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Corresponding Author: Dr. Francisco Javier Torrijo, Ph.D.

Corresponding Author's Institution: Universidad Politécnica de Valencia

First Author: Santiago Alija, Ph.D.

Order of Authors: Santiago Alija, Ph.D.; Francisco Javier Torrijo, Ph.D.; Mário Quinta Ferreira, Ph.D.

Abstract: A representative example of the problems associated with the excavation and support of tunnels in karst ground is presented. It is a peculiar case in terms of heterogeneity and spatial distribution of zones of poor geotechnical quality, requiring the need to define, preferably in the study phases, adequate site investigation, suitable design procedures, efficient construction techniques and appropriate ground treatment. The difficulties associated with the instability of the karstified ground, and the presence of cavities, wholly or partially filled with soils of low cohesion, are discussed via retrospective analysis. The solutions adopted to solve the problems encountered during the tunnel construction, enabled a systematic approach, useful for new construction projects in limestone terrains of medium to high karstification.

Cover letter

Dear Sir,

I would send them a research work on problems that occurred in a túnel in Spain, and we had the opportunity to study at the University of Valencia (Spain).

This tunnel is in the high-speed Spanish train and study their construction problems during the development of the doctoral thesis of one of the authors (Santiago Alija), in conjunction with the University of Coimbra (Portugal). This thesis received the highest rating by the court: Cum Laude.

The baseline data were obtained during construction of the tunnel andused to identifythe problem and to adopt a technical solution to it.

These Solutions are currently being implemented in other tunnels in the high speed line.

We therefore believe that the problems identified and analyzed seem interesting enough to dare to send them to your journal. We hope it is of interest and can be published in the same.

Thanking you in advance for your attention, receive a greeting:

DEPARTAMENTO
DE
INGENIERIA DEL TERRENO

Dr. Francisco Javier Torrijo

Department of Geotechnical Engineering, School of Civil Engineering

Polytechnic University of Valencia, Camino de Vera, s/n, 46022 (Valencia, Spain)

Phone: +34963877582; Fax: +34963877589

E-mail:fratorec@trr.upv.es

***Revision Notes**
Click here to download Revision Notes: ENGEO4591R1, Editors decision, revise.txt

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Para: fratorec@trr.upv.es

Dear Dr. Torrijo,

I can now inform you that the reviewers and editor have evaluated the manuscript "Engineering geological problems associated to the tunnel construction in the karstified rock masses: The case of Gavarres tunnel (Spain)" (Dr. Francisco Javier Torrijo). I am pleased to say that it has been favourably received and publication with minor revision is recommended (see below and on http://ees.elsevier.com/engeo/).

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I hope that you will find the comments to be of use to you and am looking forward with interest to receiving your revision.

Thank you for submitting your work to this journal.

With kind regards,

Harinath Subramaniam
Journal Manager
Engineering Geology
....
Important note: If a reviewer has provided a review or other materials as attachments, those items will not be in included in this letter. Please ensure therefore that you log on to the journal site and check if any attachments have been provided.

COMMENTS FROM EDITORS AND REVIEWERS

Reviewer #1: This contribution has been substantially revised in the ways suggested by the referees and is now acceptable for publication in Engineering Geology in my opinion. It presents, first, a carefully documented account of the sequence of problems that were encountered during the excavation of the Gavarres railway tunnel that is supported with useful tables of measured data from both field and lab and a summary event table, 5, that is particularly helpful; second, a solid review of the causes of the difficulties encountered (Part 7) and practical suggestions for avoiding or overcoming them (Part 8). The terrain was clearly a very difficult one for adequate exploration by boreholes and surface/downhole geophysics prior to excavation, of a type often found in the vadose zone in well developed karst terrains (such as southern China, where TGV routes are being constructed quickly today) - so its publication may assist many other engineering projects around the world. The written English remains awkward and difficult to follow at times, although it has been significantly improved. The Minor Revision that I recommend will be a further re-writing by someone who is fluent in English and also knows the correct terms for engineering protective measures such as piling, etc. I would be happy to take such a corrected draft and give it a final run-through to apply the commonly used terms of karst specialists where necessary.
In Figure 2 I recommend that the authors indicate in the tunnel sketch, the boundaries of the zones that they specify in Table 5.

EDITOR'S DECISION:

the editor whish to thank the authors for their efforts at improving the manuscript. Nevertheless, some revisions are still requested mainly relatively to the language use. Please ask a native english person r colleague or a professional service to help you at finalizing the text questions and learn more about EES via interactive tutorials. You will also find our 24/7 support contact details should you need any further assistance from one of our customer support representatives.

TITLE

Geological ~~Engineering~~ engineering ~~geological~~ problems associated to ~~the~~ tunnel construction in ~~the~~ karst~~ified~~ rock masses: The case of Gavarres tunnel (Spain)

AUTORS

S. Alija[a], F.J. Torrijo[b], M. Quinta-Ferreira[c]

[a]~~Department of Earth Sciences,~~ Geosciences Center, University of Coimbra, 3000-272 Coimbra, Portugal. Tel. +351 239860500. E-mail: santiagoalija@gmail.com

[b]Department of Earth Engineering, Universidad Politécnica de Valencia, 46022 Valencia, Spain. Tel. +34 963877582. E-mail: fratorec@trr.upv.es

[c]Department of Earth Sciences,~~-~~ Geosciences Center, University of Coimbra, 3000-272 Coimbra, Portugal. Tel. +351 239860500. E-mail: mqf@dct.uc.pt

ABSTRACT

~~This article presents a~~ A representative example of the problems associated with the excavation and support of tunnels in karst~~ified~~ ground is presented. It is a peculiar case in terms of heterogeneity and spatial distribution of zones of poor geotechnical quality ~~zones~~, requiring the ~~creating the~~ need to define, preferably in the study phases, adequate site investigation, suitable design procedures, ~~and~~ efficient construction techniques and appropriate ground treatment ~~techniques~~. The difficulties associated with the instability of the karstified ground, ~~which is the product of karstification~~ and the ~~appearance~~ presence of cavities, wholly or partially filled with soils of ~~reduced~~ low cohesion, are discussed via retrospective analysis. The ~~definition of the~~ solutions adopted to solve the problems encountered during the tunnel construction, enabled a systematic approach, useful for new construction projects in limestone terrains of medium to high karstification.

KEYWORDS

Tunneling in karst, karstified rock masses, retrospective analysis, Gavarres tunnel, NATM.

1. INTRODUCTION

The design and construction of tunnels in karst ~~areas~~ terrains is fraught with ~~associated~~ problems associated with the unexpected location, ~~to its~~ irregular geometry and unpredictable dimensions of the ~~hollow~~ karst structures.

In a karstified terrain ~~area~~, prospection and regular testing campaigns should be supplemented with other techniques adapted to locate and anticipate the ~~geotechnical~~ problematic zones. It must be taken into account that no site investigation technique is one hundred percent accurate, and therefore several techniques should be used, adapted to each specific situation, taking into consideration the budget for the work and the risks that can be assumed in the project.

A real case of a tunnel constructed in a karstified limestone ground is presented, ~~discussing the~~ the ~~past~~ problems encountered are described and the proposed solutions are discussed. A systematic approach, as a knowledge tool for future work in similar situations, is presented.

2. GEOLOGICAL FRAMEWORK

From the geological point of view, the study area is located in the Les Gavarres region, which is included within the Catalan Transverse System, directly related ~~to~~ with the Neogene ~~Depression~~ depression of the Empordà (Agustí et al., 1994).

Les Gavarres region consists of a fringe of Paleogene materials (mainly Eocene), arranged around a Hercynian rock massif, ~~that~~ outcrop~~s~~ing at south of the study area. The age ~~origin~~ of these materials is prior to the Alpine Orogeny, as they have suffered deformation and fracturing during this tectonic phase. The series is dislocated in blocks, separated by ~~fractures,~~ fractures that lead to the uplifting of the massif. The general structure is a monocline arrangement, dipping mainly to Northeast (IGME, 1983, 1995). The ~~G~~geological ~~division~~ formations affecting the tunnel are~~is~~ (Fig. 1):

- ~~1~~ *Barcons Sandstone Formation (E_A ~~Formation~~)*. ~~Its~~It is ~~are~~ composed by glauconitic sandstone~~s~~, medium to coarse grained, locally conglomeratic. The predominant colour is grey~~-~~-yellowish or ochre. The grains are mainly of quartz and feldspar with a scarce ~~minimal~~ clay matrix. It has calcareous cement and ~~frequent~~ abundant bioclasts. At the base and top of the series, the layers are decimetric to metric, presenting a more massive appearance in the middle of the formation. The average sedimentation corresponds to a deposit in the frontal area of the delta, which is rather thick, but of limited extent. The age of the series is Eocene.

- ~~2~~ *Banyolas ~~Loam~~ Limestone Formation (E_M ~~Formation~~)*. This formation is composed of layers of limestone and ~~loam~~ marl, whose relative proportion varies throughout the series. They are of grey and bluish grey colours, and ~~T~~the layers have decimetric thickness~~es~~. The carbonate content ranges from ~~slightly loamy~~ marly clay~~s~~ to ~~marl~~ limestone, affecting the materials strength, ~~alterability~~ weatherability and the stability behaviour of the ~~groundmass~~rockmass~~, according to the span of the series~~. Some spans of the series are mainly composed of hard clay and ~~loam~~marls. ~~They are of grey and bluish grey colours.~~ The age of the series is Eocene.

- It is important to note that the Banyolas Limestone Formation is in concordance ~~consistent~~ with the underlying ~~formation of~~ Girona Fossiliferous Limestone Formation~~s~~.

- ~~3~~ *Girona Fossiliferous Limestone Formation (E_C ~~Formation~~)*. ~~This~~ It is a fossiliferous limestone, presenting oolitic terms at the base. The predominant colour is ochre. It is rather recrystallized and arranged in layers of ~~varying~~ a wide range of thickness, from decimetric to metric. The environment of sedimentation corresponds to proximal marine environments of carbonate platform. The age of the series is Eocene.

- ~~4~~ *Pontils Group Conglomerates (E_{CG} ~~Formation~~)*. This ~~is a~~ formation is constituted by ~~of~~ conglomerates and red sandstones with clay layers. These deposits have fluvial origin. The age of the series is Lower Eocene, but may also include part of the Palaeocene. ~~These deposits are fluvial origin.~~

The boundary between the Les Gavarres region and the SW margin of the Ampurdán depression, is marked by a fracture alignment oriented NW~~-~~-SE, called Banyolas Fault or

Camós~~-~~–~~-~~Celrá. This alignment is part of a system of fractures orientated predominantly NW~~-~~–SE. They are normal faults~~, which are~~ related to quaternary volcanism and ~~current~~recent seismicity. This important regional fault intersects the line of the tunnel, corresponding to intense fracturing of the rock material.

3. GEOTECHNICAL CHARACTERISTICS

According to the geological cross section defined in the ~~construction project~~design, most of the tunnel would be excavated in the materials of the Banyolas ~~Loam~~ Marl Formation (Fig. 1) while the northern part is affected by the fault system associated with the Banyolas Fault or Camós~~-~~–~~-~~Celrá.

[Figure 1]

Fig. 1. Location map and geology ~~geological~~ profile ~~of~~ along the Gavarres tunnel ~~alignment~~.

The two fundamental geotechnical units are described below. ~~are defined and t~~The results of the laboratory tests, from samples collected ~~from~~ in the tunnel boreholes, are shown in the tables 1 and 2:

- *Limestone and ~~Loam~~ Marl Geotechnical Unit*. This unit is entirely constituted by calcareous rocks of the Banyolas ~~Loam~~ Limestone Formation (E_M). The rock samples tested generally present medium to low strength, with ~~the~~ a weathering grade, in the vicinity of the tunnel, ranging from III to V (according to ISRM, 1981). The seismic profiles carried out in the tunnel confirmed this data. The ~~ground~~water table ~~levels~~ detected in the ~~probe~~ boreholes~~bores~~ ~~were~~ was located below the ~~ground~~ invert~~level~~ of the tunnel ~~(average ground level height of tunnel is located 93,5 m above sea level)~~.

