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Study of the unexpected collapse of Ampurdán Tunnel (Spain) using Finite Element Model

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Abstract:	The paper presents the case study of the Ampurdán tunnel that suffered an unexpected partial collapse during construction, due to the weathering of the claystone groundmass, after excavation and wetting by infiltration water. To overcome the problems encountered, the finite elements model was used to understand the behaviour of the tunnel and surrounding ground, to determine the geotechnical properties that lead to failure, allowing to choose suitable procedures for the construction of the tunnel. The parametric study performed simulated the deformations measured in situ and related with the tunnel collapse. The geotechnical parameters used for the weathered claystone, when compared with the ones of intact situation, correspond to a wide range of reductions between 8% in the apparent density and in the effective friction angle, up to 40% in the effective cohesion and 56% in the Young modulus.
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Study of the unexpected collapse of Ampurdán Tunnel (Spain) using Finite Element Model

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Abstract The paper presents the case study of the Ampurdán tunnel that suffered an unexpected partial collapse during construction, due to the weathering of the claystone groundmass, after excavation and wetting by infiltration water. To overcome the problems encountered, the finite elements model was used to understand the behaviour of the tunnel and surrounding ground, to determine the geotechnical properties that lead to failure, allowing to choose suitable procedures for the construction of the tunnel. The parametric study performed simulated the deformations measured in situ and related with the tunnel collapse. The geotechnical parameters used for the weathered claystone, when compared with the ones of intact situation, correspond to a wide range of reductions between 8% in the apparent density and in the effective friction angle, up to 40% in the effective cohesion and 56% in the Young modulus.

Keywords Tunnel collapse, Softrock, Claystone, Finite element method

Introduction

The problems occurred in the Ampurdán tunnel during its construction are present, whilst emphasising the usefulness of computational tools based on finite elements.

The failure can be mainly related to the fast weathering of the tertiary materials on which the tunnel was built. The use of a finite element modelling program (Plaxis) allowed to identify the geotechnical parameters responsible for the problems occurred during the construction works (Wang et al. 2012).

The Ampurdán tunnel is located in the province of Girona and is part of the high-speed line between Madrid-Zaragoza-Barcelona-French border. It is a double-track tunnel, which has a length of 649 m and a maximum vertical offset of 24 m.

Geological framework

The study area is located in the Neogene tectonic depression of Ampurdán's, which is included in the Catalan Traverse System. The tunnel was built in the Miocene materials filling the depression (Agustí et al. 1994; IGME 1995). This depression was originated during a distension process in favour of normal faults, mainly oriented NW-SE, leading to the formation of a basin that was filled with materials eroded from the surrounding reliefs.

Along the tunnel route, only one lithostratigraphic unit is intersected: the Ampurdán Formation (M_A). This series of detrital origin is predominantly constituted by red claystone and siltstone, with interbedded arkosic sandstones and conglomerates constituting paleo channels of the alluvial fans that filled the Ampurdán basin.

In general, there is a gradation of materials from the most detrital, and large grain size, at the edge of the basin, South of the study area, to the finer materials with carbonated levels of caliche, toward the Northern basin centre, where more distal facies have been deposited. From the hydrogeological viewpoint, due to the predominance of mudstone materials, this formation is almost impermeable. Some

1 hanging water tables are associated with the most permeable layers of sand or
2 gravel, interbedded in the clay.
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7 **Geotechnical characterisation**

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10 The geotechnical information used in the implementation of the tunnel was
11 provided by site investigation carried out during the design, and by a
12 complementary campaign, executed before the start of the construction works.
13 This last campaign did not entail significant changes in the geotechnical
14 parameters of the Ampurdán Formation (M_A). The site investigation allowed to
15 obtaining the main geotechnical parameters of the claystone groundmass (Table 1)
16 and the preparation of the profile presented in Fig. 1.
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26 **Table 1** Summary of the geotechnical parameters considered in the design of the Ampurdán tunnel
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32 Fig. 1 Profile of the Ampurdán tunnel. The arrow points the advance direction
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38 **The construction project**

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41 The free section of the tunnel defined according to health and comfort criteria was
42 110 m^2 . The geometric features of the tunnel cross-section were designed with a
43 circular dome extending until the invert, without differentiating the sidewalls. In
44 Fig. 2 is presented the layout of the tunnel cross-section.
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53 Fig. 2 Cross-section of the tunnel with elephant's foot. I - top heading, II and III - bench, IV -
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Taking into account the characteristics of the ground and the tunnel dimensions, it was decided that mechanical excavation was the most efficient procedure (López Jimeno 1996). The New Austrian Tunnelling Method (NATM) was selected, and excavation phases were used: excavation of top heading, two lateral bench excavations and an invert excavation phase (Cornejo Álvarez 1988; Díaz Méndez 1997).

To control and verify the stress and strain conditions, a surveillance system was implemented, to ensure an early detection of the tunnel behaviour.

For the selection of the tunnel support, several known empirical methods were considered, based on geomechanical rock mass classifications (Bieniawski 1989; Romana 1997; Barton 2000; Hoek 1998; Hoek & Brown 1985, 1988). For the Ampurdán Clay unit, the geomechanical parameters considered are: RMR = 20; GSI = 20; Q = 0.10.

In the design of the tunnel support, due to the groundmass homogeneity a single section type was defined. For the portals, a special section was considered. The section types were evaluated using numerical models in 2D.

The main section of the tunnel was identified as type S-III. In this section, the excavation would take place by passes of 0.5 - 1.0 m. The support would be based on a 5 cm sealing of shotcrete reinforced with fibres, heavy HEB-160 steel ribs and shotcrete reinforced with fibres 30 cm thick (excluding the 5 cm sealing). The excavation would be done by subphases with passes of 1.0 to 2.0 m extending the support as the excavation progresses.

The portals section (S-E) was considered of "heavy" type, being in a superficial zone, more weathered and decompressed than the interior of the groundmass, due to the previous excavation of the portals slopes and to the reduced tunnel overburden. The proposed S-E section consists of a heavy umbrella of micropiles 20 m long, 150 mm drill diameter, spaced 0.5 m between their axes and reinforced with steel pipes 110 mm diameter, 8 mm thick and filled with mortar. The sequence of excavation and support for this section would be similar to that of the S-III with the difference that the steel ribs placed below the umbrella would be of type HEB-180.

1 Both sections have an inverted concrete arch with electro-welded mesh
2 150x150x6 mm. The geometry of the section influences the distribution of
3 stresses acting on the support, as a more circular shape reduces the risk of
4 developing traction in the concrete, making more efficient a section with an
5 inverted arch than a flat invert, especially in terrains of poor geotechnical quality.
6
7 After the excavation of the bench, the construction continued with the excavation
8 of the inverted arch and the concreting of the ribs foundation pad, as well as
9 starting the lining ring. A summary of the support types defined for the tunnel is
10 presented in the Table 2.
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22 Table 2 Summary of support types used in Ampurdán tunnel
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27 **Instrumentation of the tunnel**

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30 In the New Austrian Tunnelling Method, appropriate auscultation and systematic
31 control of the excavation works and support is essential, requiring the
32 implementation of geometric and topographical monitoring, the measurement of
33 convergences and the auscultation of cross sections. Geometric and topographical
34 monitoring was based on external triangulation and the constant checking of the
35 internal position of the tunnel axis and on the layout of the support elements.
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41 The convergence measurements inside the tunnel were controlled in sections
42 equipped with two bolts in the sidewalls for the use of strain gauge tape and a
43 topographic target in the crown. This allowed the control relative movements in
44 the sidewalls, and absolute movements in the crown.
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49 In auscultation cross sections, the control elements are (Fig. 3):
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- 51 • Total pressure cells: 3 units arranged between the support and the
52 permanent lining in the crown and sidewalls.
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- 54 • Rod strain gauges: 3 units installed in the crown and sidewalls. Each strain
55 gauge comprised 3 rods of 2, 4 and 6 m in length, measured from the inner side of
56 the support.
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4 Fig. 3. Sketch of the cross-section monitoring arrangement. TPC - total pressure cell
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9 10 **The construction**

