleri, R. Trapasso, "Widening the Montedomini tunnel in the presence of traffic: the evolution of the Nazzano method.", 2014*

*ITASCA sofware, "FLAC 3D version 9.0. Continuum Modeling for Geomechanics", 2023.*

*IStructE / Concrete Society, "Standard method of detailing structural concrete. A manual for best practice.", June 2006.*

*PIARC, "Road tunnels manual", October 2019*