- The average densities (Table 1) and ~~simple~~ uniaxial compressive strength (Table 2) gave very scattered values, depending on the degree of ~~alteration~~ weathering of the sample (Barton et al., 1974).

- During the ~~geotechnical exploration~~ site investigation programme, permeability tests revealed ~~a~~ medium~~-~~–~~-~~low permeability terrains (González de Vallejo et al., 2002), around 1 x 10^{-7} m/s ~~were conducted (González de Vallejo et al., 2002)~~.

- ~~The uniaxial compressive strength is highly variable, depending also on weathering degree (Table 2).~~

Considering the RQD values obtained in the borehole~~s~~ samples and the uniaxial compressive strength, a representative~~n~~ RMR value of 30 ~~representative of the unit~~ was estimated (~~C~~class IV or Bad, Bieniawski, 1989).

- *Fault Zone Geotechnical Unit (E_M very fractured)*. ~~This~~ It is a highly fractured zone, where argillite, calcareous mylonite and ~~loam~~ marl have been identified~~recognised~~. The rock weathering ranges from grade II to V (according to ISRM, 1981). Water levels were found at different heights, associated with fracture planes.

Although most of the unit consists of highly fractured limestone and ~~loam~~marl, from the Banyolas Limestone Formation, the presence of a small thickness of Girona *Fossiliferous Limestone* (E_C) ~~limestone has~~was also ~~been~~ observed in ~~probe bores~~boreholes ~~(E_C)~~ as well as conglomerates and red sandstones of the Pontils Group (E_{CG})~~, had also been observed~~. Both formations present weathering grades of IV-V (according to ISRM, 1981). The permeability tests showed low to medium ~~permeability~~values, similar to those usually presented by fractured rock masses of limestone and dolomite (1×10^{-6} m/s).

- Mainly ~~According~~ based ~~to~~on the RQD values of the rock cores and ~~to~~ on the uniaxial compressive strength values, an RMR value of 20 was estimated (~~C~~class V or Very ~~poor~~Poor, Bieniawski, 1989).

[Table 1]

Table 1. Sieve analysis and consistency limits (Atterberg limits) ~~result~~ of the soil materials of the Gavarres tunnel.

[Table 2]

Table 2. Strength parameters of the Gavarres tunnel obtained over rock cores tested in the laboratory.

4. CONSTRUCTION PROJECT

The tunnel is part of the Madrid–French border high-speed railway line, and is located within the province of Girona (Fig. 1). It is a double track tunnel having a total length of 758 m with a maximum overburden of 31 m. ~~(~~The average ~~ground level height~~altitude of the tunnel is ~~located~~ 93,5 m above sea level~~)~~.

The free section of the tunnel, defined in terms of health and comfort criteria, was 110 m^2. The geometric characteristics of the tunnel cross section ~~of the tunnel~~ were designed ~~with~~ using a circular ~~dome~~ vault ~~that~~ extending~~s~~ into the floor, without differentiating the gables (López, 1996).

Having in mind the characteristics of the in situ materials and the dimensions of the tunnel, it was considered that the mechanical excavation was the most suitable procedure and that blasting could be used in the ~~rocky~~ unweathered limestone zones (Díaz, 1997).

The ~~project~~ design recommended the use of the New Austrian Tunnelling Method (NATM), since it could allow pre-support ~~for the heading~~during tunnel advance, through mechanical pre-cutting.

The excavation phases used in the tunnel were: one excavation phase in full section in top heading, two excavation ~~digging~~ sub-phases in the bench and one excavation phase in inverted vault.

In the design ~~for~~ of the tunnel support ~~for the tunnels~~, three ~~types of~~ section~~s~~ types ~~have~~ were~~been~~ ~~identified~~ defined (Fig. 1), ranging from the better quality terrains to the weakest (Hoek and Brown, 1980, Hoek et al., 1995):

- S-II: ~~T~~this~~e~~ ~~type S-II~~ section type applies to the weathered calcareous ~~loam~~ rocks of the Banyolas ~~l~~Limestone ~~and loam unit~~Formation. The excavation should be performed in ~~passes~~ advances of 1.0 m in top heading, with a primary support based on a 5 cm sealing of shotcrete with steel fibre, light ~~trusses~~steel ribs type TH-29 and shotcrete with steel fibre, 25 cm thick in total (excluding the 5 cm of sealing). The two sub-phases of the bench were implemented in 2.0 m spans extending the support of the top heading.

- S-III: ~~The section f~~was used for the fault zone unit ~~was the section named type S-III~~. In this section type, the excavation would be done in ~~passes~~ advances ~~from~~ of 0.5 to 1.0 m with the~~a~~ support based on a 5 cm sealing of shotcrete with steel fibre, heavy ~~trusses~~steel ribs of type HEB-160 and shotcrete with steel fibre, 30 cm total thickness (excluding the 5 cm of sealing). The drilling of the bench would be done in two sub-phases, with ~~passes~~ advances from 1.0 to 2.0 m extending the support ~~foreseen in~~of the top heading.

- S-E: ~~The~~ was the section type for the tunnel ~~outlets~~ portals. It~~(S-E)~~ was characterised as type "heavy" as these ~~areas~~ zones were expected to be more weathered, ~~and~~ decompressed~~,~~ due to the previous ~~work of~~ excavation of the ~~entrance~~ portal slopes and presenting ~~and the~~a rather ~~tight~~ thin ~~lining~~ overburden ~~of~~ above the tunnel. The ~~proposed~~ S-E section consisted of a heavy micropile umbrella, 20 m long and 150 mm in drilling diameter, spaced 0.5 m between axes and fitted with steel pipes, ~~of~~ 110 mm of external diameter and 8 mm thick, ~~and~~ filled with mortar. The excavation and support sequence ~~and the support~~ for this section would be similar to ~~that of the~~ S-III, with the difference that the ~~trusses~~steel ribs ~~positioned~~ used below the umbrella would be type HEB-180.

All sections should have a ~~shotcrete~~concrete ~~,~~inverted vault~~ed~~ ~~and~~ with welded wire mesh~~,~~ 150 x 150 x 6 mm.

A summary table with the support structures defined for the tunnel is presented in **Table 3**.

[Table 3]

Table 3. Summary of the ~~planned~~ support structures proposed ~~of~~ for the three section types of the Gavarres Tunnel.

5. ~~ENFORCEMENT~~ CONSTRUCTION OF THE TUNNEL

The ~~enforcement~~ construction of the Gavarres tunnel began by its south portal in ~~calcareous~~ limestone materials ~~by its south entrance~~ (Fig. 1). First, the excavation and support of the ~~entrance~~ portal slopes ~~were~~ was carried out ~~at the outlet~~. The excavation ~~used~~ was done using mechanical heavy duty rotating machines. During ~~these~~ this early stage~~s~~ of excavation, the heterogeneity of the ~~calcareous~~ limestone rock mass was detected. The ~~excavation~~ working face presented very weathered areas, easy to excavate, alternating with ~~balls of~~ limestone, very difficult to break mechanically.

Once at the tunnel crown level, a micropile umbrella~~s~~ was carried out, for the S-E section type (35 micropiles ~~units~~ in total). During the ~~enforcement~~implementation of these micropiles, the heterogeneity of the ~~site~~ ground continued to be revealed, since the ~~enforcement~~implementation speed ranged from 1 to 4 micropiles per day. The drilling residues changed drastically from limestone fragments ~~of limestone~~ to a clay-like material.

According to the geotechnical characteristics of the ground during excavation and support, mainly associated with the karstification processes, different ~~truss~~ zones were considered along the tunnel (Alija, 2010):

- *~~Outlet~~ Portal Zone ~~1 – Top Heading~~ (~~Truss~~sections 0 – 22)*. The excavation of the tunnel started with mechanical equipment, reaching an average ~~heading~~ progress speed of 4.7 m/day. In this ~~area~~zone, four sections of convergence were installed and eight engineering ~~geological~~ geology ~~time sheets~~ front map~~for the front~~s were ~~raised~~prepared.

- The ground materials were characterised as blocks of ~~loamy and~~ limestone and marl ~~blocks~~, sometimes broken, embedded in a clay matrix. The ~~calcareous layers showed the~~ stratification was:

    - *S_0*: oriented between 200/15 – 200/30 (dip direction/dip angle), with some continuity and some roughness. Between layers, openings ~~from~~ of 5 to 10 mm ~~are~~ were observed~~seen~~, filled with clay ~~and~~ or even calcite ~~were observed~~.

  ~~-~~Two families of joints were identified (Fig. 2):

    - *J_1*: with an average orientation of 213/71, spaced about 30 cm, with some continuity and, when filled, it is with clay material.

    - *J_2*: with an average orientation of 124/70, ~~with~~ spac~~ing~~ed ~~between~~ from 20 ~~and~~ to 60 cm, very rough and usually closed.

  These two families of joints and the stratification maintained their orientation all along the tunnel, but due to the heterogeneity of the rock mass, they were not found or distinguished on all of the fronts ~~studied~~mapped.

According to the front reports, the average RMR value obtained for this area was 36, corresponding to a rock mass of class IV (poor grade).

~~**[Figure 2]**~~

~~Fig. 2. Tunnel-working faces view of pass 19 with details of the stratification (S_0) and joints (J_1, J_2). Loamy and limestone blocks, some broken, which were embebed in a clay matrix .~~

~~According to the front tabs, the average RMR value obtained for this area was 36, which corresponding to a rock mass of class IV (poor grade).~~

During the ~~execution of the~~ excavation and support ~~tasks~~operations, small falls of rock and clay ~~falls~~ occurred. In ~~passes~~ section 2 and 3, the instabilities in the roof of the tunnel achieved 12 m^3. ~~Instabilities~~ Detachments were also produced in the right~~-hand area~~ side of the roof and gable, of ~~passes~~ section 13 and 14, ~~of the order of~~achieving 15 m^3. Instabilities also occurred in the gable and right shoulder in ~~passes~~ section 20 to 21. These detachments ~~landslides~~ showed the presence of small fragments of limestone embedded in a clay matrix. ~~Due to t~~The large volume of fallen ~~material,~~materials ~~it~~ required its~~was necessary to~~ filling with concrete, ~~the cavities with shotcrete,~~ using Bernold ~~sheets~~ plates as ~~permanent~~ lost formwork ~~for these passes~~. Throughout this zone, a portal ~~n outlet type~~ section type (S-E) was implemented ~~(S-E)~~.

[Figure 2]

Fig. 2. Tunnel-working face view of section 19 with details of the stratification (S_0) and joints (J_1, J_2). Marl and limestone blocks, some broken, in a clay matrix.

- *Zone 1 ~~– Top Heading~~ (~~Truss~~sections 22 – 38)*. In this ~~area~~zone, heterogeneous tunnel-working faces appeared, ~~to be~~ composed of very compact ~~loam~~marly~~-~~ limestone layers, and occasionally with clay or even calcite filling the spaces~~,~~ ~~embedded~~ in the clay matrix. In the final metres of the zone ~~W~~wet spots in the clay were often found ~~in the clay~~, and ~~and in the final metres of the zone,~~ some karstification voids were identified (Anguita and Moreno, 1993).