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12 The excavation works in Ampurdán tunnel began by the North portal (Fig. 1). The
13 support of the portal was done using rockbolts, 12 m long and 32 mm in diameter,
14 using a 3x2 m pattern, and 10 cm of shotcrete HP-30 reinforced with wire mesh
15 (150x150x6 mm).
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18 Since the start of the excavation, the engineering geology mapping of the tunnel
19 face was done by a resident geologist, aiming to determine the main geotechnical
20 features of the groundmass. He was also in charge of monitoring the
21 instrumentation installed inside the tunnel. The staff responsible for the technical
22 assistance to the tunnel (Alija 2010) also evaluated the data collected.
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25 The initial excavation works confirmed that the Miocene claystone is
26 homogeneous, occasionally with subhorizontal interbedded layers of sandstone, of
27 decimetric thickness. It is important to highlight that the sandstone layers, once
28 intersected, showed abundant flow of water, reducing progressively the flow,
29 without completely stopping, and requiring the use of temporary drainage to allow
30 the construction works on the tunnel. With rainy weather, the flow of the drains
31 increases rapidly, indicating a fissural interconnection from the surface until the
32 sandstone layers. In the portal area, due the presence of water, 50 Swellex
33 rockbolts, 4 m long (25 on each side), were placed in the sidewalls, 1 m apart
34 (between ribs) with nearly horizontal inclination. These rockbolts were dry-fitted
35 to avoid adding water into the groundmass.
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50 An incident that occurred during the excavation of the portal must be highlighted.
51 At the top heading level, with the support of the last operation in place, a portion
52 of the ground behind the shotcrete and the rockbolts fallen due to the internal
53 erosion of the sandstone layers.
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58 During the first 40 m of the tunnel excavation, there was a constant presence of
59 water flowing through the sandy layers. This influx reduced as the tunnel
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1 advanced, but continued throughout the entire construction period, requiring the
2 implementation of border gutters to drain the entrance area.

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4 Once the 20 m long micropiles umbrella was completed, a beam joining the
5 micropile heads was constructed, and the first steel rib HEB-180 of the tunnel (rib
6 0) was placed.
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10 In subsequent excavation operations towards the south portal, the terrain
11 homogeneity was confirmed, allowing obtaining a very well-defined excavation
12 section, without overhangs, leaving a front with great stability (Fig. 4).
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21 Fig. 4. Excavation front in progress, presenting high homogeneity and good stability
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25 The good geotechnical characteristics of the terrain, allowed obtaining RMR
26 values of 45-50, exceeding the project estimates (Fig. 5).
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29 The works commenced with 1 m passes, which could be increased up to 1.3 m
30 once past the first 40 m of the tunnel. The advance rate stood at an average of 6
31 m/day due to the good overall performance of the tunnel.
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40 Fig. 5. Variation of RMR values in passes 0 to the 220
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44 The instrumentation data showed an adequate behaviour of the claystone
45 groundmass, presenting in all sections a tendency towards stabilization (Alija
46 2010). Both relative measurements (strain gauge tape) and the absolute
47 measurements in the crown (topography) showed no significant variations once
48 the movements approached stabilisation. The lower accumulated convergence
49 movements could be associated with reduced weathering and reduced
50 decompression of the claystone materials.
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57 As far as support is concerned, the initial criteria based on the use of HEB ribs
58 and shotcrete with fibres was maintained. The use of an additional mesh
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(150x150x6 mm) to reinforce the shotcrete was considered suitable for this type of material and this procedure was considered a positive experience to be used in similar tunnels.

The problem: collapse of the tunnel

The construction works of the Ampurdán tunnel were stopped after 300 m of excavation due to a severe and instantaneous collapse phenomenon occurred in the first 90 m of the tunnel. The maximum registered deformation was located at 40 m from the entrance, corresponding to a downward movement of the tunnel crown and sidewalls of approximately 1.2 m (initial descent of 0.8 m and the rest during the days following the event). The deformation vanishes towards the tunnel interior and reduces toward the entrance.

The most obvious external evidence of the failure (Fig. 6 and 7) was the existence of a depression about 0.5 m deep at the top of the portal, over the tunnel axis, and two longitudinal fissures above the sidewalls, 30 m long, 0.2-0.3 m wide, and about 2 m deep. The surfaces of these two fissures present a subvertical arrangement with slight inclination toward the tunnel axis, extending from the surface towards the sidewalls, roughly at the level of the top heading (Fig. 6).

Fig. 6 Overview of the North portal after the collapse. The horizontal dashed line highlights the 0.5 m depression at the top of the portal. The dashed harrow points the downward movement. The vertical and inclined dashed lines indicate the two longitudinal fissures 30 m long, The circular arrow indicates the circular shape of the fissure above de tunnel lining

Another evident trace of the failure was the open fissure in the shotcrete at the portal face, immediately above the lining (Fig. 7) with the separation of the shotcrete from de ground. It could clearly be seen that part of the rockbolts were still tight to the collapsed ground and that the shotcrete partially maintained its position.

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2 Fig. 7. Fissure developed above the North portal lining.
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6 The shotcrete fissure in the portal face, above the micropiles umbrella, can be
7 interpreted as the result of the portal zone rotation, sinking more on the left side
8 than in the right side (Fig. 6 and 7). In the tunnel interior, it was possible to
9 appreciate the left mortar gutter totally raised by the downward movement of the
10 sidewalls (Fig. 8).
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21 Fig. 8 View of the provisional mortar gutter rotated upwards on the left side of the tunnel
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25 Some cracks were also noted surrounding the area close to the portal, related to
26 the rotation of the entire support, which despite de failure remained in good
27 condition.
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31 32 33 34 **Materials testing** 35 36

37 Aiming to understand the causes of the tunnel failure and to determine the
38 parameters responsible for the changes in the materials properties generating the
39 collapse, a complementary site investigation campaign was carried out, and
40 samples were collected to use in laboratory tests. The site investigation works
41 performed were (Fig. 9):
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- 46 • 2 vertical long boreholes outside of the tunnel, from the surface until the
47 sidewalls;
- 48 • 2 inclined short boreholes inside the tunnel in the area of the sidewalls;
- 49 • 5 test pits, separated 40 m, inside the tunnel, next to the foundation of the
50 steel ribs.
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Fig. 9 Layout of the complementary investigation campaign

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4 The long boreholes intended to identify the characteristics of the material
5 surrounding the tunnel.
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8 The site investigation inside the tunnel intended to identify and characterize the
9 materials distribution close to the foundation of the steel ribs and to obtain their
10 mechanical properties. In the samples collected, the natural moisture content and
11 deformability parameters were determined. In particular, 3 CU triaxial tests, 2 CU
12 direct shear tests, 6 UU direct shear tests and 21 moisture content tests were
13 carried out. The boreholes inside the tunnel showed the presence of about 30 cm
14 composed of brown sandy silt, saturated with water and presenting a high degree
15 of de-structuring and weathering. The samples of this level had an average natural
16 water content of 28%. Below this first level, the same material was less
17 weathered, with an average natural water content of 21%. The outside boreholes
18 detected an increase in the water content at the height of the foundation of the
19 ribs, with an average value of 20%.
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22 The direct shear tests and the triaxial tests executed on 11 intact samples, gave
23 similar results to those obtained in the initial geotechnical characterization.
24

25 Only after evaluating the results of the boreholes, it was decided to dig the five
26 test pits with two purposes: to verify the spatial distribution of the weathered
27 claystone and to obtain large size undisturbed claystone samples to study their
28 weatherability in presence of water. Once the weatherability was analysed, the
29 intention was to test the samples in weathered conditions as close as possible to
30 the parameters at the time of failure.
31