The ~~heading~~ advance speed in this ~~zone~~ zone slowed down to an average of 3.3 m/day, because very specific blasting was required to break up the hardest limestone materials. The rock mass appeared to have higher hardness than in the previous zone. Nine ~~front heading attachments were lifted~~engineering geology front maps were prepared~~, providing an~~. The average RMR value ~~of~~ was 43~~. These characteristics are~~, ~~compatible with~~ allowing a fair to good stability ~~grade~~ of the ~~work~~ front, ~~from medium to good~~ and the advance in passes of 1 m using mechanical excavation.

~~In this stretch, the tunnel-working faces appear to be areas of greater hardness than in the previous area, increasing the execution time for the pass, so that very specific blasting was done to break up the hardest materials.~~

The RMR values obtained in this ~~area~~ section ~~require~~ were compatible with~~an~~ the S-II support type. However, taking into account the heterogeneity of the fronts and the thin overburden, it was decided to use a conservative stance, and the support defined for S-III with passes of 1 m was installed. The accumulated strain denoted a clear evolution towards stabilization, reaching values under 5 mm.

- *Zone 2 ~~– Top Heading~~ (~~Truss~~sections 38 – 91)*. Corresponds to 53 meters with ~~medium~~moderate karstification. Several instabilities occurred, composed of highly fractured ~~loam~~marl-limestone ~~ridges~~ rock fragments embedded in a clay matrix. The weathering grade of this ~~This stretch~~zone ranges from ~~weathering grade from~~ III to IV (Fig. 3).

[Figure 3]

Fig 3. Tunnel-working faces with weathering degree III in Zone 2 (~~Pass~~ section 61). Highly fractured ~~loam~~marl-limestone rock fragments ~~ridges~~ embedded in a clay matrix.

The average ~~heading~~ advance speed of the excavation was slightly reduced ~~back~~ to 3.18 m/day, due to the decrease in the geotechnical characteristics of the ground mass. ~~In addition~~ The ~~hardness~~ strength of the ~~calcareous~~ limestone fragments decrease~~d~~ from an average ~~strength~~ of 44 MPa in ~~Zone~~ Zone 1 to ~~about~~ 34 MPa in ~~zone~~ Zone 2. The materials observed in this ~~area~~ zone ~~are~~ were highly fragmented and the damp spots were a constant ~~in each pass~~. The RMR values ~~obtained in~~ were ~~around~~ in the range 25 - ~~to~~ 35, corresponding to a poor quality rock mass of class IV, in which S-II or S-III support type ~~would~~ could be installed. On site it was decided to maintain the S-III support. The convergence strain was generally lower than 4 mm.

Once passed Zone 1, ~~trusses~~steel ribs HEB-160 and 35 cm of shotcrete HM-35 (S-III) continued to be used for more 15 meters ~~more~~, until ~~pass~~ section 54 when ~~a number of large~~ rockfalls began to occur. This problem required the use of a pre-support system based on a light bolt umbrella (4 m long with an overlap of 2 m) and packed with light beams (Fig. 4).

[Figure 4]

Fig 4. Detail of the support with an umbrella of steel rods 4 m in length and packed with light beams.

Due to the increase in the number and size of the instabilities it was decided to increase the number of the bolts and to raise their length ~~of the bolts~~ from 4 m to 6 m, overlapping 3 m. ~~However, this was not the ultimate solution, since the masses of clay and limestone blocks were able to strip out the bolts as shown in Fig. 5.~~Despite this reinforcement, masses of clay and limestone blocks were still able to strip out the bolts (Fig. 5) requiring the improvement of the support solution.

[Figure 5]

Fig 5. View of an instability with a distorted light umbrella in zone 2 (~~P~~section~~ass~~ 70).

However, this light umbrella was used until ~~pass~~ section 80 where it was decided to place the first self-drilling heavy micropile umbrella, 12 m ~~in length~~long and 90 mm in diameter, with an overlap of 3 meters between umbrellas.

From then on, the support with heavy umbrellas was used systematically. These~~s~~ umbrellas were ~~formed~~made by approximately 35 micropiles, ~~although~~ separated ~~by about~~around 40 cm between their axes. The number of micropiles was dependent on the characteristics of the front at the time of ~~execution~~ excavation, and was decided according working crew experience and to the technical assistance criteria.

- *Zone 3* ~~– *Top Heading*~~ *(~~Truss~~section 91 – 235)*. In this ~~stretch~~zone~~, approximately~~ of 163 m in length, the RMR values slightly increased~~d~~, varying from 30 to 40 (class IV – bad). ~~However, m~~Many karstified rock masses appeared in the tunnel-working faces ~~appeared~~, causing several ~~major~~ rockfalls, greater than those occurred up to this point.

The geology is characterised by the dominance of limestone and clay. The limestone showed well-defined layers in the initial stretch ~~section~~ of the zone~~zone~~, being more bulky and amorphous ~~in~~ towards the final ~~stretch~~, turning difficult to disclose the orientations of S_0, J_1 and J_2. The uniaxial compressive strength ~~has~~ presented an average of 32 MPa. The tunnel-working faces ~~are~~ were dry and seemingly less fractured than in Zone 2. Fourteen convergence sections and one instrumentation cross-section were installed. Twenty engineering geology front maps were prepared. The measured strains showed tendency to stabilize, reaching maximum values under 5 mm in the convergence sections, while the extensometers measured up to 9.5 mm in the key during the advance. The pressure cells measured stresses from 0.05 to 0.1 MPa.

~~The advance speed of the tunnel increased due to the safety provided by the micropile umbrellas, reaching 3.8 m/day.~~

~~Viewing the behaviour of the convergences and monitoring section, it was decided to switch to a lighter support formed by TH-29 trusses (S-II), leaving evidence of their effectiveness in the lower accumulated deformation after the change.~~The evidence of the effectiveness of the micropile umbrellas provided by the lower accumulated deformations, allowed the decision to switch to a lighter support, formed by TH-29 steel ribs (S-II). The advance speed of the tunnel increased up to 3.8 m/day due to the safety provided by the micropile umbrellas.

- *Rockfall Zone ~~– Top Heading~~ (~~Truss~~sections 2~~5~~35 – 463)*. This ~~stretch~~ zone is characterised by a significant decrease ~~in~~of the RMR with values from 25 to 45 (poor quality, class IV), due to the presence of abundant damp spots, with some dripping being observed, and ~~a~~ the decrease ~~in~~ of the rock uniaxial compressive strength ~~of the rock, with an~~ to an average ~~value of~~ 27 MPa.

The ~~heading~~ advance rate raised ~~was~~ slightly ~~increased slightly~~ to 4 m/day, ~~;~~ due to the increased mastery of the working crew ~~on~~ on placing the ~~technique of placement the~~ micropile umbrella~~s~~. In this ~~area~~zone, two convergence sections were ~~placed~~ installed and six engineering geolog~~ical~~y cross sections of the tunnel face were mapped ~~tunnel working faces were raised in heading~~.

The ~~area~~ zone is composed of marly limestone~~-~~ ~~clay~~ materials without a clear arrangement, in which the joints are almost indistinguishable. Unlike the rest of the tunnel, here the damp spots increase, being observed~~with~~ some dripping ~~being observed, with~~and many ~~voids due to~~ karstification voids.

- ~~In this area rockfalls occurred associated with the karst phenomena, even bringing the work to a standstill at the P.K. 501+462, due to sudden, large rockfalls, which forced work to cease for the consideration of new forms of approach.~~ Associated with the karst phenomena in this zone, several rockfalls occurred, even forcing to stop the work at chainage 501+462, due to sudden, large rockfalls, requiring new work procedures.

~~The Table 4~~A ~~presented~~ summar~~ise~~y of the main characteristics of the tunnel zones~~,~~ previously ~~described~~described is presented in Table 4.

[Table 4]

Table 4. Main characteristics of the tunnel zones.

6. THE PROBLEMS

Since its ~~inception~~beginning, the Gavarres tunnel presented a series of geotechnical complexities (rockfalls, detachments, over-excavations, etc.) that slowed down and hindered the excavation. These problems, related to karst phenomena (Ford and Williams, 1989), were not foreseen in the ~~construction project~~design.

The instabilities ~~appeared as instabilities of~~occurred during the excavation or support works, mainly in materials of brecciated aspect, consisting of boulders and blocks of limestone ~~blocks~~ in a soft clay-~~loamy~~ marly matrix, which quickly collapsed or slide ~~in~~ from the front, ~~and~~ shoulders ~~area key~~or crown of the tunnel ~~to the work of excavation and support~~. As the tunnel advanced it became more frequent the presence of cavities, empty or partially filled by decalcification clays.

~~These instabilities, become more frequent as the tunnel progressed, and when in the presence of cavities, empty or partially filled by clays.~~ These cavities (Fig. 6) can also be problematic due to the ~~lack~~absence of support between the tunnel lining and the ground, ~~which can~~eventually caus~~e~~ing problems throughout the life time of the tunnel.

[Figure 6]

Fig. 6. View of a cavity of approximately 20 m^3 affecting ~~passes~~ sections 206 to 210.

Due to the poor geotechnical quality of the terrain, spiles and light micropile umbrellas were implemented but they were unable to stop the ~~successive~~ increase ~~in the size~~ of the instabilities. For this reason ~~a decision was made~~ it was decided to systematically use successive micropile umbrella~~s~~ 12 m long, overlapping 3 m. With this solution ~~still~~ gravitational instabilities still occurred, affecting the material that fell ~~through~~ between the micropiles ~~umbrellas~~.

At dawn, of ~~one~~ a normal work day ~~of work~~ on the Gavarres tunnel, a large instability~~ies~~ hit the ~~key~~ crown and right gable on ~~pass~~ section number 302 at chainage ~~P.K.~~ 501+462 (Fig. 7).

[Figure 7]

Fig. 7. View of the large instability affecting ~~pass~~ section ~~303~~302, ~~that led to the stoppage~~ obliging to stop the ~~of~~ work in the tunnel.

Thanks to the description of ~~the facts by~~ the workers at that time ~~time~~ inside the tunnel, we know that the excavation round was running normally, after excavation the shotcrete sealing was applied and the steel rib was put in place, but ~~the~~ while the shotcrete robot was going into the front to finish the support, ~~there was~~occurred a rustle and a sudden break in the shotcrete sealing, in the key and right ~~bank~~ side-wall zone, followed by the slide ~~into the tunnel,~~ of a large mass of clay and rock fragments into the tunnel. This slide gave sufficient time to workers to ~~withdraw~~ escape without personal injury.

The next day it was found that the instability was constituted of limestone blocks and sharp edges, embedded in clay materials, typical of the decalcification processes with high humidity.

The volume of material introduced into the tunnel was about 200 m^3 and left no visible cavity. The fallen material formed a “stable” cone of loose material, which occupied most of the excavated section, sustained and stopped ~~the detachment of~~ a ~~greater~~larger amount of material, as ~~.~~ it was evident that the cavity above the tunnel was not emptied. The visible consequences were the breaking of a large number of micropiles and the deformation of the last ~~truss~~steel rib ~~attached~~. Once excavated the fallen material, the gap was sealed and the ~~corresponding~~ deformed ~~truss~~steel rib was replaced.

The stability problem appeared to be due to a gravitational collapse on the front and ~~higher~~ crown, of deposits associated with karst phenomena. Later on, several dolines (sinkholes) were identified at the ground surface above the failure. As observed, the ~~In the zone over the key, depressions of circular morphology were found probably to dolines formed by karst sinkholes. These~~ deposits associated with karst phenomena~~materials~~, due to ~~their~~ their low cohesion and strength, ~~cause~~ frequently cause instabilities when traversed by a tunnel (Jianjy and Jian, 1987).

7. CAUSES AND POSSIBLE EXPLANATIONS

After the failure previously described, that obliged to stop~~ping~~ the tunnel works, new geological studies were done, based on the information obtained during the excavation and support of the tunnel. ~~From t~~These studies allowed ~~a~~the reinterpretation of the geology of the area~~ was developed~~, helping to explain the abundance of karst phenomena not previously identified ~~in~~ during the ~~project~~design.

In this new interpretation, it was concluded that ~~most of~~ the tunnel instability occurred ~~length excavated until the instability was~~ in the ~~limestone of~~ Girona Fossiliferous Limestone Formation and not in the ~~loam of~~ Banyolas Limestone Formation. The fossiliferous limestone of the Girona ~~Banyolas~~ Formation is more susceptible to karst phenomena in zones of ~~high~~ intense tectonic fracturing, ~~as~~ like ~~that~~ the one in which the tunnel was being dug.

The ~~root~~ main causes that led to this interpretation error were ~~as~~ the follow~~s~~ing:

- a diffuse contact between the two geological formations (interdigitations);
- abundant vegetation;
- ~~nonexistence~~absence of outcrops.

The ~~origin of the~~ failures c~~an~~ould thus be mainly attributed to the presence of zones of high geotechnical complexity~~,~~ ~~associated~~ related to tectonic and~~with~~ ~~the~~ karst phenomena.

8. SOLUTIONS AND RECOMMENDATIONS

The rock mass can be ~~identified~~ described as ~~a~~ brecciated ~~brechified site~~ with significant karstification of the limestone. The presence of empty or partially filled cavities, with silty and sandy clay deposits of low cohesion is common.

Under these conditions it is difficult, with the usual procedures of excavation and support, to ensure the stability of the ~~pass~~ excavation without causing major instabilities given the loose nature of the materials filling ~~the~~ cavities and fractures. For initial containment, spiles with light beams and bolt umbrellas were used. As the volume of the unstabilised materials increased, it ~~was~~ became necessary ~~to~~ a ~~use a~~ systematic use of ~~of~~ heavy micropile umbrella~~s~~. However, ~~this~~ the heavy micropile umbrella proved to be insufficient ~~in the case of~~when crossing large cavities filled with soils.

Considering all the~~se~~ ~~previous~~ problematic situations previously described, it became necessary to define ~~a~~ new working procedures for the construction of the tunnel, to suitably d~~ue to~~eal with the karstified terrain characteristics~~,~~ and to seek~~ing~~ ~~to~~the increase in safety and construction efficiency.

It is important to highlight that the karst phenomena is one of the most difficult problems to solve in the ~~top heading~~advance front of a tunnel, ~~because~~ due to~~of~~ the great diversity of circumstances that may come up, and especially because of the variability of their occurrence. This is due to the erratic development of the dissolution processes, to the multitude of phenomena associated and to their influence on stability, depending on the ~~limestone~~ rock mass characteristics in which the karst developed.

The treatment procedures described below, in incremental sequence of complexity, were considered appropriate for dealing with each instability situation, ~~due to~~ and adjusted to the specific geotechnical characteristics of the terrain traversed by the tunnel. Note that ~~these~~ the following ground treatment procedures should be added to those previously described for the general support of the tunnel:

- *Case 1: Good geotechnical characteristics*. This is the most favourable situation in which the traversed ground, start to show signs of karstification, generating a negligible impact on the ~~enforcement~~ implementation~~process~~ of the tunnel. The limestone massif is stable and slightly weathered. Small cavities in the ~~gables~~ side-walls or in the ~~key~~ crown may be filled with shotcrete, assisted by the use of Bernold sheets as permanent formwork. In this case there would hardly be any instabilities or detachment of material into the excavated tunnel.

- *Case 2: Good to ~~regular~~ fair geotechnical characteristics*. It can be found in ~~areas~~ zones with low to medium karstification, with the presence of decalcification clay, filling some cavities~~. These~~, and would not produce significant detachments.

  ~~In the case~~If ~~of~~ instabilities would ~~appearing~~ develop in the side-walls~~in the gables~~, it sh~~w~~ould be sufficient to stabilise the cavities, to do dental cleaning of the clay materials, ~~(removal of clay material if any)~~ and fill ~~in~~ the voids with shotcrete or pumped ~~concrete or~~ lean concrete, and eventually use Bernold ~~sheets~~ plates as permanent formwork.

- In the ~~key~~ crown it may be necessary to use self-drilling anchors as ~~a measure of~~ pre-support, to ensure safety ~~in consecutive passes~~. In this situation the cavities should also be filled with shotcrete or lean concrete.

Case 3: ~~Regular~~ Fair geotechnical characteristics. The limestone rock mass is ~~moderately~~ fairly weathered, ~~, showing~~presenting large cavities filled by moderately cohesive materials, generating small detachments due to the ~~lack of~~ deconfinement. The volume and weight of these fill~~ers~~ c~~would~~an ~~be able to~~ overcome the ~~resistance~~ strength of the ~~pins~~ rock bolts, ~~while~~ not guaranteeing the safety of the ~~passes~~excavation.

For these zone~~s~~, it would be appropriate to adopt the use of heavy micropile umbrella~~s~~ 12 m long, spaced 40 cm between axes (considering the micropile ~~an approximate~~ diameter ~~of~~ around 90 mm ~~micropile~~), with an overlap of 3-4 m and adjusting the dimensions to suit ~~the~~ each problem detected ~~at all times~~. The micropiles have high ~~levels of~~ rigidity and ~~consequently a~~ high capacity to withstand the ~~the~~ loads from detachments of loose ~~soil~~ ground that may occur on the ~~boundary~~edge of the section.

The use of heavy ~~trusses~~steel ribs (HEB) would improve the support of the umbrella ~~because~~ due to ~~of~~ its ~~superior~~ high rigidity, ~~and~~ helping to absorb ~~specific~~ local loads ~~in the support ring areas~~.

In the event that during the ~~incorporation~~ implementation of the first phase of the umbrella no significant anomaly is detected, the~~n~~ injection of ~~into~~ the tubes in a single phase and through their mouth should be done.

Case 4: ~~Regular~~ Fair to poor geotechnical characteristics. If during the implementation ~~enforcement~~ of the micropile umbrella~~s~~ from the previous case, ~~-~~ zones of intense fracturing, ~~void or~~ filled cavities with soft material or empty voids are detected ~~are detected~~, a second phase of micropiles in the arch of the umbrella~~, covering such zones~~ (spac~~ing~~ed 25 cm between axes) should be inserted. This previous procedure should also be ~~done~~ used when, ~~,~~ in the first phase of the umbrella, ~~,~~ the grout injection pressure ~~process of grout~~ can´t be raised, indicating an uncontrolled admission of grout~~is uncontrolled and without pressure~~.

The number and location of alternating micropiles will depend on the spatial distribution of unstable areas along the tunnel.

~~In this situation, t~~The placement of a “temporary” ~~truss~~steel rib to support the first metre of the heavy umbrella is advisable. The alternating micropiles in this second stage should be equipped with two unidirectional valves (at 180°) with a diameter of 10 to 12 mm, ~~situated~~ located along the tube and spaced one meter between consecutive drills, allowing localized injections along the micropile tube ~~for the micropiles~~.

- *Case 5: Poor geotechnical characteristics*. When, during the ~~enforcement~~implementation of the micropiles, the ~~massif~~ ground mass worsens considerably, due to karst phenomena, it will be necessary to ~~use~~ inject~~ion of~~ grout through the micropiles ~~available~~ valves ~~in the micropiles~~ of the second phase. In ~~these~~ this case~~s~~ the injection may need to be done with the use of shutter~~ings~~s, in order to distribute as evenly as possible, the flow of grout along the micropile. This procedure ~~results~~ creates~~in the construction of~~ a reinforced injection umbrella.

- ~~With t~~The~~is~~ injection ~~it is~~ intends~~ed~~ to fill the empty cavities close~~st~~ to the crown of the tunnel and, when the cavities are filled with soil, to improve their properties ~~compacting the filling, in the event that there is any~~. It creates an injected ~~soil~~ ground crown between the micropiles, which significantly increases safety during the excavation ~~work~~ and support works. Subsequently the grout injection of the micropiles from the first phase should be undertaken.

- *Case 6: Very poor geotechnical characteristics*. In this case, the ~~massif~~ ground mass would generally ~~appear~~ be ~~generally~~ very unstable, and the above procedures will not guarantee the safety of the work in the tunnel. ~~Pre~~In this case the pre-support ~~processes~~techniques become ineffective and it is necessary to do systematic ~~to improvement treatments~~ ground treatment ~~near~~ around the excavated section.

To increase the stability conditions of the ground mass, improving its mechanical characteristics, either the injection of high cohesion products (cement grout or resin)~~,~~ or ~~a Jet Grouting treatments~~Jet Grouting treatments could be used.

This last option is the most difficult to implement, because the equipment required is highly specific and the construction procedures necessary to carry out the treatment are complex.

However, if necessary, this treatment would allow to solve the problem by creating a series of horizontal columns of ~~improved~~ reinforced ground ~~near~~ around the section to be excavated.

- *Case 7: Empty cavities and ~~instabilities~~landslides*. When there is ~~the~~ admission of grout without the rise in pressure, in a ~~particular~~specific ~~area~~part of the micropile, this ~~area~~zone should be interpreted as a cavity. In the event that a cavity ~~void~~ is detected in the first six meters, the umbrella may be considered to have a "bridge" type effect, and the top heading must be planned to reach, or even surpass, the cavity ~~area~~zone.

If the ~~bore~~ cavity is located in the second half of the umbrella, having a length of about two or three meters, it may be considered that the ~~under run~~ protection ~~of~~ provided by the umbrella would not be guaranteed, thus being ineffective. In this case, ~~prior fill of the bore~~ the previous fill of the cavity would be necessary ~~in order to obtain this under run protection~~.

Among the materials that can be used to fill a cavity, ~~various types may be distinguished~~are: lean concrete, mortar, resins, polyurethane or grout. It is advisable to ~~employ~~ use the cheapest material because the volumes to be filled may be ~~great~~huge.

In the case where a particular fill area or ~~bore~~ cavity is ~~located~~ identified in several consecutive micropiles within ~~one~~ an umbrella, it might be advisable to drill 2 or 3 bores in the ~~very~~ front, in order to define the ~~narrow~~ limit of the fill area, and act on it. The presence of a filled volume that can be suddenly emptied near the upper contour of the ~~shoulder~~ tunnel~~zone~~ may~~,~~ excessively increase the free span of the umbrella, causing its deformation.

In this case, the objective of the treatment is to stabilize the fill. The process would be similar to the one proposed for the umbrellas, taking into account that in this case the treatment must be compatible with the subsequent excavation. If an empty cavity appears in the ~~gables~~ side-walls of the tunnel, its effect would not be as great as in the case of the umbrella. In the case of intersecting a cavity filled with water, the only possibility is to drain it.

The seven cases described above, are summarising in ~~the table of~~ Table 5.

[Table 5]

Table 5. Summary ~~table~~ of special treatment procedures proposed for karstified ~~areas~~ zones of the Gavarres ~~T~~tunnel.

To reduce the uncertainty ~~due~~ about the grade ~~t~~of karstification of a limestone rock mass, in which a tunnel is to be constructed, the use of geophysical prospection techniques is highly recommended (Richter et al., 2008).

Electrical tomography ~~techniques are~~is especially useful ~~for~~ to ~~determining~~ determine the spatial distribution of the ground resistivity, ~~and~~ to locat~~inge~~e discontinuities or different terrain characteristics (faults, ~~lithologic~~lithological contacts, cavities, clay fillers, bedding ~~-~~planes, etc.).

During construction, the reconnaissance shall continue with horizontal ~~probe~~ borings~~holes~~ in the excavation front, or by monitoring the ~~holes~~ drillings made from the interior of the tunnel (drill holes, micropiles, etc.). The use of modern TSP seismic systems ~~TSP~~ can also be useful, allowing ~~to~~ analysing ~~analyse~~ the propagation of the seismic waves from the inside of the tunnel towards the ~~top heading~~advancing front.