32 The test pits showed a weathered thickness of approximately 0.3 m easy to dig.
33 Below this level, it was easy to dig up to 0.5 m, but below that depth, it became
34 difficult to excavate using mechanical equipment. One day later the test pits began
35 to seep water and finally become totally clogged.
36

37 The information obtained allowed to infer that the deformation stopped at a depth
38 where the support reached unweathered material (Plaza Castel 2008).
39

40 For the study of the claystone decay, 4 samples were collected inside the
41 Ampurdán tunnel, in its central part, during the advance phase. The samples were
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1 the deepest possible to ensure unweathered conditions. The samples were tightly
2 packed in hard plastic bags, labelled and immediately transported to an accredited
3 laboratory. The tests conducted on these samples were: natural moisture content,
4 grain size analysis, Atterberg limits, Slake Durability Test (Franklin & Chandra
5 1972) and the Jar Slake Test of Lutton (Mohamad et al. 2011). All the samples
6 showed a high content of fine particles, above 90%. The samples were classified
7 as CL considering the unified soil classification system. The natural moisture
8 content of the samples collected in the pits ranged from 21.9% to 24.4%. The
9 plasticity values obtained were since 22.8% to 24.7%, being above the values
10 obtained in previous tests (18%). Based on the plasticity index, the samples were
11 classified as highly plastic soils (>15 %). The liquid limit was between 32.8% and
12 49.5%.

13
14 In relation to the samples used in the Slake Durability Test, the average result for
15 the first cycle was 12%, while 0% was obtained in the second cycle. This
16 indicates that the claystone is extremely weatherable in the presence of water,
17 degrading completely during the test, and therefore the slake durability test is not
18 suitable for this type of very sensitive claystone material. After the results
19 obtained in the previous test, the samples were submitted to the Jar Slake test, less
20 aggressive than the slake durability test, where the samples are only submitted to
21 cycles of wetting and drying. The result showed that all the samples, within 24 h
22 of being submerged in water became a mud (Gómez Ramírez 2006).

23
24 Another part of the analysis focused on the information provided by the
25 instrumentation before and after the tunnel collapse. From a detailed study of the
26 pre-failure information, it was concluded that based on the available data, the
27 failure couldn't be anticipated. The behaviour of the tunnel was good, with the
28 convergence plots showing a clear tendency towards stabilization, without any
29 significant variations once stabilised.

30
31 One day after the failure, a continuous topographic monitoring of the crown and
32 sidewalls was started, together with the installation of entire sections and
33 additional auscultation devices in the tunnel and slopes, allowing a precise control
34 of the tunnel's deformations. The evolution of the absolute movements of the
35 tunnel crown, before and after the failure are presented in Fig. 10.

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Considering the convergence evolution of the section presenting the largest vertical movement, convergence C-5, the following interpretation could be inferred (Alija 2010):

- The greatest vertical movement occurred at the moment of the collapse, and was around 80 cm;
- Following the collapse, the crown entered in a second phase of displacement, dropping 40 cm in 7 days, equivalent to a speed of about 6 cm per day;
- In a third and final phase of movement, the crown reduced its displacement until a very low rate, dropping only 1 cm in 29 days. The convergence graphs showed an "asymptotic" trend until complete stabilization 36 days after the failure.

Fig. 10 Evolution of the absolute movements of the tunnel crown before and after the failure.

Based on the evaluation and interpretation of all the information previously presented, the failure mechanism of the Ampurdán tunnel can be explained as follows:

- During the advance of the tunnel, some stress release had occurred around the excavation, inducing volume changes, decompression and the increasing of the water content of the claystone.
- In the north portal the groundmass foundation the lining become embedded in water, drained through the sandstone layers and recharged by circulation in the claystone fissures of the Ampurdán formation. The increase in the moisture content easily surpassed the plastic limit of the clays and therefore changed the geomechanical behaviour of the claystone in the foundation of the lining.
- The failure is attributed to the collapse of the foundation material after the water content increase, which significantly reduced the bearing capacity of the claystone. A generalized plasticization of the clays induced a strength loss and the shear failure of the ground at the ribs foundation level.

- The mineralogy of the clays, the degree of cementation and the microtexture could also had influenced the increase of the water absorption, and reduced the geomechanical behaviour of the claystone.

Finite element simulation

After the complementary site investigation, the sampling of the in situ materials and the evaluation of the instrumentation data, it was considered necessary to determine the changes in the geomechanical properties of the materials around the tunnel lining that caused the collapse.

In order to simulate the failure conditions, several attempts were made to run tests under conditions similar to those responsible for the failure, but due to the high weatherability of the claystone, the samples disintegrated in the presence of water, preventing the execution of the tests. Faced with the impossibility to do reliable laboratory tests, it was decided to use of a finite element model program to do a parametric study and thus to determine the evolution of the geotechnical parameters during the claystone weathering process. The stresses and strains induced by the excavation, as well as the identification of the variables that may had greater incidence in their development, could be inferred by the finite element model. The determination of the geotechnical parameters dependent on weathering is also vital for a suitable design of the construction procedures and the support solutions.

To compute the tunnel's elastoplasticity a numerical simulation was used. The models were computed using the finite element method based on the bi-dimensional PLAXIS software of the company PLAXIS B.V, considering the ground as a continuum space (Brinkgreve 2004). Finite element models can be plane strain or axisymmetric. In this case, since it could be assumed a uniform cross section, having uniform stress conditions and loads along the tunnel (direction z), flat deformation models (plane strain) were considered. Triangular elements of 15 nodes were used to model the tunnel, providing a fourth order interpolation for the displacements, and the numerical integration embrace twelve Gauss points (evaluation stress points).

1 For the support, a combined rigidity to flexion EI and the resistance to axial stress
2 EA was used. From these two parameters, an equivalent plate thickness was used.

3
4 The NATM was considered, taking into account the existence of a lining
5 composed of plates and an interface on the outside. The tunnel outline consisted
6 of arch shaped sections.
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10 For the calculation, a mechanical Mohr-Coulomb constitutive model was initially
11 considered, in order to deal with the same assumptions as those used during the
12 construction works. To use this model, it was also taken into account that the
13 collapse phenomenon occurred suddenly, and that on the days prior to failure no
14 convergence movements have been detected, nor absolute movements in the
15 crown.
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21 However, based on previous experience of other works with Plaxis, it is known
22 that with the Mohr-Coulomb model the same module is taken to load and
23 discharge, resulting in an excessive lifting of the ground (this situation was proved
24 with the numerical model). For this reason, the model that has been considered
25 was the Hardening Soil (Xu & Wang 2002), which assumes that the discharge
26 module is 3 times higher than the loading module. This model yielded more
27 realistic results for the present case study, and the deformation values were closer
28 to those actually obtained in the auscultation sections. The Hardening Soil model
29 is a variant of the hyperbolic model, formulated within the plasticity hardening by
30 friction. This model is suitable for the simulation of the behaviour of soft
31 sediments and clay soils. In contrast to a perfect elastoplastic model, in the
32 Hardening Soil, the creep surface is not fixed in the space of the main stresses,
33 since it can be expanded due to plastic deformations.
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45 The study of the tunnel's behaviour related to the variation of these properties was
46 the basis for the parametric study to determine the causes of the tunnel failure.
47 The geotechnical parameters initially used in the calculation for the materials
48 crossed by the Ampurdán tunnel are presented in Table 3.
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56 Table 3 Initial pre-failure parameters considered in the calculations
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As section C-5 presented the largest deformation when the tunnel collapsed, it was chosen for the calculations. In this section, the tunnel overburden is around 30 m. The support parameters for the modelling have been adapted according to those corresponding to section type S-III. The advance line was considered at a height of 6.0 m being the excavation and support phases analysed according to the implementation of the works: top heading advance in one phase with 1 m passes, and bench demolition in two phases with 2 m passes.