As a long-term stability procedure, it is ~~necessary~~ advisable to ~~avoid~~ prevent the presence of ~~holes~~ voids close to the lining of the tunnel. A quick and efficient way to assess of the ~~status~~ presence of ~~holes~~ voids ~~in the back of~~behind the tunnel support is to use the georradar.

9. CONCLUSIONS

In the Gavarres tunnel, the problems reported were mainly caused by unsuitable ground behaviour, due to karstification and to the heterogeneous and unpredictable limestone rock mass, corresponding to geotechnical zones of very poor quality. The reduced cohesion and unsuitable geomechanical characteristics of the soils filling the karst cavities,~~,~~ generated ~~severe~~ serious instability problems and thus, the procedures initially proposed for the tunnel excavation and support were not ~~adequate~~ able to ensure a safe construction. Despite the problems reported, the deformations generated by tensions were irrelevant.

Due to partial or total excavation of the tunnel section, landslides and emptying of karst cavities filled with soils, begun to develop. The presence of medium size blocks (even metric) of limestone embedded in the filler soils, favour the collapse due to their own weight, detaching and dragging the materials of worse competence.

The early detection of karstified zones during site investigation, allow defining ~~an~~ adequate design and construction procedures, towards a successful excavation and support. It is of vital importance a correct geologic characterisation of the ground mass and the combined use of mechanical site investigation techniques with geophysical techniques (seismic, electrical tomography, georradar, etc.).

The use of pre-support of the section to be dug (bolts, micropiles, etc.) and of ~~soil~~ ground improvement techniques in the edge of the excavation (injections, backfilling, partial substitutions, etc.) proved to be highly efficient. Using this approach, personal injuries and/or economic losses related to the stoppage of the construction work or the need to redefine the excavation and support procedures during construction can be avoided.

The solutions and recommendations presented here may provide guidance for the study, design and construction ~~for~~ of ~~future~~ tunnels to be implemented in rock masses affected by karst processes. The technical validation of the proposed solutions was demonstrated by the successful completion of the Gavarres tunnel.

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Highlights (for review)

The design and construction of tunnels in karst areas is subject to problems > We present a tunnel built in a karst area and the problems and proposed solutions > Present proposals for future work in similar situations to the one discussed here > The validity of the solutions shown in the successful completion of the tunnel

TITLE

Geological engineering problems associated to tunnel construction in karst rock masses: The case of Gavarres tunnel (Spain)

AUTORS

S. Alija[a], F.J. Torrijo[b], M. Quinta-Ferreira[c]

[a]Geosciences Center, University of Coimbra, 3000-272 Coimbra, Portugal. Tel. +351 239860500. E-mail: santiagoalija@gmail.com

[b]Department of Earth Engineering, Universidad Politécnica de Valencia, 46022 Valencia, Spain. Tel. +34 963877582. E-mail: fratorec@trr.upv.es

[c]Department of Earth Sciences, Geosciences Center, University of Coimbra, 3000-272 Coimbra, Portugal. Tel. +351 239860500. E-mail: mqf@dct.uc.pt

ABSTRACT

A representative example of the problems associated with the excavation and support of tunnels in karst ground is presented. It is a peculiar case in terms of heterogeneity and spatial distribution of zones of poor geotechnical quality, requiring the need to define, preferably in the study phases, adequate site investigation, suitable design procedures, efficient construction techniques and appropriate ground treatment. The difficulties associated with the instability of the karstified ground, and the presence of cavities, wholly or partially filled with soils of low cohesion, are discussed via retrospective analysis. The solutions adopted to solve the problems encountered during the tunnel construction, enabled a systematic approach, useful for new construction projects in limestone terrains of medium to high karstification.

KEYWORDS

Tunneling in karst, karstified rock masses, retrospective analysis, Gavarres tunnel, NATM.

1. INTRODUCTION

The design and construction of tunnels in karst terrains is fraught with problems associated with the unexpected location, irregular geometry and unpredictable dimensions of the karst structures.

In a karstified terrain, prospection and regular testing campaigns should be supplemented with other techniques adapted to locate and anticipate the problematic zones. It must be taken into account that no site investigation technique is one hundred percent accurate, and therefore several techniques should be used, adapted to each specific situation, taking into consideration the budget for the work and the risks that can be assumed in the project.

A real case of a tunnel constructed in a karstified limestone ground is presented, the problems encountered are described and the proposed solutions are discussed. A systematic approach, as a knowledge tool for future work in similar situations, is presented.

2. GEOLOGICAL FRAMEWORK

From the geological point of view, the study area is located in the Les Gavarres region, which is included within the Catalan Transverse System, directly related with the Neogene depression of the Empordà (Agustí et al., 1994).

Les Gavarres region consists of a fringe of Paleogene materials (mainly Eocene), arranged around a Hercynian rock massif, outcroping at south of the study area. The age of these materials is prior to the Alpine Orogeny, as they have suffered deformation and fracturing during this tectonic phase. The series is dislocated in blocks, separated by fractures that lead to the uplifting of the massif. The general structure is a monocline arrangement, dipping mainly to Northeast (IGME, 1983, 1995). The geological formations affecting the tunnel are (Fig. 1):

- *Barcons Sandstone Formation (E_A)*. It is composed by glauconitic sandstone, medium to coarse grained, locally conglomeratic. The predominant colour is grey-yellowish or ochre. The grains are mainly of quartz and feldspar with a scarce clay matrix. It has calcareous cement and abundant bioclasts. At the base and top of the series, the layers are decimetric to metric, presenting a more massive appearance in the middle of the formation. The average sedimentation corresponds to a deposit in the frontal area of the delta, which is rather thick, but of limited extent. The age of the series is Eocene.

- *Banyolas Limestone Formation (E_M)*. This formation is composed of layers of limestone and marl, whose relative proportion varies throughout the series. They are of grey and bluish grey colours, and the layers have decimetric thickness. The carbonate content ranges from marly clay to limestone, affecting the materials strength, weatherability and the stability behaviour of the rockmass. Some spans of the series are mainly composed of hard clay and marls. The age of the series is Eocene. It is important to note that the Banyolas Limestone Formation is in concordance with the underlying Girona Fossiliferous Limestone Formation.

- *Girona Fossiliferous Limestone Formation (E_C)*. It is a fossiliferous limestone, presenting oolitic terms at the base. The predominant colour is ochre. It is rather recrystallized and arranged in layers of a wide range of thickness, from decimetric to metric. The environment of sedimentation corresponds to proximal marine environments of carbonate platform. The age of the series is Eocene.

- *Pontils Group Conglomerates (E_{CG})*. This formation is constituted by conglomerates and red sandstones with clay layers. These deposits have fluvial origin. The age of the series is Lower Eocene, but may also include part of the Palaeocene.

The boundary between the Les Gavarres region and the SW margin of the Ampurdán depression, is marked by a fracture alignment oriented NW–SE, called Banyolas Fault or Camós–Celrá. This alignment is part of a system of fractures orientated predominantly NW–SE. They are normal faults related to quaternary volcanism and recent seismicity. This important regional fault intersects the line of the tunnel, corresponding to intense fracturing of the rock material.

3. GEOTECHNICAL CHARACTERISTICS

According to the geological cross section defined in the design, most of the tunnel would be excavated in the materials of the Banyolas Marl Formation (Fig. 1) while the northern part is affected by the fault system associated with the Banyolas Fault or Camós–Celrá.

[Figure 1]

Fig. 1. Location map and geology profile along the Gavarres tunnel.

The two fundamental geotechnical units are described below. The results of the laboratory tests, from samples collected in the tunnel boreholes, are shown in the tables 1 and 2:

Limestone and Marl Geotechnical Unit. This unit is entirely constituted by calcareous rocks of the Banyolas Limestone Formation (E_M). The rock samples tested generally present medium to low strength, with a weathering grade, in the vicinity of the tunnel, ranging from III to V (according to ISRM, 1981). The seismic profiles carried out in the tunnel confirmed this data. The water table detected in the boreholes was located below the invert of the tunnel. The average densities (Table 1) and uniaxial compressive strength (Table 2) gave very scattered values, depending on the degree of weathering of the sample (Barton et al., 1974). During the site investigation programme, permeability tests revealed medium–low permeability terrains (González de Vallejo et al., 2002), around 1 x 10^{-7} m/s. Considering the RQD values obtained in the borehole samples and the uniaxial compressive strength, a representative RMR value of 30 was estimated (class IV or Bad, Bieniawski, 1989).

- *Fault Zone Geotechnical Unit (E_M very fractured)*. It is a highly fractured zone, where argillite, calcareous mylonite and marl have been identified. The rock weathering ranges from grade II to V (according to ISRM, 1981). Water levels were found at different heights, associated with fracture planes. Although most of the unit consists of highly fractured limestone and marl, from the Banyolas Limestone Formation, the presence of a small thickness of Girona *Fossiliferous Limestone* (E_C) was also observed in boreholes as well as conglomerates and red sandstones of the Pontils Group (E_{CG}). Both formations present weathering grades of IV-V (according to ISRM, 1981). The permeability tests showed low to medium values, similar to those usually presented by fractured rock masses of limestone and dolomite (1 x 10^{-6} m/s). Mainly based on the RQD values of the rock cores and on the uniaxial compressive strength values, an RMR value of 20 was estimated (class V or Very Poor, Bieniawski, 1989).

[Table 1]

Table 1. Sieve analysis and consistency limits (Atterberg limits) of the soil materials of the Gavarres tunnel.

[Table 2]

Table 2. Strength parameters of the Gavarres tunnel obtained over rock cores tested in the laboratory.

4. CONSTRUCTION PROJECT

The tunnel is part of the Madrid–French border high-speed railway line, and is located within the province of Girona (Fig. 1). It is a double track tunnel having a total length of 758 m with a maximum overburden of 31 m. The average altitude of the tunnel is 93,5 m above sea level.

The free section of the tunnel, defined in terms of health and comfort criteria, was 110 m^2. The geometric characteristics of the tunnel cross section were designed using a circular vault extending into the floor, without differentiating the gables (López, 1996).

Having in mind the characteristics of the in situ materials and the dimensions of the tunnel, it was considered that the mechanical excavation was the most suitable procedure and that blasting could be used in the unweathered limestone zones (Díaz, 1997).

The design recommended the use of the New Austrian Tunnelling Method (NATM), since it could allow pre-support during tunnel advance, through mechanical pre-cutting.

The excavation phases used in the tunnel were: one excavation phase in full section in top heading, two excavation sub-phases in the bench and one excavation phase in inverted vault.

In the design of the tunnel support, three section types were defined (Fig. 1), ranging from the better quality terrains to the weakest (Hoek and Brown, 1980, Hoek et al., 1995):

- S-II: this section type applies to the weathered calcareous rocks of the Banyolas Limestone Formation. The excavation should be performed in advances of 1.0 m in top heading, with a primary support based on a 5 cm sealing of shotcrete with steel fibre, light steel ribs type TH-29 and shotcrete with steel fibre, 25 cm thick in total (excluding the 5 cm of sealing). The two sub-phases of the bench were implemented in 2.0 m spans extending the support of the top heading.

- S-III: was used for the fault zone unit. In this section type, the excavation would be done in advances of 0.5 to 1.0 m with the support based on a 5 cm sealing of shotcrete with steel fibre, heavy steel ribs of type HEB-160 and shotcrete with steel fibre, 30 cm total thickness (excluding the 5 cm of sealing). The drilling of the bench would be done in two sub-phases, with advances from 1.0 to 2.0 m extending the support of the top heading.