The complete calculation sequence of excavation and support for the Ampurdán tunnel was composed of the following phases:

Phase 0. Initialization of the model.

Phase 1. Excavation of the top heading.

Phase 2. Support of the top heading.

Phase 3. Excavation of the left bench.

Phase 4. Support of the left bench.

Phase 5. Excavation of the right bench.

Phase 6. Support of the right bench.

Results and discussions

Once the model was set to the initial situation, a parametric study was undertaken, reducing the values of the materials properties, until obtaining 80 cm of subsidence in section C-5. The variations of the geotechnical parameters due to the presence of water that affected the materials in which the tunnel was excavated were also studied. The reduction of the parameters for analysis was carried out by percentages of reduction. As a working assumption, it was considered that the percentage reductions should not affect equally all the parameters analysed. To carry out the study as closely as possible to the actual conditions, typical values currently used for clays, with different degrees of consistency were taken into account (Ministerio de Fomento 2008).

The first step consisted in the verification of the model in the initial unchanged situation. This analysis was named: Study 1 Initial Situation, and the

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corresponding results of the presumed strain and stress were obtained for the top heading advance phase. However, in phase 5 of Study 1, excavation of the right bench, the strain on the left wall reached 71 mm, and the crown reached 36 mm (Fig. 11). For the initial situation, the calculation of the stable phase 6 was not achieved since the tunnel destabilized.

Fig. 11 Schematic by arrows of total displacements at the end of phase 5 for Study 1

This situation that did not correspond to the behaviour of the tunnel required the reformulation of the values assigned to the Ampurdán claystone. The values considered were based on laboratory tests for the strength parameters, and on "in situ" tests for the deformation parameters. Considering the high weatherability of these materials, it seems reasonable to believe that during the execution of the boreholes the samples could have been weathered, causing a significant reduction in strength values. On the other hand, it is important to highlight that preconsolidated sediments typically present a decrease in strength values obtained in the laboratory, in respect to natural conditions, since rupture occurs by previous fissures developed during decompression, while collecting the sample (González de Vallejo et al. 2002). The good stability of the tunnel during construction is also evidence that the strength parameters considered in Study 1, were lower than the real ones. The good results of the auscultation campaign and the RMR values in the excavation face also supported that idea.

Based on the previous assumptions, it was concluded that the values for the Young modulus obtained in the pressuremeter tests could be closer to the real values of the material under natural conditions. Thus, taking into account the routine geotechnical correlations used in the study phases, it could be considered that for a deformation modulus of 200 MPa, a value of 100 kPa for the effective cohesion and of 35° for the effective friction angle could be acceptable (Jiménez Salas et al. 1976; Justo Alpañes 1968). A new modelling study (Study 2: Modified Initial Situation) was undertaken using these last parameters. In this study, phase 6 of support of the right bench can be reached with acceptable results of 10 mm

1 maximum accumulated displacement in the sidewalls, and 14 mm in the crown.
2 Once the initial situation model was determined to correspond to the real
3 conditions of the tunnel before the subsidence processes, the reduction in the
4 geotechnical characteristics of the materials that gave rise to the tunnel subsidence
5 was done. The parameters considered the most relevant, are the following:
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- 8 • Secant elastic modulus: E_{50}
- 9 • Effective cohesion: c'
- 10 • Internal friction angle: ϕ'
- 11 • Apparent density: ρ

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20 The reductions proposed for the consecutive parametric studies considered is
21 presented in Table 4.
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25 Table 4 Values considered for the parametric study and sequential reduction for each parameter
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32 The displacements computed at the end of Study 3, were similar to those of Study
33 2, with a slight increment. In this case, the displacement in the crown reached 15
34 mm towards the tunnel interior. The ground reached 14 mm and the radial
35 deformations up to 11 mm (Table 5).
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42 Table 5 Accumulated deformation in the crown for the studies performed
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49 For Study 6 (Fig. 12) the acceptable parameters were considered those of a
50 cohesive soil of hard or rigid consistency. This study showed a significant
51 increase in the deformations at the end of stage 6 as compared to the previous
52 ones. Thus, from the 22 mm in the crown for Study 5, in study 6, 32 mm were
53 reached (Table 5). In the floor, there was an uplift increment of 13 mm from
54 Study 5 (23 mm) to Study 6 (36 mm). The differences between the radial
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1 deformations of both sidewalls are accentuated in this study, reaching up to 45
2 mm on the left and 32 mm on the right.
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9 Fig. 12 Total displacements produced at the end of phase 6 in study 6
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13 In Study 9, the reduction in the ground parameters lead to the failure of the tunnel
14 in the bench excavation phase. At the end of phase 2 (support in top heading) the
15 deformations surrounding the tunnel were very high, particularly in the floor, next
16 to the sidewalls, reaching deformations around 260 mm.
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21 Considering values representative of a cohesive material of soft consistency, that
22 were used in Study 10, the excavation of the top heading (phase 1) is close to
23 failure. The floor deforms 95 mm into the tunnel, while the crown drops about 20
24 mm. During the support of the top heading, the tunnel collapse was produced.
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27 Figure 13 presents the deformations towards the interior of the tunnel, reaching
28 800 mm in the crown, and in the floor in the foundation area of the ribs, reached
29 1430 mm.
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39 Fig. 13 Diagram of total displacements produced up to failure in phase 2 of Study 10
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43 A summary of the accumulated values obtained for the crown deformation in the
44 Studies 2 to 10, is presented in the Table 5.
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47 In Table 5 and in Fig. 14 it is evident how the deformations increment are low
48 until Study 5, with an almost linear tendency in the six phases of excavation and
49 support. In Study 5, the deformations in the crown slightly increase after the
50 support phase of the left blasting (phase 4). After Study 6, the deformations
51 dramatically increase, reaching destabilization during the bench excavation, in
52 Studies 8 and 9. In Study 10, the excavation phase is still stable, but the failure
53 arises during the support phase.
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Fig. 14 Evolution of displacements in the crown for each study, by phases

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4 Considering the parametric study performed, it can be stated that the parameters
5 of Study 10 are representative of the material of Ampurdán Claystone Formation
6 (M_A) in weathered conditions, as they allowed reproducing the deformations
7 measured in situ after the tunnel failure. These parameters are summarized in
8 Table 6.
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16 Table 6 Comparison between assumed intact parameters and computed parameters at failure of
17 Ampurdán claystone Formation M_A
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24 Comparing the values presented in Table 6, considered representative of the in
25 situ materials properties at the time of the tunnel failure (Study 10), with the intact
26 pre-failure parameters assumed in the finite element simulation (Study 2), they
27 correspond to reduce the initial values to 69 % in the apparent density and in the
28 effective friction angle, to 12 % in effective cohesion and to 4% in the Young
29 modulus.
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39 **Conclusions and recommendations**