- S-E: was the section type for the tunnel portals. It was characterised as type "heavy" as these zones were expected to be more weathered, decompressed due to the previous excavation of the portal slopes and presenting a rather thin overburden above the tunnel. The S-E section consisted of a heavy micropile umbrella, 20 m long and 150 mm in drilling diameter, spaced 0.5 m between axes and fitted with steel pipes, 110 mm of external diameter and 8 mm thick, filled with mortar. The excavation and support sequence for this section would be similar to S-III, with the difference that the steel ribs used below the umbrella would be type HEB-180.

All sections should have a concrete inverted vault with welded wire mesh 150 x 150 x 6 mm.

A summary table with the support structures defined for the tunnel is presented in **Table 3**.

[Table 3]

Table 3. Summary of the support structures proposed for the three section types of the Gavarres Tunnel.

5. CONSTRUCTION OF THE TUNNEL

The construction of the Gavarres tunnel began by its south portal in limestone materials (Fig. 1). First, the excavation and support of the portal slopes was carried out. The excavation was done using mechanical heavy duty rotating machines. During this early stage of excavation, the heterogeneity of the limestone rock mass was detected. The working face presented very weathered areas, easy to excavate, alternating with limestone, very difficult to break mechanically.

Once at the tunnel crown level, a micropile umbrella was carried out, for the S-E section type (35 micropiles in total). During the implementation of these micropiles, the heterogeneity of the ground continued to be revealed, since the implementation speed ranged from 1 to 4 micropiles per day. The drilling residues changed drastically from limestone fragments to a clay-like material.

According to the geotechnical characteristics of the ground during excavation and support, mainly associated with the karstification processes, different zones were considered along the tunnel (Alija, 2010):

- *Portal Zone (sections 0 – 22)*. The excavation of the tunnel started with mechanical equipment, reaching an average progress speed of 4.7 m/day. In this zone, four sections of convergence were installed and eight engineering geology front maps were prepared. The ground materials were characterised as blocks of limestone and marl, sometimes broken, embedded in a clay matrix. The stratification was:

    - S_0: oriented between 200/15 – 200/30 (dip direction/dip angle), with some continuity and some roughness. Between layers, openings of 5 to 10 mm were observed, filled with clay or even calcite.

  Two families of joints were identified (Fig. 2):

    - J_1: with an average orientation of 213/71, spaced about 30 cm, with some continuity and, when filled, it is with clay material.

    - J_2: with an average orientation of 124/70, spaced from 20 to 60 cm, very rough and usually closed.

  These two families of joints and the stratification maintained their orientation all along the tunnel, but due to the heterogeneity of the rock mass, they were not found or distinguished on all of the fronts mapped.

  According to the front reports, the average RMR value obtained for this area was 36, corresponding to a rock mass of class IV (poor grade). During the excavation and support operations, small falls of rock and clay occurred. In section 2 and 3, the instabilities in the roof of the tunnel achieved 12 m^3. Detachments were also produced in the right side of the roof and gable, of section 13 and 14, achieving 15 m^3. Instabilities also occurred in the gable and right shoulder in section 20 to 21. These detachments showed the presence of small fragments of limestone embedded in a clay matrix. The large volume of fallen materials required its filling with concrete, using Bernold plates as lost formwork. Throughout this zone, a portal section type (S-E) was implemented.

[Figure 2]

Fig. 2. Tunnel-working face view of section 19 with details of the stratification (S_0) and joints (J_1, J_2). Marl and limestone blocks, some broken, in a clay matrix.

Zone 1 (sections 22 – 38). In this zone, heterogeneous tunnel-working faces appeared, composed of very compact marly limestone layers, and occasionally with clay or even calcite filling the spaces in the clay matrix. In the final metres of the zone wet spots in the clay were often found, and some karstification voids were identified (Anguita and Moreno, 1993). The advance speed in this zone slowed down to an average of 3.3 m/day, because very specific blasting was required to break up the hardest limestone materials. The rock mass appeared to have higher hardness than in the previous zone. Nine engineering geology front maps were prepared. The average RMR value was 43, allowing a fair to good stability of the front, and the advance in passes of 1 m using mechanical excavation. The RMR values obtained in this section were compatible with the S-II support type. However, taking into account the heterogeneity of the fronts and the thin overburden, it was decided to use a conservative stance, and the support defined for S-III with passes of 1 m was installed. The accumulated strain denoted a clear evolution towards stabilization, reaching values under 5 mm.

- *Zone 2 (sections 38 – 91)*. Corresponds to 53 meters with moderate karstification. Several instabilities occurred, composed of highly fractured marl-limestone rock fragments embedded in a clay matrix. The weathering grade of this zone ranges from III to IV (Fig. 3).

[Figure 3]

Fig 3. Tunnel-working faces with weathering degree III in Zone 2 (section 61). Highly fractured marl-limestone rock fragments embedded in a clay matrix.

The average advance speed of the excavation was slightly reduced to 3.18 m/day, due to the decrease in the geotechnical characteristics of the ground mass. The strength of the limestone fragments decrease from an average of 44 MPa in Zone 1 to 34 MPa in Zone 2. The materials observed in this zone were highly fragmented and the damp spots were a constant. The RMR values were in the range 25 - 35, corresponding to a poor quality rock mass of class IV, in which S-II or S-III support type could be installed.

On site it was decided to maintain the S-III support. The convergence strain was generally lower than 4 mm. Once passed Zone 1, steel ribs HEB-160 and 35 cm of shotcrete HM-35 (S-III) continued to be used for more 15 meters, until section 54 when rockfalls began to occur. This problem required the use of a pre-support system based on a light bolt umbrella (4 m long with an overlap of 2 m) and packed with light beams (Fig. 4).

[Figure 4]

Fig 4. Detail of the support with an umbrella of steel rods 4 m in length and packed with light beams.

Due to the increase in the number and size of the instabilities it was decided to increase the number of the bolts and to raise their length from 4 m to 6 m, overlapping 3 m. Despite this reinforcement, masses of clay and limestone blocks were still able to strip out the bolts (Fig. 5) requiring the improvement of the support solution.

[Figure 5]

Fig 5. View of an instability with a distorted light umbrella in zone 2 (section 70).

However, this light umbrella was used until section 80 where it was decided to place the first self-drilling heavy micropile umbrella, 12 m long and 90 mm in diameter, with an overlap of 3 meters between umbrellas. From then on, the support with heavy umbrellas was used systematically. These umbrellas were made by approximately 35 micropiles, separated around 40 cm between their axes. The number of micropiles was dependent on the characteristics of the front at the time of excavation, and was decided according working crew experience and to the technical assistance criteria.

Zone 3 (section 91 – 235). In this zone of 163 m in length, the RMR values slightly increased, varying from 30 to 40 (class IV – bad). Many karstified rock masses appeared in the tunnel-working faces, causing several rockfalls, greater than those occurred up to this point. The geology is characterised by the dominance of limestone and clay. The limestone showed well-defined layers in the initial stretch of the zone, being more bulky and amorphous towards the final, turning difficult to disclose the orientations of S_0, J_1 and J_2. The uniaxial compressive strength presented an average of 32 MPa. The tunnel-working faces were dry and seemingly less fractured than in Zone

2. Fourteen convergence sections and one instrumentation cross-section were installed. Twenty engineering geology front maps were prepared. The measured strains showed tendency to stabilize, reaching maximum values under 5 mm in the convergence sections, while the extensometers measured up to 9.5 mm in the key during the advance. The pressure cells measured stresses from 0.05 to 0.1 MPa. The evidence of the effectiveness of the micropile umbrellas provided by the lower accumulated deformations, allowed the decision to switch to a lighter support, formed by TH-29 steel ribs (S-II). The advance speed of the tunnel increased up to 3.8 m/day due to the safety provided by the micropile umbrellas.

- *Rockfall Zone (sections 235 – 463)*. This zone is characterised by a significant decrease of the RMR with values from 25 to 45 (poor quality, class IV), due to the presence of abundant damp spots, with some dripping being observed, and the decrease of the rock uniaxial compressive strength to an average 27 MPa. The advance rate raised slightly to 4 m/day, due to the increased mastery of the working crew on placing the micropile umbrella. In this zone, two convergence sections were installed and six engineering geology cross sections of the tunnel face were mapped. The zone is composed of marly limestone materials without a clear arrangement, in which the joints are almost indistinguishable. Unlike the rest of the tunnel, here the damp spots increase, being observed some dripping and many karstification voids. Associated with the karst phenomena in this zone, several rockfalls occurred, even forcing to stop the work at chainage 501+462, due to sudden, large rockfalls, requiring new work procedures.

A summary of the main characteristics of the tunnel zones previously described is presented in Table 4.

[Table 4]

Table 4. Main characteristics of the tunnel zones.

6. THE PROBLEMS

Since its beginning, the Gavarres tunnel presented a series of geotechnical complexities (rockfalls, detachments, over-excavations, etc.) that slowed down and hindered the excavation. These problems, related to karst phenomena (Ford and Williams, 1989), were not foreseen in the design.

The instabilities occurred during the excavation or support works, mainly in materials of brecciated aspect, consisting of boulders and blocks of limestone in a soft clay-marly matrix, which quickly collapsed or slide from the front, shoulders or crown of the tunnel. As the tunnel advanced it became more frequent the presence of cavities, empty or partially filled by decalcification clays. These cavities (Fig. 6) can also be problematic due to the absence of support between the tunnel lining and the ground, eventually causing problems throughout the life time of the tunnel.

[Figure 6]

Fig. 6. View of a cavity of approximately 20 m^3 affecting sections 206 to 210.

Due to the poor geotechnical quality of the terrain, spiles and light micropile umbrellas were implemented but they were unable to stop the increase of the instabilities. For this reason it was decided to systematically use successive micropile umbrella 12 m long, overlapping 3 m. With this solution gravitational instabilities still occurred, affecting the material that fell between the micropiles.

At dawn, of a normal work day on the Gavarres tunnel, a large instability hit the crown and right gable on section number 302 at chainage 501+462 (Fig. 7).

[Figure 7]

Fig. 7. View of the large instability affecting section 302, obliging to stop the work in the tunnel.

Thanks to the description of the workers at that time inside the tunnel, we know that the excavation round was running normally, after excavation the shotcrete sealing was applied and the steel rib was put in place, but while the shotcrete robot was going into the front to finish the support, occurred a rustle and a sudden break in the shotcrete sealing, in the key and right side-wall zone, followed by the slide of a large mass of clay and rock fragments into the tunnel. This slide gave sufficient time to workers to escape without personal injury.

The next day it was found that the instability was constituted of limestone blocks and sharp edges, embedded in clay materials, typical of the decalcification processes with high humidity. The volume of material introduced into the tunnel was about 200 m^3 and left no visible cavity. The fallen material formed a “stable” cone of loose material, which occupied most of the excavated section, sustained and stopped a larger amount of material, as it was evident that the cavity above the tunnel was not emptied. The visible consequences were the breaking of a large number of micropiles and the deformation of the last steel rib. Once excavated the fallen material, the gap was sealed and the deformed steel rib was replaced.

The stability problem appeared to be due to a gravitational collapse on the front and crown, of deposits associated with karst phenomena. Later on, several dolines (sinkholes) were identified at the ground surface above the failure. As observed, the deposits associated with karst phenomena, due to their low cohesion and strength, frequently cause instabilities when traversed by a tunnel (Jianjy and Jian, 1987).

7. CAUSES AND POSSIBLE EXPLANATIONS

After the failure previously described, that obliged to stop the tunnel works, new geological studies were done, based on the information obtained during the excavation and support of the tunnel. These studies allowed the reinterpretation of the geology of the area, helping to explain the abundance of karst phenomena not previously identified during the design.

In this new interpretation, it was concluded that the tunnel instability occurred in the Girona Fossiliferous Limestone Formation and not in the Banyolas Limestone Formation. The fossiliferous limestone of the Girona Formation is more susceptible to karst phenomena in zones of intense tectonic fracturing, like the one in which the tunnel was being dug.