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42 The abundance of water in the groundmass of the Ampurdán tunnel allowed the
43 weathering of the mudstone softrocks on which the tunnel support relies, reducing
44 their geotechnical properties, and causing the tunnel failure.
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48 The fact that the usual monitoring indicators used to evaluate the behaviour of the
49 tunnel, did not gave warning about the failure that subsequently occurred,
50 highlight the importance of using preventive procedures in the tunnels excavated
51 in similar softrock materials. Therefore, the evolution and quantification of the
52 geomechanical properties of softrocks, when subjected to excavation,
53 decompression, and exposure to water, is of particular importance to determine
54 the actions to be taken during the tunnel's construction.
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1 The support procedure installed in the tunnel appeared to be suitable for the initial
2 conditions of the materials, but became insufficient when the properties of the
3 groundmass materials degraded. The weatherability tests showed that due to the
4 presence of water, a support suitable for a terrain of medium-soft consistency
5 should have been used.
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9 Special attention should be given to the process of defining the geotechnical
10 parameters in softrocks, requiring a depth study for a correct understanding of
11 their behaviour. They are materials that in unaltered conditions present good
12 geotechnical behaviour, facilitating the excavation of the tunnel, but when
13 weathered, because of unloading and wetting, may suffer important loss in
14 strength.
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21 When a tunnel is to be built in softrocks, during the design it is necessary to
22 define adequate construction and supervision methods, adjusted to the most
23 unfavourable conditions. The contemporary modelling tools using finite elements
24 are especially useful, allowing the simulation of the behaviour of the structure
25 whilst adjusting the geotechnical characteristics of the groundmass material.
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31 The parametric study performed in the present case study allowed to simulate the
32 deformations measured in situ after the tunnel failure. The final results of the
33 finite element modelling were considered representative of the material of the
34 Ampurdán claystone Formation (M_A) in weathered conditions. All the parameters
35 at failure are worse than those of intact situation, corresponding to a wide range of
36 reductions to 69 % of the original effective friction angle and the apparent density,
37 to 12 % of the effective cohesion, and only to 4 % of the initial Young modulus.
38 This example stresses the fact that a detailed study must be done for each tunnel
39 geometry and groundmass material in order to reach suitable behaviour
40 characterization and adequate construction solutions.
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49 The experience obtained during the construction of the tunnel in the claystone
50 materials, allowed proving that the finite element method is a suitable "tools" to
51 adjust the design, helping to minimize the number of incidents during the
52 excavation and support works for tunnels built in similar materials.
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2 FCT – Fundação para a Ciência e a Tecnologia under the project PEst-
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FIGURES

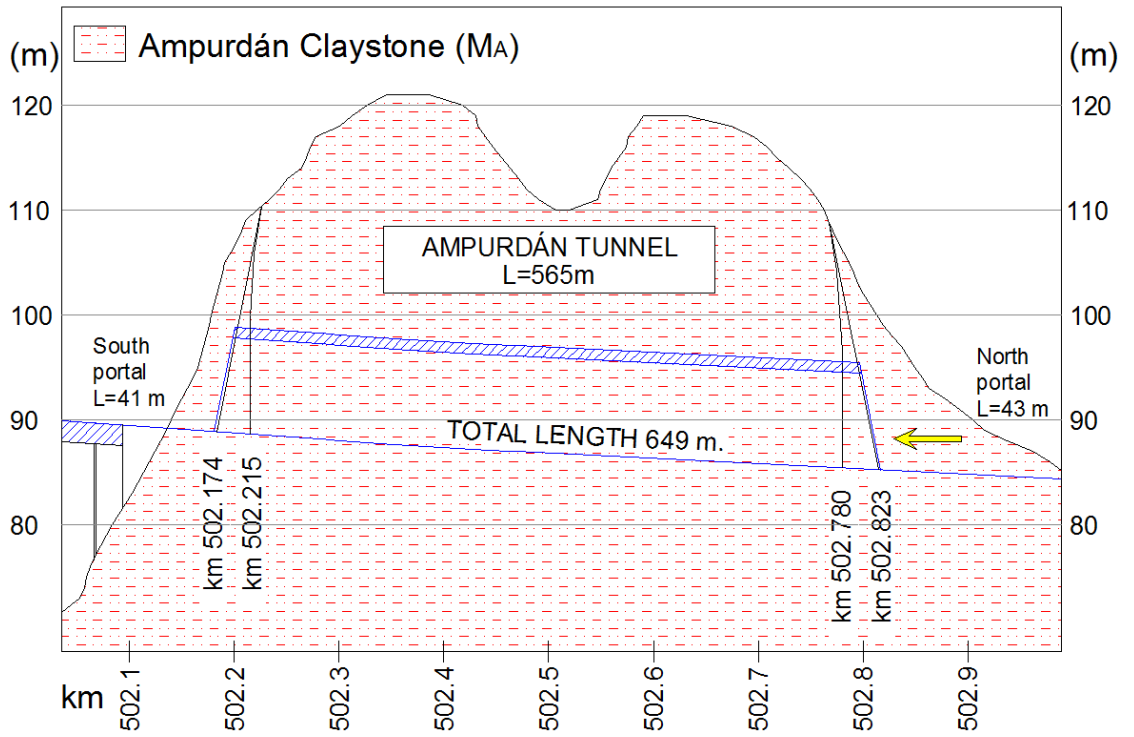


Fig. 1 Profile of the Ampurdán tunnel. The arrow points the advance direction

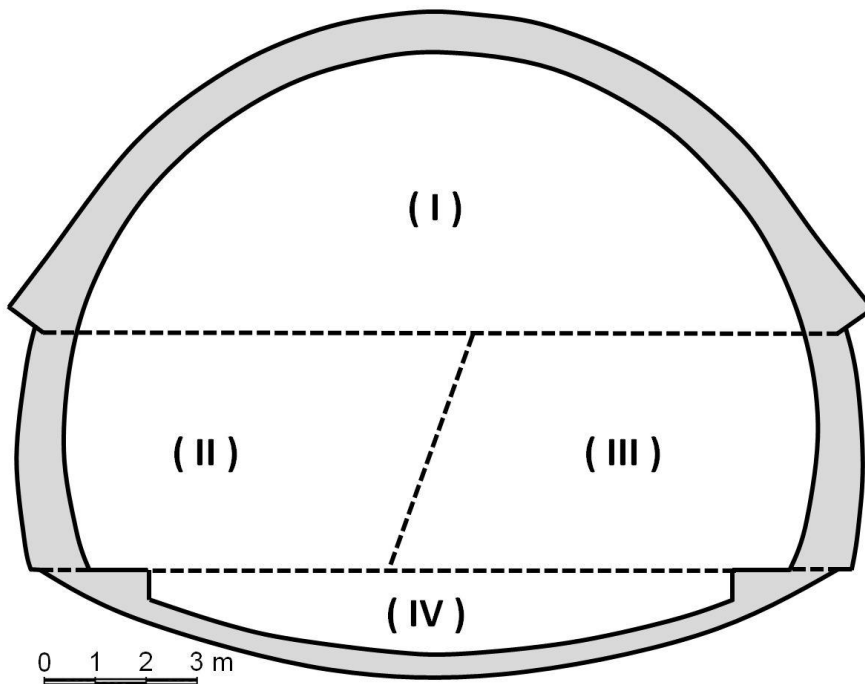
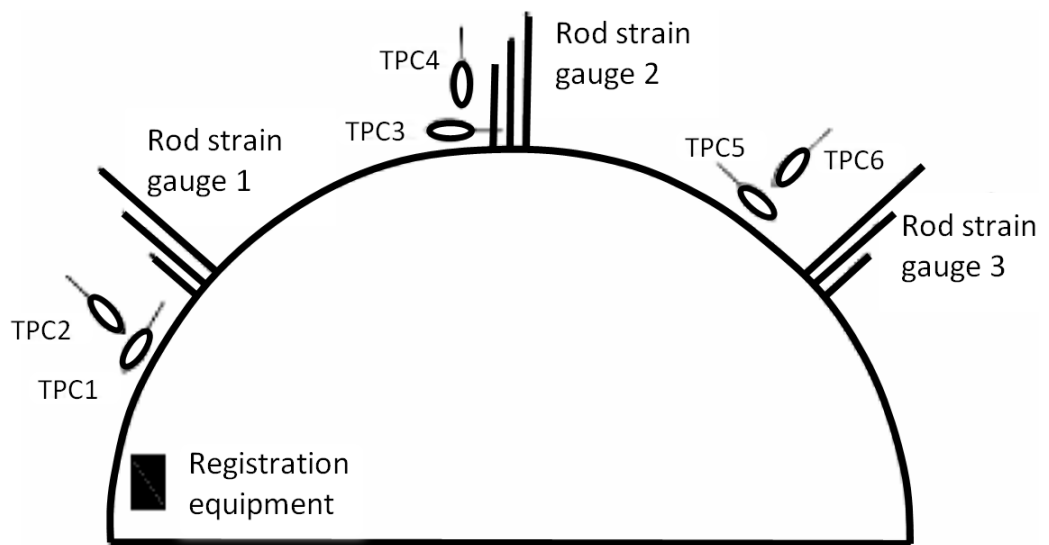