The main causes that led to this interpretation error were the following:

- a diffuse contact between the two geological formations (interdigitations);
- abundant vegetation;
- absence of outcrops.

The failures could thus be mainly attributed to the presence of zones of high geotechnical complexity related to tectonic and karst phenomena.

8. SOLUTIONS AND RECOMMENDATIONS

The rock mass can be described as brecciated with significant karstification of the limestone. The presence of empty or partially filled cavities, with silty and sandy clay deposits of low cohesion is common. Under these conditions it is difficult, with the usual procedures of excavation and support, to ensure the stability of the excavation without causing major instabilities given the loose nature of the materials filling cavities and fractures. For initial containment, spiles with light beams and bolt umbrellas were used. As the volume of the unstabilised materials increased, it became necessary a systematic use of heavy micropile umbrella. However, the heavy micropile umbrella proved to be insufficient when crossing large cavities filled with soils. Considering all the problematic situations previously described, it became necessary to define new working procedures for the construction of the tunnel, to suitably deal with the karstified terrain characteristics and to seek the increase in safety and construction efficiency.

It is important to highlight that the karst phenomena is one of the most difficult problems to solve in the advance front of a tunnel, due to the great diversity of circumstances that may come up, and especially because of the variability of their occurrence. This is due to the erratic development of the dissolution processes, to the multitude of phenomena associated and to their influence on stability, depending on the rock mass characteristics in which the karst developed.

The treatment procedures described below, in incremental sequence of complexity, were considered appropriate for dealing with each instability situation, and adjusted to the specific geotechnical characteristics of the terrain traversed by the tunnel. Note that the following ground treatment procedures should be added to those previously described for the general support of the tunnel:

- *Case 1: Good geotechnical characteristics*. This is the most favourable situation in which the traversed ground, start to show signs of karstification, generating a negligible impact on the implementation of the tunnel. The limestone massif is stable and slightly weathered. Small cavities in the side-walls or in the crown may be filled with shotcrete, assisted by the use of Bernold sheets as permanent formwork. In this case there would hardly be any instabilities or detachment of material into the excavated tunnel.

- *Case 2: Good to fair geotechnical characteristics*. It can be found in zones with low to medium karstification, with the presence of decalcification clay, filling some cavities, and would not produce significant detachments. If instabilities would develop in the side-walls, it should be sufficient to stabilise the cavities, to do dental cleaning of the clay materials, and fill the voids with shotcrete or pumped lean concrete, and eventually use Bernold plates as permanent formwork. In the crown it may be necessary to use self-drilling anchors as pre-support, to ensure safety. In this situation the cavities should also be filled with shotcrete or lean concrete.

Case 3: Fair geotechnical characteristics. The limestone rock mass is fairly weathered, presenting large cavities filled by moderately cohesive materials, generating small detachments due to the deconfinement. The volume and weight of these fill can overcome the strength of the rock bolts, not guaranteeing the safety of the excavation. For these zone, it would be appropriate to adopt the use of heavy micropile umbrella 12 m long, spaced 40 cm between axes (considering the micropile diameter around 90 mm), with an overlap of 3-4 m and adjusting the dimensions to suit each problem detected. The micropiles have high rigidity and high capacity to withstand the loads from detachments of loose ground that may occur on the edge of the section. The use of heavy steel ribs (HEB) would improve the support of the umbrella due to its high rigidity, helping to absorb local loads. In the event that during the implementation of the first phase of the umbrella no significant anomaly is detected, the injection of the tubes in a single phase and through their mouth should be done.

Case 4: Fair to poor geotechnical characteristics. If during the implementation of the micropile umbrella from the previous case, zones of intense fracturing, filled cavities with soft material or empty voids are detected, a second phase of micropiles in the arch of the umbrella (spaced 25 cm between axes) should be inserted. This previous procedure should also be used when, in the first phase of the umbrella, the grout injection pressure can´t be raised, indicating an uncontrolled admission of grout. The number and location of alternating micropiles will depend on the spatial distribution of unstable areas along the tunnel. The placement of a "temporary" steel rib to support the first metre of the heavy umbrella is advisable. The alternating micropiles in this second stage should be equipped with two unidirectional valves (at 180°) with a diameter of 10 to 12 mm, located along the tube and spaced one meter between consecutive drills, allowing localized injections along the micropile tube.

Comment [M1]: 20 ?
If the second micropiles phase is to be insert between the existing micropiles in th 1st phase, the half distance between axes is 20 cm.
The 2nd micropiles would be placed in the same plan as the ones in the 1st phase? or they should be placed in a different alignment.

- *Case 5: Poor geotechnical characteristics*. When, during the implementation of the micropiles, the ground mass worsens considerably, due to karst phenomena, it will be necessary to inject grout through the micropiles valves of the second phase. In this case the injection may need to be done with the use of shutters, in order to distribute as evenly as possible, the flow of grout along the micropile. This procedure creates a reinforced injection umbrella. The injection intends to fill the empty cavities close to the crown of the tunnel and, when the cavities are filled with soil, to improve their properties. It creates an injected ground crown between the micropiles, which significantly increases safety during the excavation and support works. Subsequently the grout injection of the micropiles from the first phase should be undertaken.

Case 6: Very poor geotechnical characteristics. In this case, the ground mass would generally be very unstable, and the above procedures will not guarantee the safety of the work in the tunnel. In this case the pre-support techniques become ineffective and it is necessary to do systematic ground treatment around the excavated section. To increase the stability conditions of the ground mass, improving its mechanical characteristics, either the injection of high cohesion products (cement grout or resin) or Jet Grouting treatments could be used. This last option is the most difficult to implement, because the equipment required is highly specific and the construction procedures necessary to carry out the treatment are complex. However, if necessary, this treatment would allow to solve the problem by creating a series of horizontal columns of reinforced ground around the section to be excavated.

Case 7: Empty cavities and landslides. When there is admission of grout without the rise in pressure, in a specific part of the micropile, this zone should be interpreted as a cavity. In the event that a cavity is detected in the first six meters, the umbrella may be considered to have a “bridge” type effect, and the top heading must be planned to reach, or even surpass, the cavity zone. If the cavity is located in the second half of the umbrella, having a length of about two or three meters, it may be considered that the protection provided by the umbrella would not be guaranteed, thus being ineffective. In this case, the previous fill of the cavity would be necessary. Among the materials that can be used to fill a cavity, are: lean concrete, mortar, resins, polyurethane or grout. It is advisable to use the cheapest material because the volumes to be filled may be huge. In the case where a particular fill area or cavity is identified in several consecutive micropiles within an umbrella, it might be advisable to drill 2 or 3 bores in the front, in order to define the limit of the fill area, and act on it. The presence of a filled volume that can be suddenly emptied near the upper contour of the tunnel may excessively increase the free span of the umbrella, causing its deformation. In this case, the objective of the treatment is to stabilize the fill. The process would be similar to the one proposed for the umbrellas, taking into account that in this case the treatment must be compatible with the subsequent excavation. If an empty cavity appears in the side-walls of the tunnel, its effect would not be as great as in the case of the umbrella. In the case of intersecting a cavity filled with water, the only possibility is to drain it.

The seven cases described above, are summarising in Table 5.

[Table 5]

Table 5. Summary of special treatment procedures proposed for karstified zones of the Gavarres tunnel.

To reduce the uncertainty about the grade of karstification of a limestone rock mass, in which a tunnel is to be constructed, the use of geophysical prospection techniques is highly recommended (Richter et al., 2008). Electrical tomography is especially useful to determine the spatial distribution of the ground resistivity, to locate discontinuities or different terrain characteristics (faults, lithological contacts, cavities, clay fillers, bedding planes, etc.).

During construction, the reconnaissance shall continue with horizontal borings in the excavation front, or by monitoring the drillings made from the interior of the tunnel (drill holes, micropiles, etc.). The use of modern TSP seismic systems can also be useful, allowing analysing the propagation of the seismic waves from the inside of the tunnel towards the advancing front. As a long-term stability procedure, it is advisable to prevent the presence of voids close to the lining of the tunnel. A quick and efficient way to assess of the presence of voids behind the tunnel support is to use the georradar.

9. CONCLUSIONS

In the Gavarres tunnel, the problems reported were mainly caused by unsuitable ground behaviour, due to karstification and to the heterogeneous and unpredictable limestone rock mass, corresponding to geotechnical zones of very poor quality. The reduced cohesion and unsuitable geomechanical characteristics of the soils filling the karst cavities, generated serious instability problems and thus, the procedures initially proposed for the tunnel excavation and support were not able to ensure a safe construction. Despite the problems reported, the deformations generated by tensions were irrelevant.

Due to partial or total excavation of the tunnel section, landslides and emptying of karst cavities filled with soils, begun to develop. The presence of medium size blocks (even metric) of limestone embedded in the filler soils, favour the collapse due to their own weight, detaching and dragging the materials of worse competence.

The early detection of karstified zones during site investigation, allow defining adequate design and construction procedures, towards a successful excavation and support. It is of vital importance a correct geologic characterisation of the ground mass and the combined use of mechanical site investigation techniques with geophysical techniques (seismic, electrical tomography, georradar, etc.).

The use of pre-support of the section to be dug (bolts, micropiles, etc.) and of ground improvement techniques in the edge of the excavation (injections, backfilling, partial substitutions, etc.) proved to be highly efficient. Using this approach, personal injuries and/or economic losses related to the stoppage of the construction work or the need to redefine the excavation and support procedures during construction can be avoided.

The solutions and recommendations presented here may provide guidance for the study, design and construction of tunnels to be implemented in rock masses affected by karst processes. The technical validation of the proposed solutions was demonstrated by the successful completion of the Gavarres tunnel.

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Table 1

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Geotechnical Unit	Characteristics	Values	Specific gravity (g/cm^3)	Unit weight (g/cm3)	Atterberg limits			Sieve analysis		
					Liquid limit (%)	Plastic limit (%)	Plasticity index (%)	Gravel (%)	Sand (%)	Silt or Clay (%)
Limeston and Loam		Test number	4	11	30	30	30	29	29	29
	Bedrock	max	2,67	2,06	38,00	21,00	18,00	64,00	38,00	98,00
	Altered Rock	min	1,76	1,54	23,40	12,40	7,90	0,00	2,00	25,00
		mean	2,12	1,79	30,09	16,34	13,75	14,11	15,74	70,26
Fault Zone		Test number		15	10	10	10	10	10	10
	Bedrock	max		2,55	40,30	18,70	21,60	21,00	49,00	98,00
	Altered Rock	min		1,71	26,80	13,00	11,90	0,00	1,00	30,00
		mean		2,03	33,31	16,34	16,97	3,60	10,80	85,60

Table 2

Click here to download Table: Table 2.docx

Geotechnical Unit	Characteristics	Values	UCS (kg/cm^2)	Triaxial test (CU) Effective cohesion (kg/cm^2)	Triaxial test (CU) Effective friction angle (º)	Direct shear test (CD) Effective cohesion (kg/cm^2)	Direct shear test (CD) Effective friction angle (º)
Limeston and Loam		Test number	18	4	4	1	1
	Bedrock	max	183,6	0,42	35,83	0,10	44,00
	Altered Rock	min	0,3	0,14	24,95	0,10	44,00
		mean	26,03	0,24	31,12	0,10	44,00
Fault Zone		Test number		1	1		
		max		0,3	28		
		min		0,3	28		
		mean		0,3	28		