Fig. 2 Cross-section of the tunnel with elephant's foot. I - top heading, II and III - bench, IV - invert



21 Fig. 3. Sketch of the cross-section monitoring arrangement. TPC - total pressure cell.



53 Fig. 4. Excavation front in progress, presenting high homogeneity and good stability

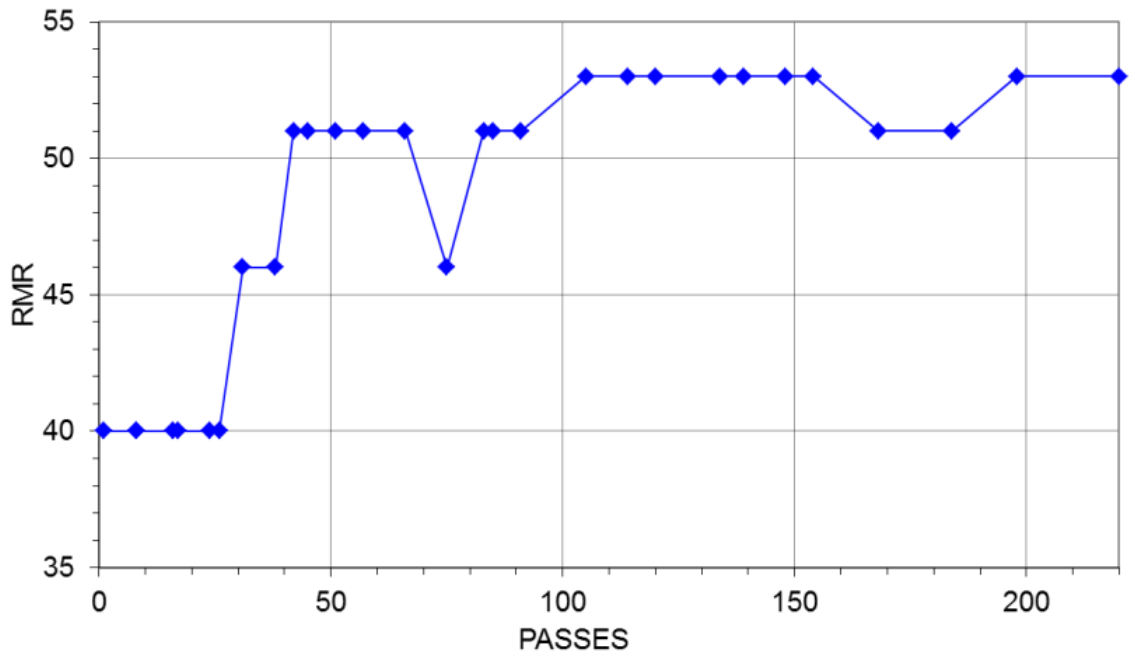


Fig. 5. Variation of RMR values in passes 0 to the 220

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Fig. 6 Overview of the North portal after the collapse. The horizontal dashed line highlights the 0.5 m depression at the top of the portal. The dashed harrow points the downward movement. The vertical and inclined dashed lines indicate the two longitudinal fissures 30 m long, The circular arrow indicates the circular shape of the fissure above de tunnel lining



Fig. 7. Fissure developed above the North portal lining.



Fig. 8 View of the provisional mortar gutter rotated upwards on the left side of the tunnel

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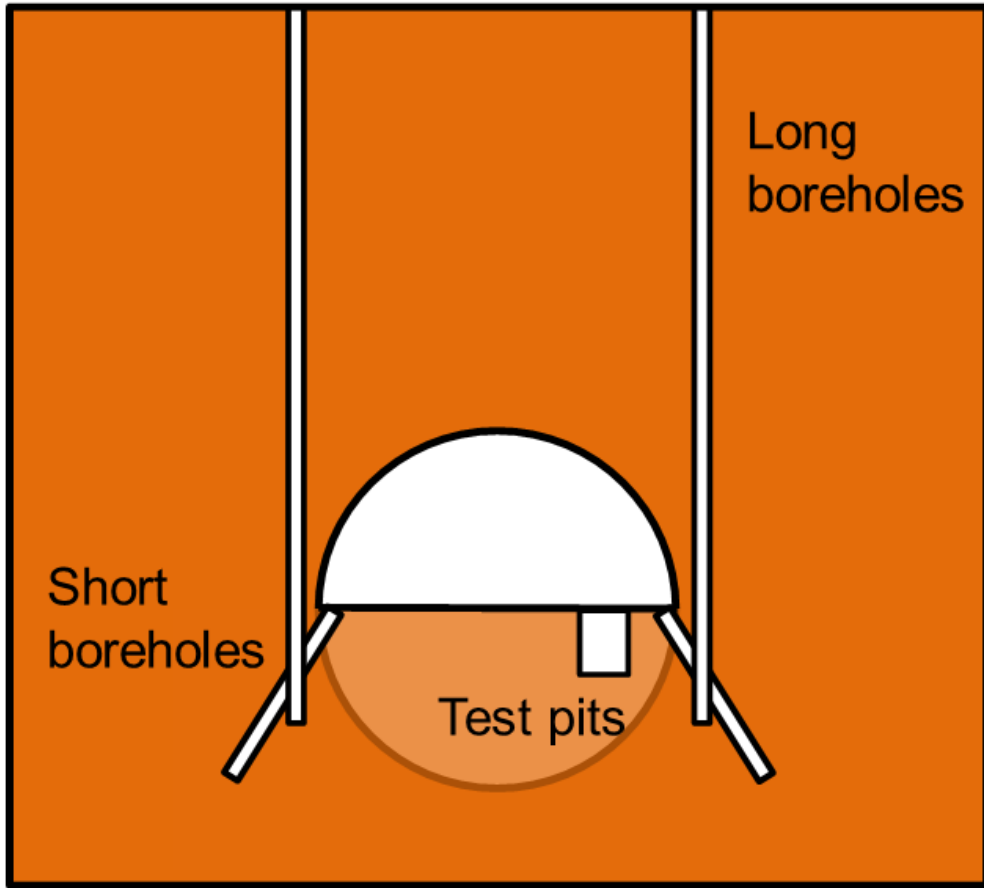


Fig. 9 Layout of the complementary investigation campaign

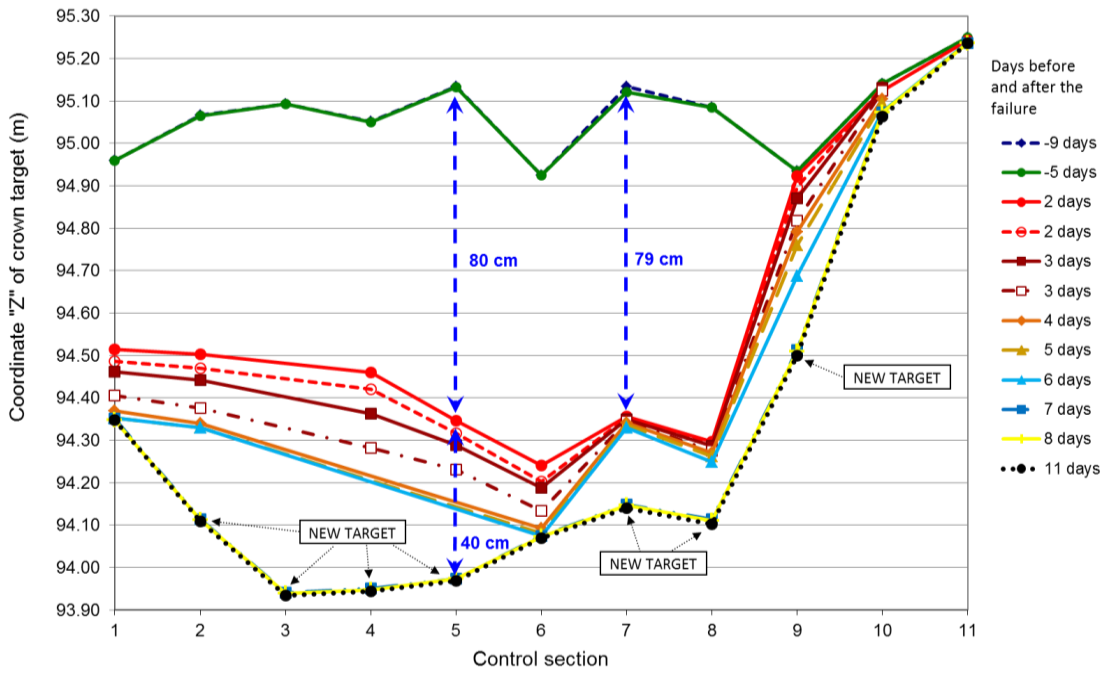


Fig. 10 Evolution of the absolute movements of the tunnel crown before and after the failure

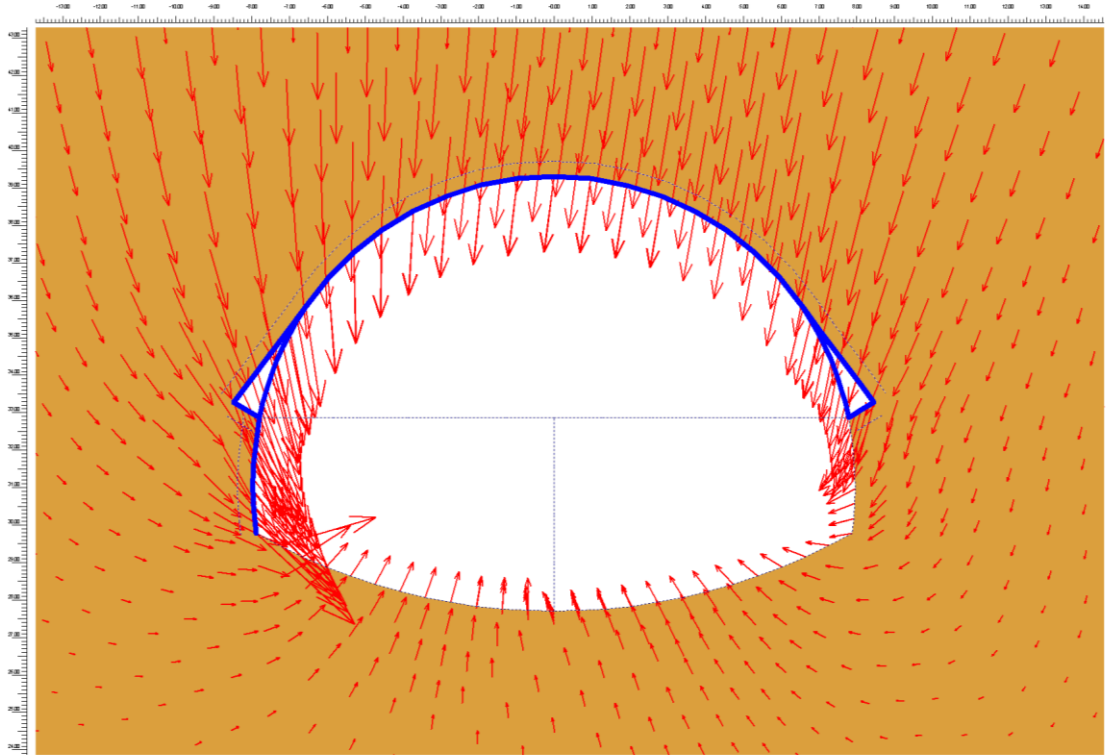


Fig. 11 Schematic by arrows of total displacements at the end of phase 5 for Study 1