Table 3
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Section Type	Geotechnical Unit	RMR	Excavation /pass	Shotcrete with steel fibre (cm)	Trusses	Special systems
S-II	Limestone and Loam	30-45	Top heading (1 phase) and bench (2 phases) 2m bench	5 (sealing)+25 HM-35	TH-29 // 1 - 1,5 m	Elephant foot Inverted vault
S-III	Fault Zone	20-29	Top heading (1 phase) and bench (2 phases) 2m bench	5 (sealing)+30 HM-35	HEB-160 // 0,5 - 1 m	Elephant foot Inverted vault
SE	Outlets Portals	<19	Top heading (1 phase) and bench (2 phases) 2m bench	5 (sealing)+30 HM-35	HEB-180 // 0,5 - 1 m	Heavy micropile umbrellas ϕ = 150 mm Elephant foot Inverted vault

Table 4

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	OUTLET ZONE 1	ZONE 1	ZONE 2	ZONE 3	DETACHMENT ZONE
Length (m)	20 (P0 - P22)	16 (P22 - P38)	53 (P38 - P91)	162 (P91 - P253)	50 (P253 - P303)
Material	Calcareous blocks in clay matrix	Calcareous blocks in clay matrix	Calcareous blocks in clay matrix G-III --> G-IV	Initially calcareous blocks and clay, clay masses at the end	Loamy-clay materials without clear structure
Structure		So = 195/14 J1 = 215/74 J2 = 127/76			-
Pass heading rate	Pass 1 m 4,7 m/day	Pass 1 m 3,3 m/day	Pass 1 m 3,18 m/day	Pass 1 m 3,8 m/day	Pass 1 m 4 m/day
Water	Dry (occasional damp spots)	Frequent spots	Abundant moisture	Dry fronts and less fractured	Abundant damp spots and drips
RCS		44 MPa	34 MPa (↓)	32 MPa (↓)	28 MPa
Support	S-E	S-III'	S-III'	S-III' --> SII'	SII'
Convergences maximum shortening (mm)	A-B: 8 - 11 Z: 3 - 17	A-B: 3 - 5 Z: 3 - 4	A-B: 1 - 9 Z: 0 - 4	A-B: 0 - 2 Z: 0 - 5	-
Monitoring sections	-	-	-	0,05 MPa A --> 10 mm D --> 17 mm	-
Special treatments	Tie beam - Visor Micropiles 22 m	-	Light micropile Heavy micropile	Heavy micropile	Heavy micropile
Excavation	Regular mechanical	Mechanical and specific blasting	Regular mechanical	Easy mechanical	Easy mechanical
Observations	Small detachments of calcareous blocks embedded in clay matrix	Cavities from the karstification processes	Vey karstified fronts, and broken up by frequent detachments	Increased instability caused by karstification. Large detachments	Very frequent detachments and holes. Stop the tunnel in drive (large detachment)

Table 5

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CASE	TYPE	DESCRIPTION	PRE-SUPPORT TREATMENT	IMPROVEMENT TREATMENT
1	Site with good geotechnical characteristics	Slightly karstified limestone massif. Small cavities	<>	Fill cavity with gunite. Use of Bernold sheets
2	Site with good-regular geotechnical characteristics	Massif with little karstification, with cavities of certain body (filled or not)	Light bolt umbrella	Fixing and filling the cavities with gunite or lean concrete.
3	Site with regular geotechnical characteristics	Moderately karstified massif. Frequent large cavities. Frequent landslides.	Micropile umbrellas of f 90, 12 m long and spaced 40 cm from the axis and with an overlap of 3 m (phase 1)	<>
4	Site with regular-poor geotechnical characteristics	Moderately karstified massif with located detachments and soil flows in favour of cavities between micropiles	Micropile umbrellas of f 90, 12 m long and spaced 25 cm from the axis and with an overlap of 3 m (phase 2)	<>
5	Site with poor geotechnical characteristics	Very weathered massif. Unstable ground and loosely cohesive with significant landslides	Micropile umbrellas of f 90, 12 m long and spaced 25 cm from the axis and with an overlap of 3 m (phase 2)	Injection of grout or mortar through the valves of the micropiles of phase 2 (Armed Injection)
6	Site with very poor geotechnical characteristics	Very karstified massif. Highly unstable and unsafe terrain. Ineffectiveness of the pre-support treatments	<>	Injection of high cohesion products or soil improvement by replacement (Jet Grouting)
7	Empty cavities and Landslides	Detection of areas with cavities, cavities generated by landslides or fallen debris produced by landslides	<>	Consolidation injections and filling cavities. Use of lean concrete, mortar, synthetic resins or grout.

Figure 1
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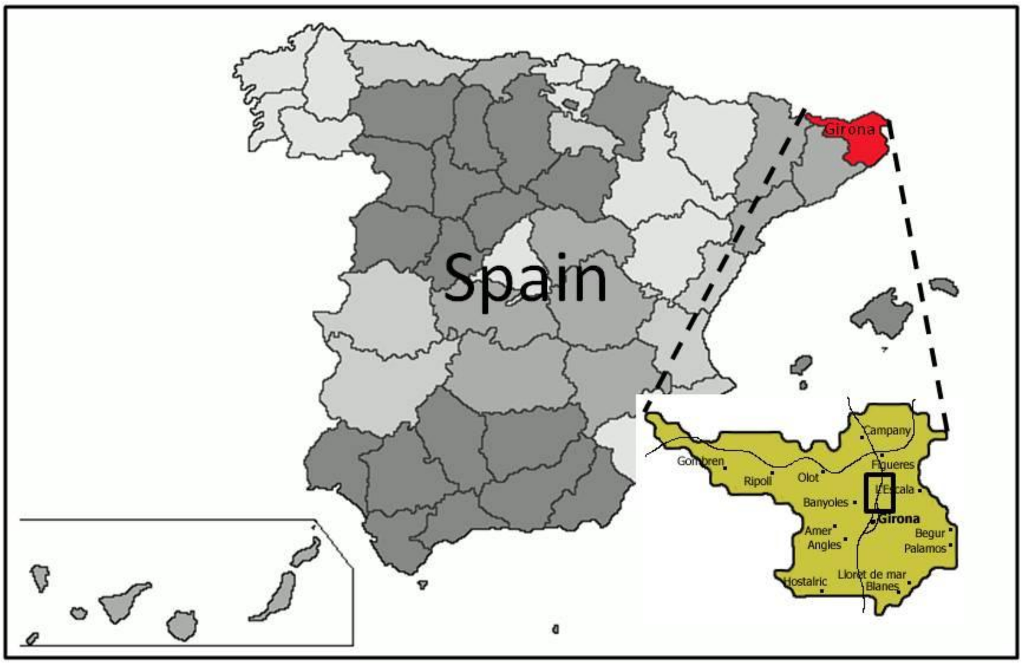

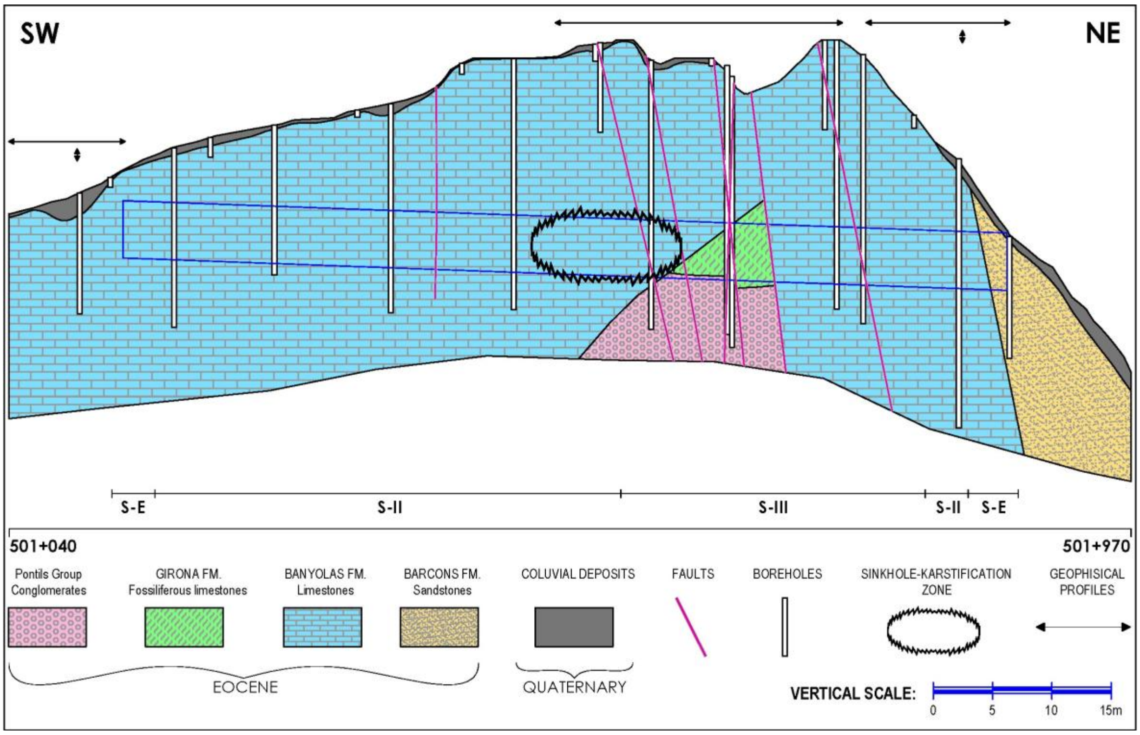

Figure 2

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Figure 3

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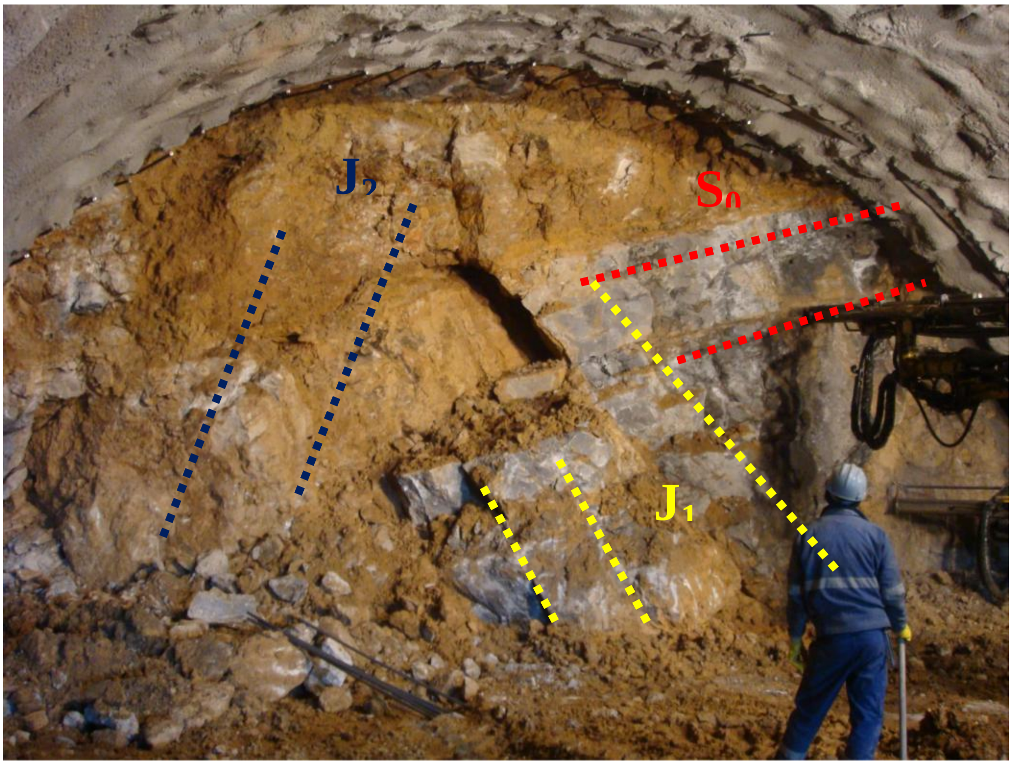

Figure 4

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Figure 5
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Figure 6
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Figure 7

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