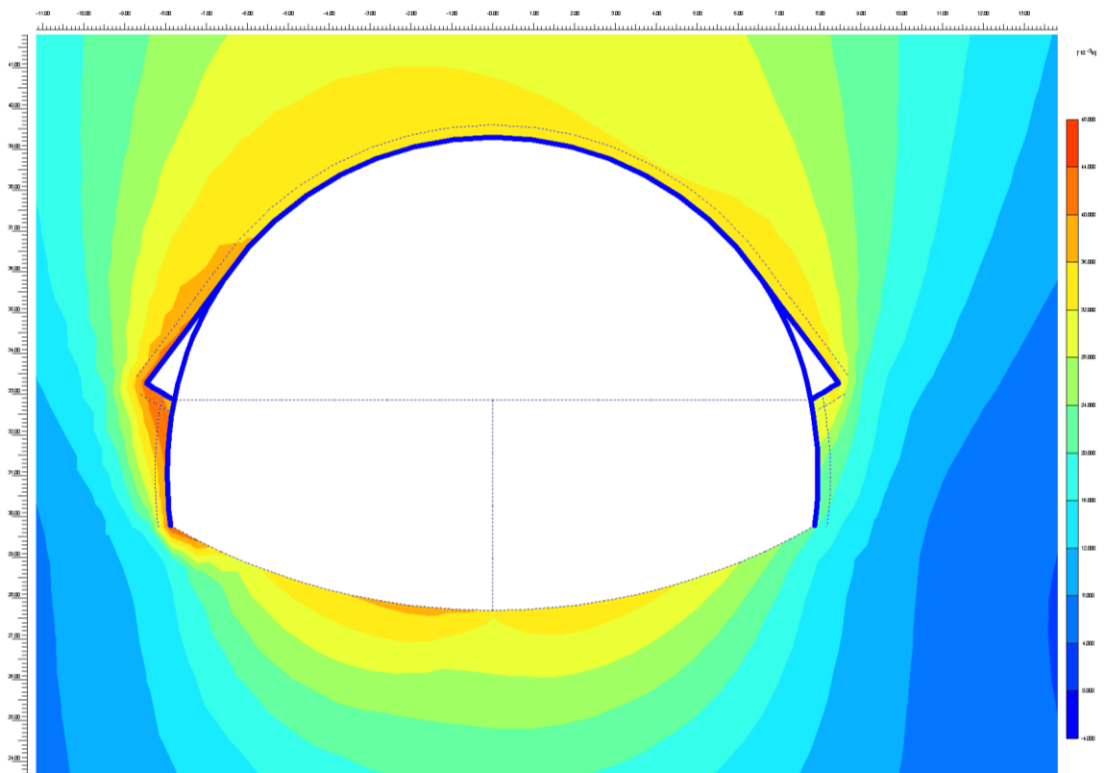


Fig. 12 Total displacements produced at the end of phase 6 in study 6

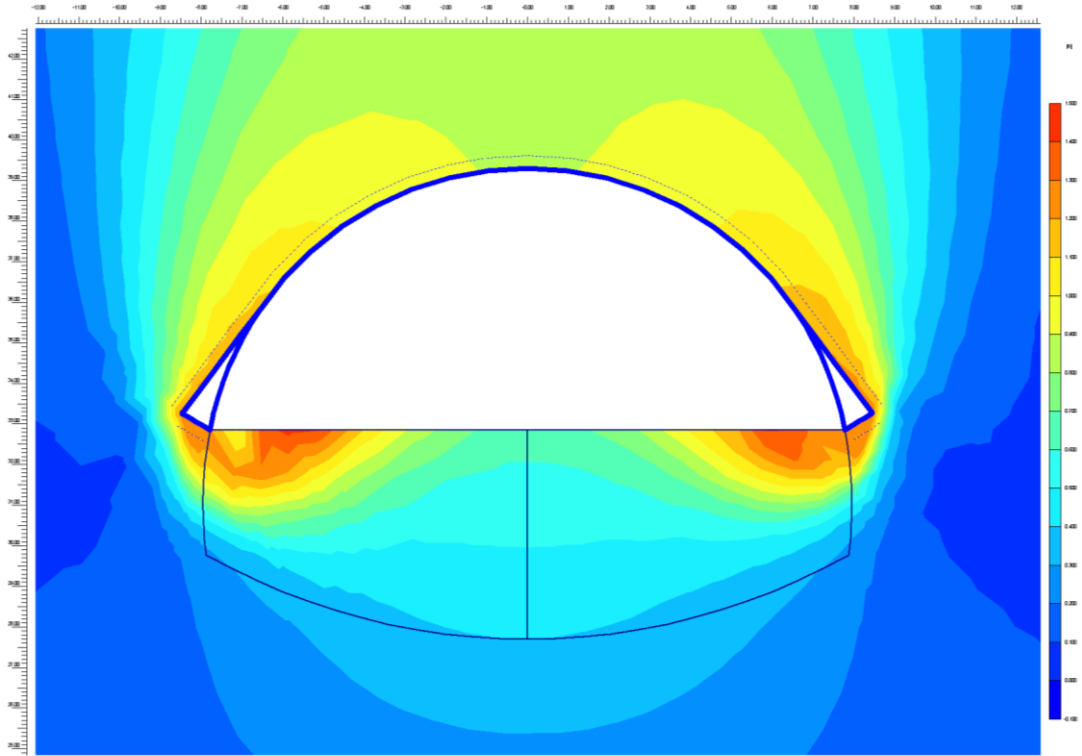


Fig. 13 Diagram of total displacements produced up to failure in phase 2 of Study 10

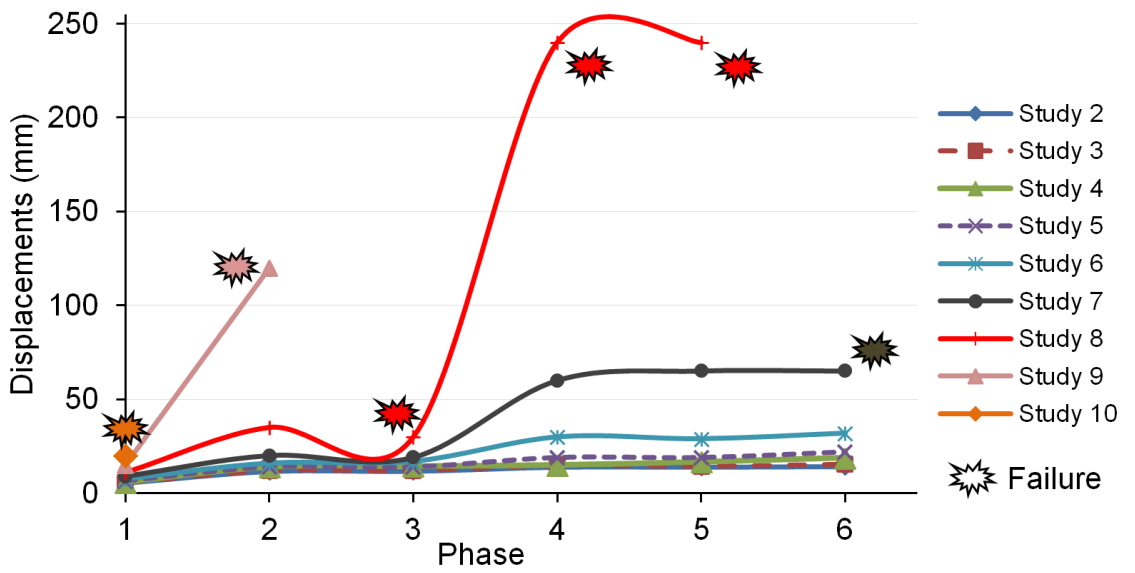


Fig. 14 Evolution of displacements in the crown for each study, by phases

TABLES

Table 1 Summary of the geotechnical parameters considered in the design of the Ampurdán tunnel

Formation	Apparent density	Uniaxial compressive strength	Effective cohesion	Internal friction angle	Deformation module	Thrust at rest coefficient	Poisson's coefficient
	γ_{ap} (kN/m ³)	q_u (MPa)	c' (kPa)	ϕ' (°)	E (MPa)	k_o	ν
Ampurdán Claystone Formation (M _A)	21	0.36	40	30	200	0.8	0.3

Table 2 Summary of support types used in Ampurdán tunnel.

Section type	Terrain	Length (m)	RMR	Zone / phases; passes	Shotcrete with fibers (cm)	Steel ribs
S-III	M _A	505	20-30	Top heading / 1; 0.5-1 m Bench / 2; 1-2 m	5 seal + 30 HM-35	HEB-160
SE	Portals	2 x 60	20	Top heading / 1; 0.5-1 m Bench / 2; 1-2 m	5 seal + 30 HM-35	HEB-180

Table 3 Initial parameters used in the calculations

Formation	Apparent density	Residual cohesion	Residual friction angle	Secant modulus	Angle of dilatancy	Poisson's coefficient
	γ_{ap} (kN/m ³)	c_r (kPa)	ϕ_r (°)	E_{50} (MPa)	ψ (°)	ν
Ampurdán Claystone Formation (M _A)	21	40	30	200	5	0.2

Table 4 Values considered for the parametric study and sequential reduction for each parameter

Study	Δ R.F. (%)	γ_{ap} (kN/m ³)	Δ R.F. (%)	c_r (kPa)	Δ R.F. (%)	ϕ_r (°)	Δ R.F. (%)	E_{50} (MPa)	Cohesive soil consistency
1	0	21.0	0	40.0	0	30.0	0	200	
2	0	21.0	0	100.0	0	35.0	0	200	-
3	1	20.8	5	95.0	1	34.7	7	186	-
4	2	20.4	10	85.5	2	34.0	14	160	-
5	3	19.8	15	72.7	3	32.9	21	126	-
6	4	19.0	20	58.1	4	31.6	28	91	Stiff
7	5	18.0	25	43.6	5	30.0	35	59	Stiff
8	6	16.9	30	30.5	6	28.2	42	34	Medium
9	7	15.8	35	19.8	7	26.3	49	17	Medium
10	8	14.5	40	11.9	8	24.2	56	7.7	Soft

Δ R.F. = Reduction Factor in consecutive Studies

Table 5 Accumulated deformation in the crown for the studies performed

Study	Deformation in the crown (mm)					
	F1	F2	F3	F4	F5	F6
2	5.3	11.5	11.6	13.8	13.8	14.1
3	5.5	12.0	12.0	14.8	14.4	15.4
4	5.9	14.0	14.4	15.0	16.7	18.9
5	6.6	14.0	14.0	19.0	19.0	22.0
6	7.7	16.0	17.0	30.0	29.0	3.02
7	9.0	20.0	19.0	60.0	65.0	65.0
8	11.0	35.0	30.0	240.0	240.0	-
9	15.0	120.0	-	-	-	-
10	20.0	-	-	-	-	-

Table 6 Comparison between assumed intact parameters and computed parameters at failure of Ampurdán claystone Formation M_A

Ampurdán Claystone Formation (MA)	Apparent density	Residual cohesion	Residual friction angle	Secant modulus
	γ_{ap} (kN/m ³)	c_r (kPa)	ϕ_r (°)	E_{50} (MPa)
Assumed intact parameters (Study 2)	21.0	100	35	200
Computed parameters at failure (Study 10)	14.5	11.9	24.2	7.7
% of initial parameters at failure	69 %	12 %	69 %	4 %